GOVERNMENT OF BANGLADESH

BANGLADESH WATER DEVELOPMENT BOARD

MEGHNA RIVER BANK PROTECTION

SHORT TERM STUDY

IDA Credit 1870 BD (Part D), March 1990

FINAL REPORT

VOLUME IV

ANNEX: D SCALE MODEL STUDIES

~~ANNEX: E MATHEMATICAL MODEL STUDIES~~

February 1992

HASKONING, Royal Dutch Consulting
Engineers and Architects

...iation with:

...YDRAULICS
...DESH ENGINEERING & TECHNOLOGICAL SERVICES LTD.

2

GOVERNMENT OF BANGLADESH

BANGLADESH WATER DEVELOPMENT BOARD

FAP 9B

MEGHNA RIVER BANK PROTECTION

SHORT TERM STUDY

IDA Credit 1870 BD (Part D), March 1990

FINAL REPORT

VOLUME IV

ANNEX: D **SCALE MODEL STUDIES**
ANNEX: E **MATHEMATICAL MODEL STUDIES**

February 1992

HASKONING, Royal Dutch Consulting Engineers and Architects

in association with:

DELFT HYDRAULICS
BANGLADESH ENGINEERING & TECHNOLOGICAL SERVICES LTD.

PREFACE

The Meghna, one of Bangladesh' major rivers, flows through the eastern part of Bangladesh and discharges into the Bay of Bengal.

Like other rivers in Bangladesh the Meghna erodes it banks in many points and this erosion has assumed an alarming magnitude since the severe floods of 1987 and 1988. Consequently, a number of locations requires prompt attention to prevent further damage or even events of a catastrophic nature.

This Final Report describes the surveys, studies, designs, cost estimating and economic evaluation carried out during 1990-1992 as part of the Short Term Study (FAP-9B) for Meghna Bank Protection.

The Report consists of seven volumes comprising a Main Report and eight Annexes A to G and I. Some Annexes are accompanied by a series of APPENDICES containing detailed information or supporting data relevant to them.

Vol I			Main Report
Vol II	Annex	A :	Hydrology
		B :	River Morphology and Geomorphology
Vol III	Annex	C :	Geotechnical Investigations
Vol IV	Annex	D :	Scale Model Studies
		E :	Mathematical Model Studies
Vol V	Annex	G :	River Bank Protection
Vol VI	Annex	F :	Economics of Protection Works
Vol VII	Annex	H :	(not used)
		I :	Environmental Impact Assessment.

INTRODUCTION TO THE PROJECT

1. Background

There are three major rivers in Bangladesh; the Ganges, the Brahmaputra and the Meghna. Originating form Assam in India, the Meghna River flows through the eastern part of Bangladesh and discharges into the Bay of Bengal. The Meghna River drains an area of 77,000 km^2, of which about 46,500 km^2 is located in Bangladesh. The major contributors to the river upstream of Bhairab Bazar are the Boulai, the Surma and the Kushiyara rivers, covering an area of 62,960 km^2. The Ganges joins the Brahmaputra near Aricha and thereafter takes the name of the Padma. The Padma joins the Meghna at Chandpur. The Lower Meghna River conveys the melt and rain water form the Ganges and Jamuna basins, combined in the Padma River, and from the Upper Meghna basin to the sea. The total catchment area is about 1,637,000 Km^2. Maximum flows can be as high as 160,000 m^3/s. The major contribution of the discharge originates from the Jamuna River (annual average 19,642 m3/s) and the Ganges River (annual average 10,874 m^3/s).

The reach of the Meghna River from Bhairab Bazar to Haimchar is about 160 km in length. Width of the river varies from 1 km to more than 10 km. The river channel is more or less well defined upstream of its confluence with the Padma and is braided in the reach downstream of Chandpur. The river is considerably deep all along and the depth ranges to 35 m in the bends. The river bed and banks consist mainly of clayey-silt which is often loosely packed and is susceptible to liquefaction at some places. Of the three major rivers, the Meghna carries relatively less sediment. The velocity of flow of the river is high during monsoon. The river banks are also subjected to heavy wave action at some points.

Like other rivers in Bangladesh, the Meghna erodes its banks in many points. Erosion at the Meghna since the severe flood of 1988 has assumed an alarming proportion at the following locations which require prompt attention.

- The Railway bridge at Bhairab Bazar;
- Bhairab Bazar Township along the right bank;
- Maniknagar; along the left bank, falling within the proposed Gumti - Phase II Project;
- Meghna R & H Bridge;
- Eklashpur (near Meghna-Dhonagoda Project);
- Chandpur Town;
- Haimchar (adjacent to Chandpur Irrigation Project);

The Dhaleswari River, a tributary of Meghna, has been eroding its right bank at Munshiganj for quite some time and has threatened the existence of Munshiganj Town.

2. Meghna River Bank Protection -Short term Study

The study of possible bank protection works at critical locations along the Meghna river commenced officially in September 1990 when BWDB, Bangladesh Water Development Board commissioned HASKONING, Royal Dutch Consulting Engineers and Architects in association with DELFT HYDRAULICS and BETS, Bangladesh Engineering and Technological Services, to carry out the Meghna River Protection Short Term Study, financed under Credit IDA BD-1870, Part D.

The objectives of the study are:

- to provide short term measures for protection against erosion for seven locations on the Meghna river and one location on the Dhaleswari;

- to gradually implement a coherent and phased programme of works, aiming at the control of erosion on the defined stretches of the rivers Meghna and Dhaleswari. The protection of the locations indicated above should logically fit in this programme.

The Inception Phase started in November, 1990 with the mobilisation of the Expatriate Consultants. During the Inception Phase, the inter-action between this study and Flood Action Plan (FAP) Components was identified and maintained as far as possible.

The Meghna River Bank Protection Short Term Study, is now one of the **main components** of the Flood Action Plan for Bangladesh (FAP-9B. MEGHNA LB PROTECTION PROJECT), as included in the Review Report FPCO, December, 1990.

It has been recognised that during the Inception Phase, due to the internal and international situation during November 1990 to February 1991, delays were experienced, hampering the normal development of the activities planned. Therefore, activities in the critical path of the study were delayed (i.e, hydrometric surveys, geotechnical investigations, model investigations at RRI).

Furthermore, during the first phase of the project it became more and more clear that the inclusion of the flood season in the survey would considerably improve the designs of the protection works, the Consultants were supposed to submit at the end of the Study. Moreover, strengthening of the relation with the studies of the Bangladesh Action Plan for Flood Control (FAP) would also have a positive contribution to the outcome of this project. Therefore the BWDB instructed the Consultants to review and update the work plan taking note of the flood season of 1991 and the aforementioned studies of FAP.

As part of the Study a priority ranking was established. Accordingly, it was decided:

- to carry out a feasibility study, detailed designs and tender documents for bank protection works at the following locations:

    - Bhairab Bazar Township and Railway Bridge;
    - Munshiganj Town located on the Dhaleswari River;
    - Chandpur Town;

- to carry out a full feasibility study and prepare tender documents for bank protection works in the following locations:

    - Eklashpur;
    - Haimchar;

- and only a pre-feasibility study for:

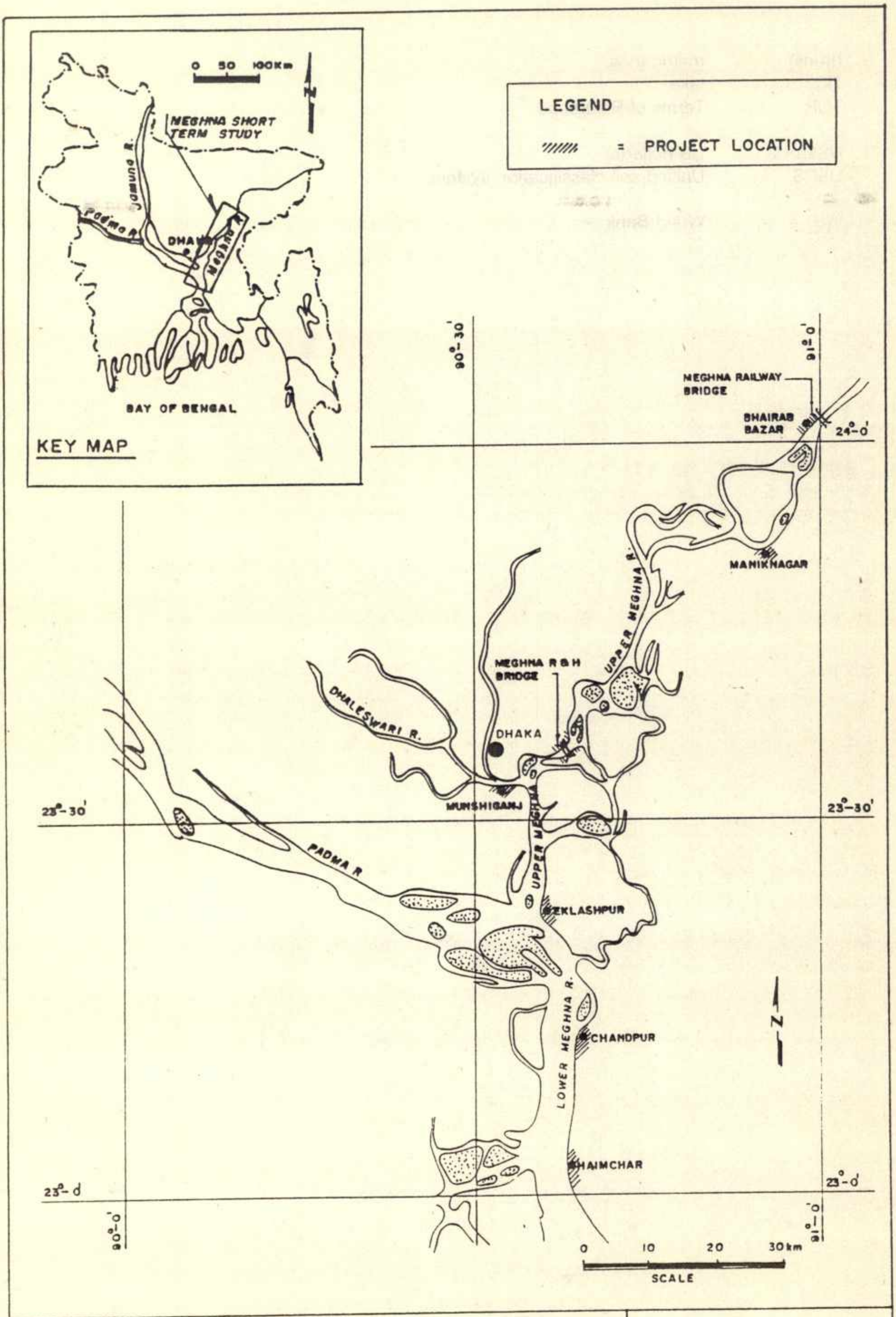
  - Meghna Roads & Highways Bridge;
  - Maniknagar, part of Gumti Phase II Project.

This Final Report submitted in accordance with the (Revised) Terms of Reference comprises all feasibility studies carried out as well as the detailed designs for bank protection works at the three locations mentioned above.

ABBREVIATIONS AND GLOSSARY OF TERMS

ADB	Asian Development Bank
BCSIR	Bangladesh Council for Scientific and Industrial Research
BBS	Bangladesh Bureau of Statistics
B/C	benefit cost ratio
BCL	Bangladesh Consultants Limited
BETS	Bangladesh Engineering and Technological Services Ltd
BH	Bore hole
BIWTA	Bangladesh Inland Water Transport Authority
BIWTC	Bangladesh Inland Water Transport Corporation
BOD	Biological Oxygen Demand
BR	Bangladesh Railway
BS	British Standards
BUET	Bangladesh University of Engineering and Technology
BWDB	Bangladesh Water Development Board
^{0}C	degree Celsius
CC blocks	concrete blocks
CIF	Cost, insurance and freight
CPT	Cone Penetration Test
Crore	10,000,000
DH	Delft Hydraulics (Netherlands)
Dollar (US)	taken at an exchange rate of Tk.36 for the Study
EIA	environmental impact assessment
EIRR	economic internal rater of return
FAO	Food and Agricultural Organization (United Nations)
FAP	Flood Action Plan
F/C	foreign currency
Fig(s)	figures(s)
FML	fortnightly mean water level
FPCO	Flood Plan Coordination Organization
g	acceleration due to gravity
GL	ground level
ha	hectare(s)
hr	hour(s)
IBRD	International Bank for Reconstruction and Development
ICB	international competitive bidding
IDA	International Development Association
IRR	internal rate of return
IWTA	Inland Water Transport
JICA	Japan International Cooperation Agency

kg	kilogramme(s)
km	kilometre(s)
Km^2	square kilometre(s)
km/h	kilometre per hour
Kn	kilonewton
Lakh	100,000
L/C	local currency
LCB	local competitive bidding
LWL	Low water level
m	metre(s)
MAT	Manual and automatic tidal gauge
MCA	multi-criteria analysis
m/s	metre(s) per second
m^2	square metre(s)
m^3	cubic metre(s)
m^3/s	cubic metre(s) per second (cumecs)
MG	Metre Gauge
mm	millimetre(s)
MMSS	Mica schist silty sand
MN	meganewton
MPO	Master Plan Organization
MSL	mean sea level
N	Newton
NEDECO	Netherlands Engineering Consultants
NMC	natural moisture content
N-value	standard penetration test value
ODA	Overseas Development Agency
OECF	Overseas Economic Cooperation Fund
OMC	optimum moisture content
p.a	per annum
PDB	Power Development Board
PDF	Probability density function
PWD	Public Works Department (datum)
RC	reinforced concrete
RHD	Roads and Highways Department
RPT	Rendel, Palmer & Tritton Limited
RRI	River Research Institute
RTW	river training works
s,sec	second
SHW(L)	standard high water (level
SLW(L)	standard low water (level)
SOB	Survey of Bangladesh
SPT	standard penetration test
SWMC	Surface Water Modelling Centre
sq.km	square kilometre(s)



INDEX MAP SHOWING PROJECT LOCATIONS FOR THE MEGHNA SHORT TERM STUDY

t(tons)	metric tons
Tk	taka
TOR	Terms of Reference
US$(or$)	US dollar(s)
USCS	Unified soil classification system
WB	World Bank

ANNEX D

SCALE MODEL STUDIES

SCALE MODEL STUDIES

VOLUME IV - ANNEX D

CONTENTS

25

TABLES

FIGURES

PHOTOGRAPHS

LIST OF SYMBOLS

A	cross section area	(m^2)
b	width of the weir	(m)
C	Chezy roughness coefficient	$(m^{\frac{1}{2}}s^{-1})$
D	diameter of bed material (sand)	(m)
D_{50}	diameter of which 50 % of the weighted material has a sieve diameter bigger than D_{50}	(m)
F	Froude number	$(-)$
g	acceleration by gravity	$(m.s^{-2})$
H	total head, water depth and energy head	(m)
h	water depth	(m)
h_0	initial water depth	(m)
k	coefficient in the formula of Ahmad	$(-)$
k_s	hydraulic roughness	(m)
l	length	(m)
p	height of the crest of the weir above the bottom	(m)
q	specific discharge	$(m^3/s\ m^2)$
Q	total discharge	(m^3/s)
R	hydraulic radius	(m)
Re	Reynolds number	$(-)$
u	flow velocity	(m/s)
u_{cr}	critical flow velocity	(m/s)
u_w	flow velocity just upstream of the weir	(m/s)
$\hat{u}$	maximum flow velocity	(m/s)
y_s	scour depth	(m)
Δ	relative density	$(-)$
Ψ_{cr}	critical shear stress parameter	$(-)$

D.1 INTRODUCTION

Within the framework of the Meghna River Bank Protection Short Term Study the bank erosion in seven priority locations along the Upper Meghna River and the Lower Meghna River is being studied. Along the Upper Meghna River the following locations were studied: Bhairab Bazar both, the Town on the right bank and the Railway Bridge, Munshiganj, Roads and Highways Bridge and along the Lower Meghna River: Eklashpur, Chandpur and Haimchar, see Figure 1.1. From these locations three have been selected to be investigated in a physical model: a flow field model of Bhairab Bazar, and two local scour models of the river near Chandpur and near Eklashpur. These scale models have been constructed and tested in the River Research Institute in Faridpur.

From each scale model study a report was made that is included as one chapter in this Annex. These reports can be read as independent reports, these are: for Bhairab Bazar Chapter 3, for Chandpur Chapter 4 and for Eklashpur Chapter 5. Summary and conclusions for each model are presented in Chapter 2. In an Appendix scaling procedures are presented.

D.2 SUMMARY AND CONCLUSIONS

D.2.1 General

Within the framework of the Meghna River Bank protection Short Term Study three locations, Bhairab Bazar, Chandpur and Eklashpur, were investigated in a scale model. The design discharge is the 1 in 100 year flood. For each model investigation, the summary and the main conclusions are presented separately in the following.

D.2.2 Bhairab Bazar model

D.2.2.1 Scope of work

In a distorted scale model the flow field from 3 km upstream of the Railway Bridge near Bhairab Bazar up to 2 km downstream of this Railway Bridge was studied in three alternative solutions to reduce the bank erosion at Bhairab Bazar town along the right bank of the Upper Meghna River. These alternatives are: a continuous bank protection downstream of the Railway Bridge along the right bank, a sill between the first three piers of the railway bridge and a groyne, which is located upstream of the railway bridge along the right bank. In addition, some tentative measurements of the scour depths were analyzed. it is also mentioned that the groyne and the sill improve the stability of the right bank piers of the railway bridge to some extent.

D.2.2.2 Continuous revetment

A continuous "advanced" (as opposed to "retired") revetment has been designed along the right bank downstream of the Railway Bridge to protect Bhairab Bazar town from bank erosion. In two tests the slope of this revetment was varied: 1:3 and 1:4. This revetment will reduce the cross-sectional area of the river to a certain extent. This reduction of the cross section results in a small increase of the flow velocities along the left bank and a small increase in scour depths along the revetment. The effect of a continuous revetment on the flow lines can be neglected, if compared with the flow lines in the calibration test.

The measured model scour depths are the result of a combination of constriction scour (dominant in cross sections 3 to 5), the effect of the bank slope (dominant in cross sections 6 to 11) and some bend scour. The bend scour is rather small because the radius of the bend is big while the piers and the river bed protection of the Railway Bridge disturb the development of the typical secondary flow in the bend. The local scour just downstream of the Railway Bridge is caused by the Railway Bridge, which has a similar effect on the local scour in cross sections 2 to 4 as a sill.

The recommended scour depths for designs due to local scour, i.e. an increase in bend scour and constriction scour near a revetment:

cross section 2 to 5:		10 m,
cross section 6 to 11:	slope 1:3	2.5 to 5 m,
	slope 1:4	0 to 2.5 m.

These values are tentative because in the distorted model the scour processes are not reproduced with sufficient accuracy. From these scour depths the values of the k-coefficient in the formula of Ahmad were calculated. These values have been used for the design of the bank protection and in the geomorphologic analysis.

D.2.2.3 **Sill**

Under the two bridge spans along the right bank two types of sills were tested. Type two with the crest at a height of P.W.D -12.75 m and type one a sill with a sloping crest from P.W.D. -7 to -11 m.
A sill reduces the flow velocities in the bridge spans 9 - 10 and 10 - 11 by 0.3 and 0.7 m/s to 2.6 and 1.8 m/s respectively, in the other bridge spans the flow velocity increases by 0.35 to 0.60 m/s to 2.5 - 2.6 m/s.

Along the right bank of Bhairab Bazar town it is not clear from the measurements if the flow velocity will reduce. Probably the flow velocities measured during the calibration test are not sufficiently accurate. A low sill results in maximum flow velocities of 1.7 to 1.9 m/s and a high sill in velocities of 1.5 to 1.7 m/s along this bank.

A low sill will increase the flow velocities along the left bank by 0.5 to 0.7 m/s to a maximum velocity of 2 m/s, and therefore some erosion along the left bank may be expected.

A sill in the railway bridge bends the flow lines towards the left bank and downstream of cross section 5 this results in a constant shift of the flow lines up to cross section 11.

Along the right bank the scour depths are reduced by a high sill, but they will increase slightly, if a low sill is applied. This reduction varies from cross section to cross section. The coefficient k in the scour formula of Ahmad has been determined from the measured scour depths to be between 2.0 to 2.9.

D.2.2.4 **Groyne**

At the place of the existing ferry ghat at the right bank a groyne was designed with two lengths: 170 and 210 m. The axis of the groyne with a rounded head makes an angle smaller than 90 degrees with the main flow direction.

Near the head of the two groynes the maximum flow velocity is 2.2 to 2.5 m/s. At that location a scour depth of around 14 m is assessed, and this results in a level of the deepest point of the scour hole at 33 m - P.W.D.

A short groyne gives no reduction in flow velocities along the right bank downstream of the Railway Bridge. A long groyne can even result in 0.3 m/s higher flow velocities along that bank (compared with the calibration test).

The scour depths along the right bank downstream of the railway bridge are not reduced by a long groyne, the scour depths are almost the same as in the calibration test. But a short groyne will probably result in a reduction of about 5 m, if compared with the scour depths in the calibration test be spite there is no reduction in flow velocities.

All mentioned conclusions and results of the model tests are to be considered as tentative, because of the inaccuracy of the measurements and because the model is not a local scour model but a flow field model.

D.2.3 <u>**Chandpur model**</u>

D.2.3.1 **Scope of work**

Within the frame work of the Meghna River Bank Protection Short-Term Study a scale model was constructed of a part of the Lower Meghna River near Chandpur, a town at the left bank of this river. This scale model was used to study the existing bank erosion problems along the Lower Meghna River and to test different short-term solutions.

A part of the Lower Meghna River near Chandpur with a length of about 5 km and a width of about 1.5 km was modelled at a length scale of 1 in 150.

The non-distorted scale model has one main upstream inflow boundary, a small inflow boundary representing the flow over the char along the left bank upstream of Chandpur, a small inflow from the Dakatia river and one outflow boundary some kilometres downstream of Chandpur.

In the steady flow model of Chandpur a constant discharge has been pumped around. The influence of the tidal fluctuations on the scour process was neglected, as is usual in this type of models, because of the conflicting time scales of the tidal flow and the local scour process. Since the initiation of motion of the bed material in the model is observed locally, no facilities for the circulation of the sediment are required. The discharge distribution at the upstream boundary of the model is based on the results of the mathematical model study of this project.

D.2.3.2 Tests carried out

For the alternative solution with a groyne upstream of Chandpur three tests were executed, and the most accurate one, T9, was analyzed.

The alternative solution with an advanced protection in the shape of a guide bund around Nutan Bazar and Puran Bazar was tested in T8. This alternative solution was split up in a guide bund only around Nutan Bazar, test T10, in a guide bund only around Puran Bazar, test T11 and as a construction stage a partial advanced protection as a hard point around Nutan Bazar, test T13. In test T11 Nutan Bazar was supposed to be eroded almost completely, therefore the left bank of the Dakatia River at Puran Bazar needs to be protected also against the flow from the Lower Meghna River.

TABLE D.2.1 DESCRIPTION OF THE TESTS CARRIED OUT FOR CHANDPUR

Test number	
1 *	Calibration test with discharge according the Froude law
2	Calibration test with the discharge higher than in test T1
3 *	Calibration test with the discharge higher than in test T2
4 *	Emergency designs, variant 2
5 *	Upstream groyne, direction
6 *	Upstream groyne, direction
7	Repetition of T2 with correct inflow boundary
8	Guide bund around Nutan Bazar and Puran Bazar
9	Upstream groyne, direction -7 degrees
10	Guide bund for only Nutan Bazar
11	Guide bund for only Puran Bazar
12	Two short groynes at Nutan Bazar
13	Partial advanced protection at Nutan Bazar

* test with a not correct inflow boundary

The tests without an asterisk in the table above are analyzed in this report. In all these tests the discharge, the water levels, the flow velocities and the scour depths were measured.

D.2.3.3 Calibration test

In the calibration test the maximum flow velocities, which were measured in the model along the bank of Nutan Bazar, are 0.1 to 0.2 m/s higher than the flow velocities, which were measured in the flood survey of August and September 1991. Some flow lines in the model are compared with the flow lines in the mathematical model. At the inflow and outflow boundaries of the model these flow lines coincide, but in the middle of the model the flow lines in the model are stronger curved than the flow lines in the mathematical model, because of the relatively inaccurate modelling of the scour holes in the mathematical model and because of a scale effect in bed roughness. However, it can be concluded that the flow field in the model was reproduced with sufficient accuracy.

D.2.3.4 Upstream groyne with a length of 1600 m

The maximum flow velocities around the groyne are 3.9 m/s, near the head, 2.5 to 3.0 m/s at the most exposed part of the body and 2 to 2.5 m/s along the sand dam of the groyne near the bank.

Along the bank of Nutan Bazar the maximum flow velocity is reduced by 20% from 3.2 m/s to 2.6 m/s. Along the bank of Puran Bazar the maximum flow velocity is reduced by about 10% from 2.3 to 2.1 m/s at 75 m distance from the bank and from 3.05 to 2.9 m/s at 150 m distance from the bank.

D.2.3.5 Guide bund with advanced protection

The maximum flow velocity along the completed guide bund with advanced protection is 3.8 m/s, in case of a guide bund only at Nutan Bazar 3.5 m/s. These flow velocities are used as a design parameter for the top layers of the bank protection (see Annex G).

With a guide bund only at Nutan Bazar the maximum flow velocity near the existing bank of Puran Bazar is 1.5 m/s (in the existing situation it is 2.3 m/s).

If Nutan Bazar has been eroded almost completely, a guide bund at Puran Bazar will be attacked by a maximum flow velocity of 3.2 m/s.

D.2.3.6 Two groynes at Nutan Bazar

The maximum flow velocity near the head of groyne I is 1.9 m/s just downstream of the flow separation point. Along the body or the shaft of that groyne a maximum flow velocity of 1.5 m/s is to be expected.

The maximum flow velocity near the head of groyne II is 3.75 m/s in the axis of the groyne, and 2.3 to 3.6 m/s just downstream of the flow separation point.

Between these two groynes the flow velocities vary between 0.3 m/s to about 1.4 m/s, and the maximum flow velocity along the existing bank upstream of groyne I will be about 0.8 m/s.
Along the existing bank of Puran Bazar the maximum flow velocities at 40 m from the bank vary from 0.25 to 0.6 m/s in upstream direction (in calibration test 0.8 to 1.6 m/s), because the flow along that bank from cross section 23 to 26 is part of a big eddy.

All above mentioned flow velocities are time averaged flow velocities, which do not include the variations in the flow velocities by the eddies and the turbulence.

2d

D.2.3.7 **Scour depths**

The scour depths are important parameters for the design of the toe level of the bank protection and the size of the falling apron.

In the calibration test the maximum depth of the scour hole increased from 51 m-P.W.D. up to 55 m-P.W.D. The flow velocities in the model are just high enough to reproduce the scour at the depth scale in the model.

D.2.3.8 **Upstream groyne with a length of 1600 m**

The maximum measured depth of the scour hole in front of Nutan Bazar is 53 m-P.W.D.

Near the head of the upstream groyne a maximum depth of 39 m-P.W.D. is measured, while in the scour hole 300 m downstream of this groyne the maximum measured depth is 53 m-P.W.D.

D.2.3.9 **Guide bund with advanced protection**

In the test with the completed guide bund the maximum measured depth of the scour hole in front of Nutan Bazar is 58 m-P.W.D.

A guide bund only at Nutan Bazar results in almost the same depth: a maximum measured depth of 57 m-P.W.D. However, in a construction phase of this guide bund at Nutan Bazar a hard point as a partial guide bund induces a deep scour hole with the deepest point at 65 m-P.W.D., due to the poor guidance of the flow. These scour depths are used in the Geomorphological study (Annex B) of this project.
If Nutan Bazar is eroded almost completely than the maximum measured scour depth in front of the advanced protection around Puran Bazar is 43 m-P.W.D.

D.2.3.10 **Two groynes at Nutan Bazar**

Downstream of groyne II at Nutan Bazar a maximum depth of the scour hole of 65 m-P.W.D. has been measured. Along the existing bank of Puran Bazar this maximum depth is 48 m-P.W.D.

From the comparison of the different alternative solutions the following conclusions can be drawn:

- The upstream groyne has induced a small reduction of the scour depths near Nutan Bazar.

- The two groynes at Nutan Bazar and also a partial guide bund (as a hard point) result in a deep scour hole downstream of these groynes.

- The completed guide bund creates a small increase in scour depth.

- The advanced protection or guide bund at Nutan Bazar is comparable with the completed guide bund and gives some protection to Puran Bazar.
  Therefore this alternative seems to be optimal from a hydraulic and morphologic point of view. However, the first phase (during construction) of this advanced protection can induce a deep scour hole.

The formula of Ahmad was applied to analyze the results in the calibration test. The value of an empirical coefficient in that formula was determined from the measured data. These values are within the range recommended in literature for this coefficient. It is emphasized that in the model no falling apron was modelled.

D.2.4 Eklashpur model

D.2.4.1 Scope of work

Within the frame work of the Meghna River Bank Protection Short-Term Study a scale model was constructed of a part of the Lower Meghna River near Eklashpur, and some aspects of the proposed bank protection works were studied in that model. In this scale model two types of solutions were tested: a groyne upstream of Eklashpur and a continuous revetment from Mohanpur to downstream of Eklashpur.

A part of the Lower Meghna River from the confluence of the Padma River with the Upper Meghna River to a few kilometres downstream of Eklashpur and a width of about 1.5 km was modelled at a length scale of 1 in 150. The non-distorted model has either three or four inflow boundaries and one outflow boundary with different tailgates. The discharge distribution along the inflow boundaries was obtained from the mathematical model study.

In the steady flow model a constant discharge was pumped around during a test. The influence of the tidal fluctuations on the scour process was neglected in this model, as is usual in this type of models, because of the conflicting time scales of the tidal flow and the scour process. Since the initiation of motion of the bed material in the model is observed locally, no facilities for the circulation of the sediment were required.

D.2.4.2 Tests carried out

In the first tests T1 to T6 the model bed represented the bed geometry, which was measured during the field survey in February 1991. The flow velocities were calibrated with flow velocities measured during the field survey in August-September 1991.

In the actual situation the BWDB protection and a groyne upstream of Eklashpur were tested.

For design conditions a so-called worst case situation has been defined in the geomorphological study (Annex B). For this worst case situation two extreme discharge distributions were determined in the hydrological study (Annex A). With the mathematical model (see Annex E) the detailed discharge distributions over the inflow weirs were calculated. The continuous revetment and the upstream groyne were tested in these worst case conditions, see Table below.

TABLE D.2.2 DESCRIPTION OF TESTS CARRIED OUT FOR EKLASHPUR

number	
T1 ... T2	calibration tests
T3	BWDB bank protection
T6	groyne upstream of Eklashpur
T6 ... T9	continuous revetment
T10 ... T13	groyne upstream of Eklashpur

The BWDB protection has a length of about 600 m and is located just upstream of Eklashpur. The continuous revetment starts just upstream of the bend and continues up to downstream of Eklashpur. The upstream groyne has a length of 600 to 700 m.

D.2.4.3 **Flow velocities**

For the flow velocities along the bank a representative flow velocity was defined and this flow velocity was calibrated with the flow velocities measured during the flood survey in August September 1991.

As a design velocity along the upstream side of the groyne a maximum flow velocity of 2.1 m/s was measured, and around the head of the groyne a maximum flow velocity of 2.8 m/s was measured.

The groyne reduces the representative flow velocities along the existing bank with 0.5 m/s up to 2.5 km downstream of the groyne, which is about 4 times the length of the groyne.

In the bend near the confluence (i.e at the revetment) the representative flow velocity is 1.9 m/s and this is the maximum flow velocity. Downstream of the bend this flow velocity reduces to 1.0 m/s and downstream of cross section 45 this flow velocity increases up to 1.7 m/s.

D.2.4.4 **Scour depths**

Near the head of the groyne the local scour hole coincides with the confluence scour hole and the deepest point has a level of 36 m-P.W.D. Along the bank downstream of the groyne the scour depths are reduced by about 2 m up to cross section 40 if compared with the continuous revetment.

Along the continuous revetment the deepest point of the scour hole has a level of 27 m-P.W.D. near Eklashpur. The scour depths are estimated at 5 m in that location. In the bend of the bank near the confluence the equilibrium bed levels after scouring are higher than 27 m-P.W.D., but scour depths of about 10 m are the maximum to be expected at that location.

CHAPTER D.3 BHAIRAB BAZAR MODEL

Contents

52

D.3 BHAIRAB BAZAR MODEL

D.3.1 Introduction

Within the frame-work of the Meghna River Bank Protection Short-Term Study some additional tests for Bhairab Bazar town are required in an already existing scale model at RRI. This model has been used for the study of the conditions upstream of the railway bridge in Bhairab Bazar. The model had to be extended in downstream direction to study also the bank erosion problems at Bhairab Bazar town. It was decided to replace the model by a fresh model site and to build a complete new model at that new site.

The purpose of the scale model investigation for Bhairab Bazar town is:

(i) to check whether certain measures proposed for the problems upstream of the bridge do also have a sufficiently positive effect on the erosion problems in Bhairab Bazar town area, and

(ii) if not sufficient, to study in the model short-term measures to stop the bank erosion at this location.

The following three types of short-term measures have been tested in the model: a groyne along the right bank upstream of the railway bridge, a sill between the bridge sections along the right bank and a continuous revetment along Bhairab Bazar town.

In this Chapter also a review of the previous scale model study has been included (Section D.3.2). The available prototype data, the model set-up, the measurements and the test program are described, (Sections D.3.3 and D.3.4). In the interpretation of the measurements the flow velocities (Section D.3.6.3), the flow lines (Section D.3.6.4) and the scour depths (Section D.3.6.5) are distinguished. In addition the water level gradient and the hydraulic roughness in the calibration tests are analyzed. The determination of the model scales, the application of scale conditions and scale laws are treated in Appendix D/1 Scaling procedures.

D.3.2 Review previous scale model

The review of the previous flow field model of the Upper Meghna River near Bhairab Bazar has been required in the Terms of Reference. This review is based on the Report no 106 of RRI, see ref[3], completed with some information from discussions with the staff of RRI.

The previous scale model investigation has been focused on a solution for the stability of the railway bridge near Bhairab Bazar, after the bank slide just upstream of the bridge in 1988. The distorted flow field model had a vertical scale of 60 and a horizontal scale of 300. In the scale model the cross sections of a survey of BWDB, Morphology have been moulded to approximately 3 km upstream of the railway bridge, see general lay out in Figure D.3.1. The inflow section, more upstream, is important for the development of the bend flow. This part has been intuitively modelled and is probably not correct. Consultants expect this effect will be significant for the flow along the right bank upstream of the railway bridge. Therefore the flow field is not reproduced with sufficient accuracy in the model.

The model discharge has been determined correctly with the Froude law, however in the calibration the reproduction of the roughness of the bed and the banks in the model are not checked. It was stated that the water level gradient in the model is equal to the water level gradient in the prototype. This is not a sufficient condition, however, the roughness condition results in a scale factor for the water level gradient of 0.45. Most probably the flow field in the model was affected by a too low model roughness. In the shallow parts of the model the flow is probably not fully turbulent as in the prototype, therefore a scale effect is introduced.

In the Report the analysis of the measurements is far from complete, only the main conclusions are mentioned. The proposed solution is to construct in the first phase four relatively small T-head groynes

60

with a length of 120 m to 310 m along the right bank, upstream of the railway bridge. In the final solution also two or more T-head groynes are designed more upstream along the left bank. Consultants are of the opinion that this solution is too expensive and each T-head needs an extensive protection against local scour. In the Report it is mentioned that heavy scour occurred at the sides and the toes of the groynes. However in the calibration of the model the scale relations for the scour process were not established. Moreover, Consultants are of the opinion that the scour process can not be studied in this model.

D.3.3 Prototype data and model set-up

The bed of the Upper Meghna River around the railway bridge has been modelled with the data of the previous model of Bhairab Bazar. Downstream of the railway bridge the bed is modelled by using 11 cross sections according to the bathymetric survey by BIWTA in April 1989. Upstream of the railway bridge the results of the field survey by BWDB, Morphology of December 1988 were used in tests T1 to T15. From test T16 onwards the bed has been modelled with the results of the field survey by BIWTA of 9 to 12 December 1988, see Figures 3.2 to 3.4. These cross sections have been compared with the results of the recent field survey by BWDB, Morphology. These results show some unlogic and inaccurate cross sections, but the majority of the cross sections fits well with the BIWTA data. Both field surveys give data about the cross sections up to about 2 km upstream of the railway bridge. For the alignment of the more upstream bend use has been made of a satellite image of the Upper Meghna River taken on 22th February 1989, see Figure D.3.5.

The characteristic size of the sand in the river bed has been determined from a sieve analysis of bed samples: D_{50} = 0.16 mm, D_{90} = 0.27 mm.

Model set-up

The distorted model has one inflow section and one outflow section and a steady discharge is pumped around. The tidal influences are small in Bhairab Bazar and they are neglected in this model. During a test the water level has been kept constant.

In prototype the flood plain at both sides of the banks is flooded during a 1 in 100 year flood. In the model the banks have been raised to a level above the 1 in 100 year flood to facilitate the operation of the model. The effect of this schematization on the main results of this model study can be neglected.

The model has been moulded in sand, and at places where the banks become too steep because of the distortion of the model, the banks are strengthened by cementing. In an open air model these banks are sensitive to damage by rain, which has an adverse effect on the accuracy of the model. No special adjustments has been designed to adjust the model roughness of the sandy bed in the model. The bed material in the model is sand, which is the same type of sand as in the prototype.

In general the available head difference between the inflow and the outflow of the model is too small and in many tests it was difficult to drain the deepest places in the model. Two drainage pipes improved the drainage of the model. These places are often the deepest points of the scour holes.

D.3.4 Test program

The test program consists of two calibration tests and some tests for the main alternative solutions for the right bank erosion: a revetment along Bhairab Town, a groyne upstream of the railway bridge and a sill in the bridge sections. In the test with two types of groynes at the location of the existing ferry ghat at the right bank only the length of the groyne has been changed from 210 to 170 m, see Figures 3.6 and 3.7. Also two types of sills have been tested: a relatively high sill, see Figure D.3.8, and a relatively low sill, see Figure D.3.9. The locations of the sills have not been changed.

According to the test program the following tests have been executed:

9

Test-number	description of the test
T16	A short, 170 m long groyne upstream of the railway bridge. The bed topography is accurate, only the upstream approach channel is very approximately modelled.
T17	A short, 170 m long groyne upstream of the railway bridge combined with a sill between the bridge spans 9 - 10 and 10 - 11. The bed topography and the approach channel are modelled accurately.
T18	A relatively high sill in the railway bridge, between the piers 9 - 10 and 10 - 11.
T19	Revetment along the bank of the town, with side slope 1:3
T20	Revetment along the bank of the town, with side slope 1:4
T21	A relatively low sill in the railway bridge, between the piers 9 - 10 and 10 - 11.
T22	A long, 210 m long groyne upstream of the railway bridge.
T23	Calibration test with Q = 175 l/s and Q = 220 l/s

Tests T1 to T15 were done with a wrong bed geometry upstream of the railway bridge and a very approximately and therefore not accurate modelled approach channel. Planned tests (T7 and T8) with geomorphological alternatives have not been defined because the Geomorphological study (Annex B) had shown that the banks of the Upper Meghna River are stable near Bhairab Bazar.

D.3.5 Measurements

In each test the discharge, the water level, the flow velocities and the flow lines were determined. In most tests also the scour depths were measured.

Discharge:

The model discharge has been calculated from the water level upstream of a Rehbock-weir (rectangular weir with a sharp crest with a brass strip, however in the model corrugated weirs were applied with a irregular crest). The upstream water level has been substituted in the standard formula for these type of weirs to determine the model discharge (if the upstream flow velocity can be neglected):

$$Q = 1.86 \ . \ b \ . \ h_w^{1.5}$$

in which Q = discharge (m3/s), b = width of the weir (m) and h_w = vertical distance between the crest of the weir and the upstream water level (m).

62

However, in the measuring weir in the model the upstream flow velocity can not be neglected and a more detailed formula has to be applied:

$$Q = (0.407 + 0.053 \cdot \frac{h_w}{p}).\sqrt{2.g}.b.[(h_w + \frac{u_w}{2.g})^{1.5} - (\frac{u_w}{2.g})^{1.5}]$$

in which u_w = the average flow velocity in the cross section in which h_w has been measured (m/s), p = the height of the crest of the weir above the bottom (m) and g = acceleration by gravity (m/s^2).

No calibration formula has been determined for each weir separately as was required in the Terms of Reference for the model study in RRI. The supply of air under the falling jet was not guaranteed as required for a standard weir. The upstream water level showed considerable fluctuations, so the model discharge is more or less an estimated time averaged discharge. The accuracy was not high; the accuracy of the discharge calculated with the first formula is estimated at 5 to 7 % in case of a weir with a corroded crest.

Water level:

In the model the water level can be regulated by the 2 tailgates. After the calibration test the position of the gates should be kept constant. Although their position could not be read accurately, the position has been kept constant approximately during all tests. These positions of the two tailgates are only important for the distribution of the discharge in a cross section just upstream of the weirs. The average water level is checked with the point gauges and is well controlled. From the point gauges accurate readings of the water level could be made.

Flow velocity:

The time averaged flow velocities have been measured by an OTT flow propeller having a diameter of about 0.03 m. The measurement time for counting the revolutions of the propeller has been reduced from 1 minute to 0.5 minute, and this has resulted in an accuracy of + or - 0.025 m/s in the prototype values.

Flow direction:

The flow direction has been measured with flow lines, which have been determined by floats. These floats have been made from small bottles, which float with a depth of approximately 0.04 m into the water. The position of the float was determined at the moment the float passes the line of a cross section. The accuracy of this method to locate the float was about 0.05 m.

In some tests the accuracy of the flow lines has been affected by side wind, see for example test T16.

Scour depths:

In the model the scour depths were measured by a rod with a conical top. The depth was read from the position of the rod relative to the reference line. This line was also used to fix the location relative to the bank.

An impression of the accuracy of this very simple method to measure the scour depths has been obtained by comparing the measured deepest point in the initial bed level with the deepest point in the initial cross sections of the bathymetrical map. In about 8 cross sections the differences between the measured and the given deepest point were about 0 to 0.3 m, in 2 cross sections these differences were about 0.5 m and in 1 cross section this difference was 1.5 m. With a more accurate moulding of the model bed these differences can be reduced to less than 0.3 m.

The grain size distribution of the sand in the model has been determined from only one sample. It is recommended to analyze three samples and to include an hydrometer analysis, because 5 to 10% has a diameter less than 0.074 m. As a first estimate D_{50} = 0.18 mm, D_{90} = 0.35 mm and D_{10} = 0.10 mm were used, see Figure D.3.10.

As an additional measurement the temperature of the water has been measured in each test. In tests number T16 to T23 the lowest temperature was 28 degrees Celsius, the highest temperature 32 degrees Celsius and the average temperature around 30 degrees Celsius. In the analysis the effects of this variation in the water temperature on the test results has been neglected.

D.3.6 Interpretation of the results

D.3.6.1 Introduction

The interpretation of the test results is concentrated on tests T16 ... T23, because in the earlier tests the bed geometry and the approach section upstream of the railway bridge were not accurately modelled. The report of RRI on this physical model contains all the data, which have been used for this interpretation.

The following aspects are treated in the next sections:

- model discharge and hydraulic gradients (Section D.3.6.2),
- flow velocities (Section D.3.6.3),
- flow lines (Section D.3.6.4) and
- scour depths (Section D.3.6.5).

In the Summary and Conclusions (Chapter D.2) the results of the different sections are combined for each type of short term measures to prevent and to stop the bank erosion near Bhairab Bazar town.

D.3.6.2 Model discharge and hydraulic gradient

The model discharge in calibration test T1 has been 0.16 m^3/s as per Froude condition and in calibration test T2 the model discharge has been increased up to 0.22 m^3/s. All the alternatives have been tested with a discharge of 0.22 m^3/s, because the scour process will be reproduced more accurate in a test with this higher discharge. As already mentioned, the part upstream of the railway bridge was not correctly modelled in tests T1 and T2, therefore the calibration test was repeated with the correct modelled river in test T23.

A comparison of the flow velocities in 4 cross sections shows that in test T1 higher flow velocities are measured than in test T2. Probably the number of the tests T1 and T2 are changed on the forms.
In the calibration tests the hydraulic gradient has been determined from the readings of 4 point gauges along the model:

TABLE D.3.1 THE MEASURED HYDRAULIC GRADIENT

Test number	hydraulic gradient sections	average	geometry
T1	0.000315 0.000312 0.000337 0.000333	0.00032	deviating approach channel
T2	0.0003 0.00034 0.00036	0.00033	deviating approach channel
T23	0.00033 0.00036 0.00034	0.00034	good approach channel

From Table D.3.1 it can be concluded that the differences in the average hydraulic gradient are very small. The Chezy coefficient, C, in the model has been calculated with the average flow velocity and the hydraulic radius in cross section 3 or 4 downstream of the railway bridge. This calculation resulted in C = around 41 $m^{0.5}/s$. This means that the roughness condition is fulfilled and that the flow field is reproduced accurately in the model:

$$n_C = \{n_l / n_h\}^{0.5}$$

$$n_C = 90 / 41 = 2.19$$

$$\{n_l / n_h\}^{0.5} = 2.24$$

in which $n_l = 300$ and $n_h = 60$.

It is recommended to execute calibration test T23 once more in two separate tests: one with a constant discharge of 157 l/s and the other with a discharge of 220 l/s. The presented documentation of the measurements in this test T23 is not sufficient or, alternatively, inaccurate.

D.3.6.3 Flow velocities

D.3.6.3.1 Introduction

The flow velocities are measured in the following locations:

- In tests T16 to T23 in cross sections 3, 6 and 11 downstream of the railway bridge.

- In the tests with a sill in the bridge axis the flow velocities in the middle of the bridge spans have been measured. The sill tests are treated in Section D.3.6.3.2.

- In nearly all tests the flow velocities along the right bank downstream of the railway bridge have been measured. The tests with a continuous revetment along the right bank are analyzed in Section D.3.6.3.3.

- In the tests with a groyne along the right bank upstream of the railway bridge the flow velocities around the head of the groyne are measured, Section D.3.6.3.4.

Ob

D.3.6.3.2 Sill in the Railway Bridge

The flow velocities in the bridge axis have been measured in the middle of the bridge spans. In test T18 with a relatively high sill, see Figure D.3.8, and in T21 with a relatively low sill, see Figure D.3.9, placed in the bridge spans 9 - 10 and 10 - 11. In Table D.3.2 these velocities are compared with the velocities, which have been measured in the calibration test.

TABLE D.3.2 FLOW VELOCITIES IN THE MIDDLE OF THE BRIDGE SPANS IN TESTS WITH A SILL AND THE CALIBRATION TEST

bridge span	test T18	test T21	(3) - (2)	calibration test T23-2
(1)	(2)	(3)	(5)	(6)
-	m/s	m/s	m/s	m/s
4 - 5	2.59	2.53	-0.06	2.02
5 - 6	2.38	2.35	-0.03	1.94
6 - 7	2.53	2.50	-0.03	2.16
7 - 8	2.59	2.41	-0.18	2.51
8 - 9	1.60	2.56	0.96	2.71
9 - 10	1.79	2.23	0.44	2.53
10 - 11	1.54	2.23	0.67	2.26

Effect of a sill in bridge spans 9 - 10 and 10 - 11 on the flow velocities in the middle of the bridge spans:

- In bridge spans 9 - 10 and 10 - 11:
  low sill (T21): a reduction of 0 to 0.3 m/s compared with the flow velocity in the calibration test, a maximum flow velocity of 2.25 m/s,
  high sill (T18): a reduction of about 0.7 m/s compared with the flow velocity in the calibration test, a maximum flow velocity of 1.8 m/s.

- In the other bridge spans 4 to 9:
  low sill (T21): an increase of 0.35 to 0.5 m/s compared with the flow velocity in the calibration test, a maximum flow velocity of 2.55 m/s
  high sill (T18): an increase of 0.40 to 0.60 m/s compared with the flow velocity in the calibration test, a maximum flow velocity of 2.6 m/s.

- The low sill (T21) results in a more uniform flow velocity distribution in the bridge spans than in the velocity distribution in the calibration test. And the absolute maximum flow velocity is reduced from 2.7 m/s to 2.6 m/s. For the inland navigation, which uses bridge spans 4 to 7, the flow velocity will increase with 0.4 to 0.5 m/s. This seems to be acceptable and therefore a low sill (T21) can be considered as an improvement of the velocity distribution in the railway bridge.

The effect of a sill on the flow velocities along the right bank follows from Table D.3.3.

TABLE D.3.3 FLOW VELOCITIES ALONG THE RIGHT BANK DOWNSTREAM OF THE RAILWAY BRIDGE

cross section	flow velocities T18	flow velocities T21	flow velocities calibration test T23-2
	m/s	m/s	m/s
2	1.67	1.79	1.62
3	1.49	1.79	1.67
4	1.70	1.91	1.62
5	1.49	1.66	1.53
6	1.47	1.85	1.65
7	1.49	1.78	1.87
8	1.67	1.82	1.65
9	1.61	1.85	1.82
10	1.66	1.81	1.86
11	1.61	1.84	1.67

The interpretation of the flow velocities along the right bank downstream of the railway bridge in Table D.3.3 is as follows:

- Comparison of the flow velocities in cross sections 2 to 11 in the test with the high sill and in the test with the low sill learns:
  low sill 1.7 to 1.9 m/s
  high sill 1.5 to 1.7 m/s
  This result seems to be logic: a lower sill results in higher flow velocities than a high sill. It is however remarkable that this difference in flow velocities can be measured up to cross section 11, (see also the analysis of the flow lines).

- When comparing the flow velocities along the right bank in the tests with the sills and with the calibration test, it seems that the flow velocities along the right bank downstream of the bridge are increased by the low sill. This could not to be expected from Table D.3.2: also a low sill should reduce the flow velocities along the bank at least in the first cross sections near the bridge. It is concluded that the accuracy of these measurements is not sufficient and it is not possible to obtain a clear result from these measurements.

The measured flow velocities along the left bank have been presented in Table D.3.4 for the tests with a sill in the railway bridge.

TABLE D.3.4 FLOW VELOCITIES NEAR THE LEFT BANK DOWNSTREAM OF THE RAILWAY BRIDGE

cross section, distance from right bank	flow velocities T18	flow velocities T21	flow velocities T23
-, m m	m/s	m/s	m/s
3, 800 - 850	1.52 1.26	2.08 1.30	1.81 1.21
6, 840 - 890	- -	2.03 1.50	1.32 0.94
11, 900 - 950	- -	1.53 0.94	1.50 1.57

From Table D.3.4 the following tendencies in the flow velocities along the left bank are determined:

- A low sill will increase the flow velocities along the left bank up to cross section 11.
- The highest increase in flow velocity of 0.5 to 0.7 m/s has been measured in cross section 6. This will induce some erosion along the left bank.

These tendencies seem to be logic.

D.3.6.3.3 A continuous revetment

In these tests a continuous advanced revetment has been constructed along the right bank downstream of the railway bridge from cross section 1 to 11. In cross sections 3, 6 and 11 the flow velocity distribution in the cross section has been measured, see Table D.3.5 and in Figure D.3.11. These cross sections are representative for the velocity distributions near the revetment.

6+

TABLE D.3.5 FLOW VELOCITY DISTRIBUTIONS IN CROSS SECTIONS 3 AND 6

Distance from the right bank	cross-sect. 3	cross-sect. 3	cross sect. 3		cross sect. 6	cross sect. 6	cross sect. 6
	T23	T19	T20		T23	T19	T20
m	m/s	m/s	m/s		m/s	m/s	m/s
0	0.73	0.74	0.68		1.00	1.21	1.00
50	0.83	0.92	0.83		1.01	1.31	1.11
100	0.79	1.03	0.94		1.23	1.55	1.26
150	1.01	1.26	1.00		1.53	1.53	1.53
200	1.21	1.65	1.18		1.62	1.95	1.69
250	1.31	1.64	1.70		1.69	1.79	1.99
300	1.41	1.59	1.66		1.71	2.12	1.96
350	1.59	1.72	1.67		1.51	2.03	2.07
400	1.58	1.85	1.65		1.53	1.92	1.87
450	1.48	1.42	1.89		1.65	1.61	1.81
500	1.44	1.53	1.71		1.61	1.79	1.91
550	1.86	2.03	1.85		1.63	1.89	1.99
600	1.74	1.53	1.88		1.69	2.12	1.94
650	1.73	2.12	1.88		1.75	1.94	2.00
700	1.81	1.43	2.03		1.32	1.82	1.88
750	1.21	1.28	1.45		0.94	1.49	1.61

From Table D.3.5 and Figure D.3.11 it follows that a continuous and advanced revetment will induce higher flow velocities along the left bank, because this revetment is slightly built into the river and accordingly the cross sectional area of the river will be reduced.

From the graphs in Figure D.3.12 it can be concluded that:

- The maximum flow velocities along the bank are on an average the same for 1:3 and 1:4 slopes.
- In the middle of the river the flow velocities in the test with the slopes 1:3 are about 0.2 to 0.3 m/s higher than the flow velocities in the calibration test, because the area of the cross section is reduced by the new revetment.

D.3.6.3.4 Groynes

A groyne with a length of 170 m has been built near the ferry ghat in test T16, in which the upstream bend has been modelled approximately. To have an impression about the effect of the length of the groyne this length has been increased up to 210 m in test T22. The combination of a small groyne and a sill in the railway bridge has been tested in T17.

The maximum flow velocities along the right bank downstream of the railway bridge have been presented in Table D.3.6.

TABLE D.3.6 THE FLOW VELOCITIES ALONG THE RIGHT BANK DOWNSTREAM OF THE RAILWAY BRIDGE IN THE TESTS WITH A GROYNE

cross section number	T16 flow velocity	T17 flow velocity	correction T17 *)	T22 flow velocity	T23 calibration
-	m/s	m/s	m/s	m/s	m/s
2	1.27	1.80	2.26	1.45	1.62
3	1.30	1.18	1.48	1.49	1.67
4	1.66	1.41	1.77	1.64	1.62
5	1.54	1.33	1.67	1.72	1.53
6	1.60	1.33	1.67	2.01	1.65
7	1.66	1.41	1.77	1.71	1.87
8	1.54	1.33	1.67	1.77	1.65
9	1.48	1.26	1.58	1.84	1.82
10	-	-	-	1.71	1.86
11	-	-	-	1.71	1.67

*) correction T17 means correction for the relatively too low discharge in T17

Detailed measurements of the flow velocities near the right bank downstream of the railway bridge have resulted in some unreliable measurements, in which the flow velocity near the bottom is higher than the flow velocity at half the water depth. The other inconsistency is that the relation between the flow velocities along the bank, and near the water surface, at half the water depth, and at the toe of the bank varies too much in different cross sections and in tests T16 and T17. A possible explanation is that the determination of the position of the Ott flow meter related to the right bank was not accurate.

From Table D.3.6 it follows that a small groyne will reduce the flow velocities by 0 to 0.3 m/s in cross sections 2 to 9.

The additional reduction by the sill on the flow velocities near the right bank downstream of the railway bridge can be observed up to cross section 4; more downstream there is no influence by the sill on the flow velocities.

A 210 m long groyne and the combination of a short groyne and a sill will not result in an improvement, see Table D.3.6, however this seems to be an unlikely tendency.

It is checked if this tendency is confirmed by the flow velocity distribution in cross section 6 (Table D.3.7).

TABLE D.3.7 FLOW VELOCITIES (M/S) IN CROSS SECTION 6 DOWNSTREAM OF THE RAILWAY BRIDGE

distance (m)	T16	T17 corr.	T23	T22
140	0.97	0.94 1.17	1.00	0.73
190	1.00	0.94 1.17	1.01	1.40
240	1.24	1.02 1.27	1.23	1.54
290	1.63	1.10 1.38	1.53	1.87
340	1.81	1.18 1.48	1.62	1.98
390	2.05	1.57 1.96	1.69	2.00
430	2.05	1.64 2.05	1.71	2.00

From Table D.3.7 follows:

- The short groyne in T16 gives no reduction in flow velocities near the bank in cross section 6, (if compared with the calibration test).
- The long groyne and the combination of a sill and a small groyne results in slightly higher velocities near the bank in cross section 6 than in the calibration test.

These results confirm the conclusions from Table D.3.6.

Flow velocity around the head of the groyne

The maximum flow velocity, which is measured at half the water depth and at 45 to 60 m from the groyne, is presented for T16, T17 and T22:

T16 short groyne with deviating upstream bend:

$u\hat{} = 2.20$ m/s

T17 short groyne with right modelled upstream bend and a sill between bridge piers 9 - 10 and 10 - 11:

$u\hat{} = 2.06$ m/s

T22 long groyne with right modelled upstream bend:

$u\hat{} = 2.44$ m/s

If the length of the groyne is increased, then also the maximum flow velocity around the head of the groyne will increase. This is a logical tendency.

The deviating upstream model bed results in a too high flow velocity (0.15 m/s) around the head of the small groyne.

D.3.6.4 Flow lines

In tests T16 to T23 flow lines have been determined rather accurately, see Figures 3.13 to 3.28 of the Report of RRI. From these figures the following tendencies can be determined:
Continuous revetment:

Downstream of the railway bridge and near the beginning of the revetment (from cross section 1 to cross section 5) the flow lines are pushed towards the centre of the river. Downstream of cross section 5 the flow lines are more or less parallel to the bank, and shifted from the right bank, if compared with the flow lines in the calibration test T23.

Sill:

Under the bridge the flow lines bend more towards the left bank, than in the calibration test T23. A higher sill induces a stronger bend effect (test T18 compared with test T21) and lower flow velocities along the right bank, see Section D.3.6.3.2. Downstream of cross section 5 the flow lines are more or less parallel to the right bank and shifted towards the left bank, if compared with the calibration test.

Groyne:

A long groyne (test T22) induces a shift of the flow lines towards the centre of the river up to cross section 4 to cross section 11 downstream of the railway bridge. It is possible that this shift between these cross sections is influenced by side wind. Side wind had drifted the float towards the left bank in test T16. It is noted that no flow lines were measured in test T17.

D.3.6.5 Scour depths

D.3.6.5.1 Introduction

The distorted scale model of Bhairab Bazar is mainly a flow field model, in which also an impression of the local scour depths can be obtained. The applied method for scaling of the scour depths in this model has been treated in Appendix D/1 Scaling procedures.

The scour depths were measured by a very simple method using a rod which has a conical shaped end. The position of the rod was read relative to a reference line. The fixation of the position of the rod in the x,y-coordinate system needs special attention.

An impression of the accuracy of these measurements could be obtained by comparing the measured deepest point in the initial bed level before the start of a test with the deepest points in the initial cross sections of the bathymetric map. In about 8 cross sections the differences between these deepest points were about 0 to 0.3m, in 2 cross sections this differences were about 0.5 m and in 1 cross section this difference amounts to 1.5 m. Differences between 0.5 a 1.5 m are too high and more attention should be paid to modelling the river bed before the start of a test.

The scour depths in the calibration tests are analyzed in Section D.3.6.5.2.

Scour depths were measured around the head of the groyne (see Section D.3.6.5.3 about the tests with a groyne) and along the right bank downstream of the railway bridge (see Section D.3.6.5.4 about the test with a revetment). In addition some scour measurements along the left bank have been executed.

D.3.6.5.2 Calibration tests

The modelled river bed has been based on the bathymetric maps just after the 1 in 25 years flood in 1988. For design purposes a flood of 1 in 100 years has been reproduced in the model. Therefore some scouring of the bed can be expected.

The calibration test T23 was split into two parts: the first two hours the discharge was kept constant at 0.157 m3/s, then the discharge was increased to 0.220 m3/s, because it was supposed that no scour occurred during the first two hours. No measurements are available to prove this assumption.

The bed levels in the scour holes have been measured 2 to 5 hours after the start of the test. From these measurements the equilibrium bed level has been estimated by graphical extrapolation, see Figure D.3.29. It is mentioned that the sedimentation process is not reproduced at scale in the model, therefore only attention has been paid to the places with erosion.

TABLE D.3.8 THE MEASURED AND EXTRAPOLATED SCOUR DEPTHS IN CALIBRATION TEST T23

cross section	initial bed level	equilibrium scour depth	equilibrium bed level
-	m, prototype	m, prototype	m, prototype
2	-23.8 m PWD	-11.9 m PWD	-35.7 m PWD
3	-24.0	-4.6	-28.6
4	-25.1	-6.9	-32.0
5	-23.6	-10.5	-34.1
6	-19.4	0	-19.4
7	-16.3	-5.0	-21.3
8	-13.9	-7.3	-21.2
9	-12.2	-9.2	-21.4
10 *	n.m.	n.m.	n.m.
11	-10.1	-7.3	-17.4

* n.m. = not measured

The maximum scour along the revetment has been observed between cross sections 2 and 3 downstream of the railway bridge. The river bed protection near the railway bridge has a function similar to a sill for a scour hole downstream of it. The scour holes more downstream of cross section 3 and along the right bank are on average less deep than the scour hole between cross sections 2 and 3.

The equilibrium scour depths in the existing geometry and during a 1 in 100 year flood are estimated at (from Table D.3.8):

- downstream of the railway bridge (cross section 2): 10 to 12.5 m,
- Bhairab Bazar town (cross sections 3 to 11): 5 to 7.5 m.

It is mentioned that the higher scour depths, which have been measured in cross section 5 and 9, are considered to be an overestimation.

D.3.6.5.3 <u>Groynes</u>

In test T16 (a short groyne having a wrongly modelled upstream bend) a maximum scour depth of 0.04 m in 3 model hours has been measured near the head of the groyne. The equilibrium scour depth has been determined by extrapolation: a maximum measured scour depth of 14 m can be considered as a good estimation of the equilibrium scour depth. The deepest point of the scour hole has a level of -33 m - P.W.D.

Along the right bank:

Downstream of the railway bridge the local scour along the right bank in the tests with a groyne are summarised in Table D.3.9. It is mentioned that in test T17 no scour depths were measured.

TABLE D.3.9 SCOUR DEPTHS ALONG THE RIGHT BANK IN TESTS WITH A GROYNE

cross section	initial bed level T16	equilibrium scour depth T16	scour depth T16	scour depth T22
-	m + PWD	m + PWD	m	m
2	-25.0	-30.0	-5.0	-6.0
3	-24.6	-29.1	-4.5	-6.75
4	-24.0	-28.5	-4.5	-4.5
5	-23.6	-23.6	0	-10.0
6	-19.3	-25.3	-6.0	-5.0
7	-15.8	-15.3	+0.5	-5.25
8	-14.4	-16.9	-2.5	-5.0
9	-12.2	-15.2	-3.0	-8.5
10	-	-	-	-
11	-11.1	-16.6	-5.5	-6.5

A long groyne having a length of 210 m will induce deeper scour holes along the right bank downstream of the railway bridge than a short groyne having a length of 170 m, see Table D.3.9. This confirms the conclusion from the flow velocities, that a long groyne results in higher flow velocities along the right bank than a short groyne.

A short groyne will result in a reduction of the scour depths of about 5 m, if compared with the calibration test T23.

A short groyne will result in similar scour depths as in the test T20 with a continuous revetment at a slope 1 in 4.

D.3.6.5.4 Continuous revetment

In the model a continuous, advanced revetment at slopes 1:3 and 1:4 has been tested in tests T19 and T20. By extrapolating the observed tendency in the measured scour depths the equilibrium scour depth has been estimated, see Figure D.3.30. The method of extrapolation can only be applied if the bed erodes, not at locations with sedimentation.

The maximum scour along the revetment has been observed between cross sections 2 and 3 downstream of the railway bridge. The river bed protection near the railway bridge has a function similar to a sill for a scour hole downstream of it. The scour holes more downstream cross section 3 and along the right bank revetment are less deep than the scour hole between cross section 2 and 3.

The maximum scour depth along the revetment has not been measured accurately up to now. Possible reasons for this deficiency are the insufficient drainage of the model, the inaccurate reading of the measurements, and the definition of the locations of the measurements which were not optimal. A part of the river bed is covered by ripples, which disturb the scour depth measurements. To determine the net scour the variations by the ripples should be eliminated by averaging the scour depths.

TABLE D.3.10 THE EQUILIBRIUM SCOUR DEPTHS ALONG THE CONTINUOUS REVETMENT AND THE CALIBRATION TEST

cross sect.	calibration T23			revetment 1:3 T19			revetment 1:4 T20	
	equilibrium scour depth	bed level		equilib. scour depth	bed level		equil. scour depth	bed level
-	m	m		m	m		m	m
2	-11.9	-35.7		- 2.3	- 26.1		sed.	
3	- 4.6	-28.6		- 6.0	- 30.3		sed.	
4	- 6.9	-32.0		- 9.2	- 33.3		- 0.9	-24.9
5	-10.5	-34.1		-13.8	- 37.1		-13.8 -17.0	-37.4 -40.6
6	+ 5.0	-14.4		- 2.3	- 21.6		sed.	
7	- 5.0	-21.3		- 4.1	- 20.6		sed.	
8	- 7.3	-21.2		sed.			- 0 - 8.7	-23.1
9	- 9.2	-21.4		- 1.8	- 14.0		- 2.3	-14.5
10	n.m.	n.m.		n.m.	n.m.		n.m.	n.m.
11	- 7.3	-17.4		sed.			sed.	

n.m. = not measured
sed. = sedimentation

From Table D.3.10 the following can be concluded:

- The bed protection and the piers of the railway bridge cause a scour hole just downstream of the railway bridge (in cross section 2). The effect of the railway bridge on the scour process can in fact be compared with a sill.

- The reduction of the cross sectional area in the reach of cross sections 3 to 5 results in constriction scour.

- The reduction of the slope of the bank results in a reduction of the scour depths (cross section 6 to 11).

TABLE D.3.11 MAXIMUM RECOMMENDED SCOUR DEPTHS

cross section	calibration	revetm. 1:3	revetm. 1:4
-	m	m	m
2	10 to 12.5	10	10
3 to 5	5 to 7.5	10	10
6 to 11	5 to 7.5	2.5 to 5	0 to 2.5

From Table D.3.11 and Figure D.3.31 with the recommended maximum scour depths, it follows:

- The relation between the scour depths in the calibration test, T19 and T20 is the combination of the effect of the constriction (i.e. the reduction of the cross sectional area) and the effect of the bank slope.
  The effect of the constriction of the cross section is estimated at 3 to 5%, which will result in approximately 5 to 10% increase in scour depth.

- Downstream of cross section 6 the scour depth is relatively small. The maximum scour depth occurs between cross section 2 and 6, that is just downstream of the railway bridge.

It is noted that the design scour depths in Table D.3.11 are based on a 1 in 100 year flood, of which the maximum discharge is supposed to have a duration of some weeks. This period is sufficient to reach the equilibrium scour depths.

In the prototype the scour depths can be estimated by the formula of Ahmad, ref.[1], and supported by Breusers, ref.[5]:

$$h_0 + y_s = k \cdot q^{0.67}$$

The scour depth is only a function of the initial water depth and the flow velocity, which can be seen from the rewritten formula:

$$y_s = k \cdot u_0^{0.67} \cdot h_0^{0.67} - h_0$$

in which

y_s	= scour depth	(m)
h_0	= initial water depth	(m)
k	= coefficient	$(m^{-0.34}\ s^{0.67})$
u_0	= flow velocity	(m/s)
q	= specific discharge	(m^2/s)

The specific discharge is defined as q = Q/b, with b = channel width (m). From the design scour depths in Table D.3.11 the values of the coefficient k are determined in the following Table D.3.12.

TABLE D.3.12 RECOMMENDED VALUES OF COEFFICIENT K IN THE FORMULA *OF AHMAD*

cross section	water depth	width	q	calibration	revetment 1:3	revetment 1:4
-	m	m	m^2/s	k ($m_{-.34}$ $s^{0.67}$)	k ($m^{-0.34}$ $s^{0.67}$)	k ($m^{-0.34}$ $s^{0.67}$)
2	16.2	800	27.5	2.8 to 3.1	2.8	2.8
3 to 5	26.0	500	44.0	2.4 to 2.7	2.8	2.8
6 to 11	20.0	650	33.8	2.4 to 2.6	2.1 to 2.4	1.9 to 2.1

Earlier investigations have resulted in a recommended value of k between 2.1 and 3.2. These values are confirmed by the results of the model.

It is mentioned that with an optimal alignment of the bank between cross sections 3 to 5 probably a reduction in the value of k can be obtained. In the model this alignment was not optimal.

The accuracy of the scour measurements can be estimated from the variation in the initial bed level. In each test the initial bed level should be the same.

TABLE D.3.13 THE INITIAL BED LEVEL IN TESTS T19, T20 AND T23

cross section	calibration	revetm. 1:3	revetm. 1:4
-	m-P.W.D.	m-P.W.D.	m-P.W.D.
2	- 23.8	- 23.8	- 25.0
4	- 25.1	- 24.1	- 24.0
5	- 23.6	- 23.3	- 23.6
9	- 12.2	- 12.2	- 12.2

From Table D.3.13 it follows that the variation in the initial bed level can be as high as 1 meter.

D.3.6.5.5 Sill in the Railway Bridge

Two types of sills in the bridge spans 9 - 11 have been tested, and the scour depths of the high sill in test T18 has been analyzed. By extrapolating the observed tendency in the measured bed levels in the scour hole the equilibrium scour depth has been estimated, see Figure D.3.30.

The maximum scour along the existing bank of Bhairab Bazar has been observed in cross section 4 to 6, see Figure D.3.31 and/or Table D.3.14. Compared with the calibration test T23 the scour depth just downstream of the bridge is rather small: only 4 to 5 m instead of 10 to 12.5 m. The high sill reduces the discharge passing the sill so much that a possible increase in turbulence intensity has been compensated and the size of the resulting scour hole is reduced. In the downstream section (cross sections 7 to 11) the sill has still some effect in reducing the scour depth: from 5 to 7.5m in the calibration test to 1 to 2 m in T18. This reduction is remarkable, see also Figure D.3.31.

The following scour depths are recommended for design purposes:

TABLE D.3.14 THE DESIGN SCOUR DEPTHS FOR A HIGH SILL, COMPARED WITH THE SCOUR DEPTHS IN THE CALIBRATION

cross section	calibration	high sill	low sill
-	m	m	m
2	10 to 12.5	4 to 5	10 to 14
3	5 to 7.5	4 to 5	8 to 10
4 - 6	5 to 7.5	5 to 7.5	8 to 10
7 - 11	5 to 7.5	1 to 2	7 to 9

TABLE D.3.15 CALCULATED AND RECOMMENDED VALUES OF COEFFICIENT K IN THE FORMULA OF AHMAD FOR THE SILL ALTERNATIVE, COMPARED WITH THE CALIBRATION TEST

cross section	water depth	width	q	calibration	high sill	low sill
-	m	m	m2/s	k ($m^{-0.34}$ $s^{0.67}$)	k ($m^{-0.34}$ $s^{0.67}$)	k ($m^{-0.34}$ $s^{0.67}$)
2	16.2	800	27.5	2.8 to 3.1	2.2 to 2.3	2.8 to 3.3
3	26.0	500	44.0	2.4 to 2.7	2.4	2.7 to 2.9
4 to 6	26.0	500	44.0	2.4 to 2.7	2.4 to 2.7	2.7 to 2.9
7 to 11	20.0	650	33.8	2.4 to 2.6	2.0 to 2.1	2.5 to 2.7

The following remarks are deduced from Tables D.3.14 and D.3.15:

- A low sill gives apparently no improvement compared with the calibration test. This is an unexpected result and this means probably that this test is not so accurate. A small reduction in scour depths by a sill should be expected and recommended for design purposes.

- A high sill results in some reduction of the scour depths and the value of k. This tendency is to be expected. The influence of the sill can be observed to a distance rather remote from the sill, e.g. up to cross section 11.

81

REFERENCES

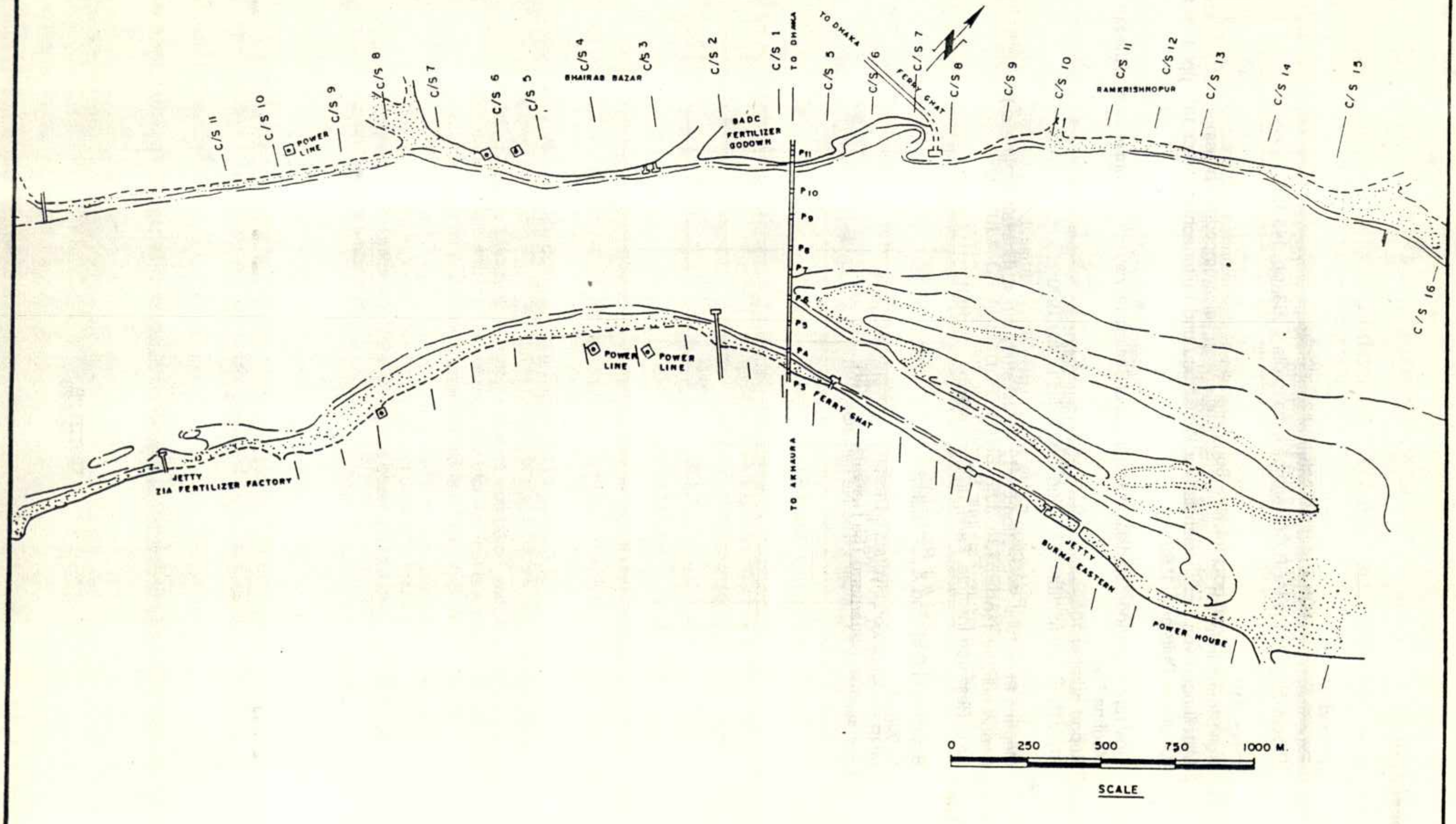
1 Ahmad, M.
Experiments on design and behaviour of spur dikes
Proceedings of the IAHR Conference Minnessota, 1953, pp. 145-159

2 Dietz, J.W.
Kolkbildung in feinen oder leichten Sohlmaterialen bei stromenden Abfluss
Mitteilungen von Versuchsanstalt fur Wasserbau und Kulturtechnick, Heft 155, pp. 1 ... 120, Karlsruhe, Duitsland, 1969

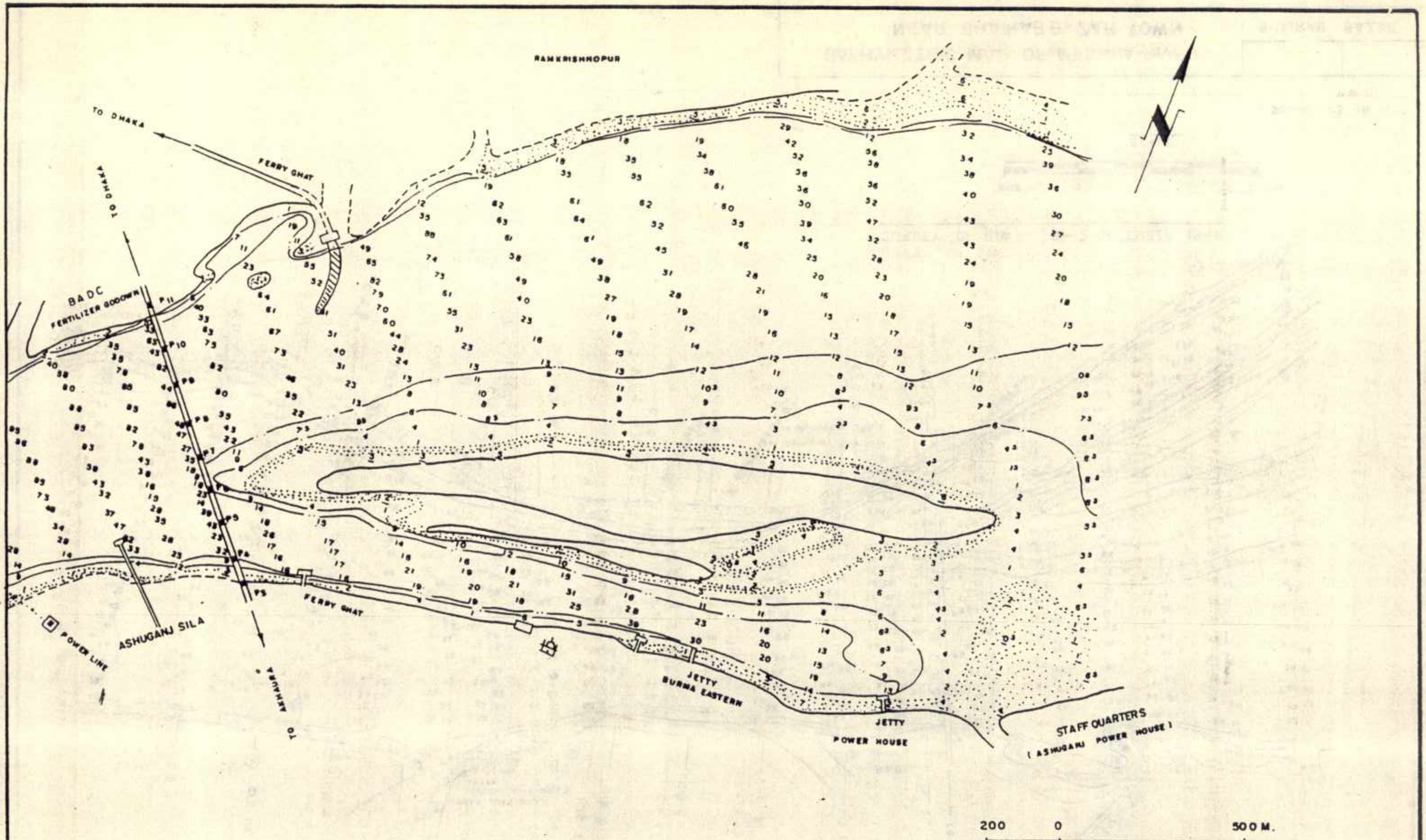
3 Final report on model studies for protection of Bhairab railway bridge from erosion of the Surma-Meghna River.
Report number 106,
River Research Institute, Faridpur, Bangladesh, July 1990

4 Meghna River Bank Protection Study, Model study report (Bhairab Bazar area)
Report number 110,
River Research Institute, Faridpur, Bangladesh, July 1991

5 Breusers H.C.M. and A.J. Raudkivi
Scouring
IAHR monograph on Hydraulic Design Considerations
A.A. Balkema, Rotterdam, The Netherlands, 1991



INDEX MAP	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MRBPSTS | FIG. D. 3·1

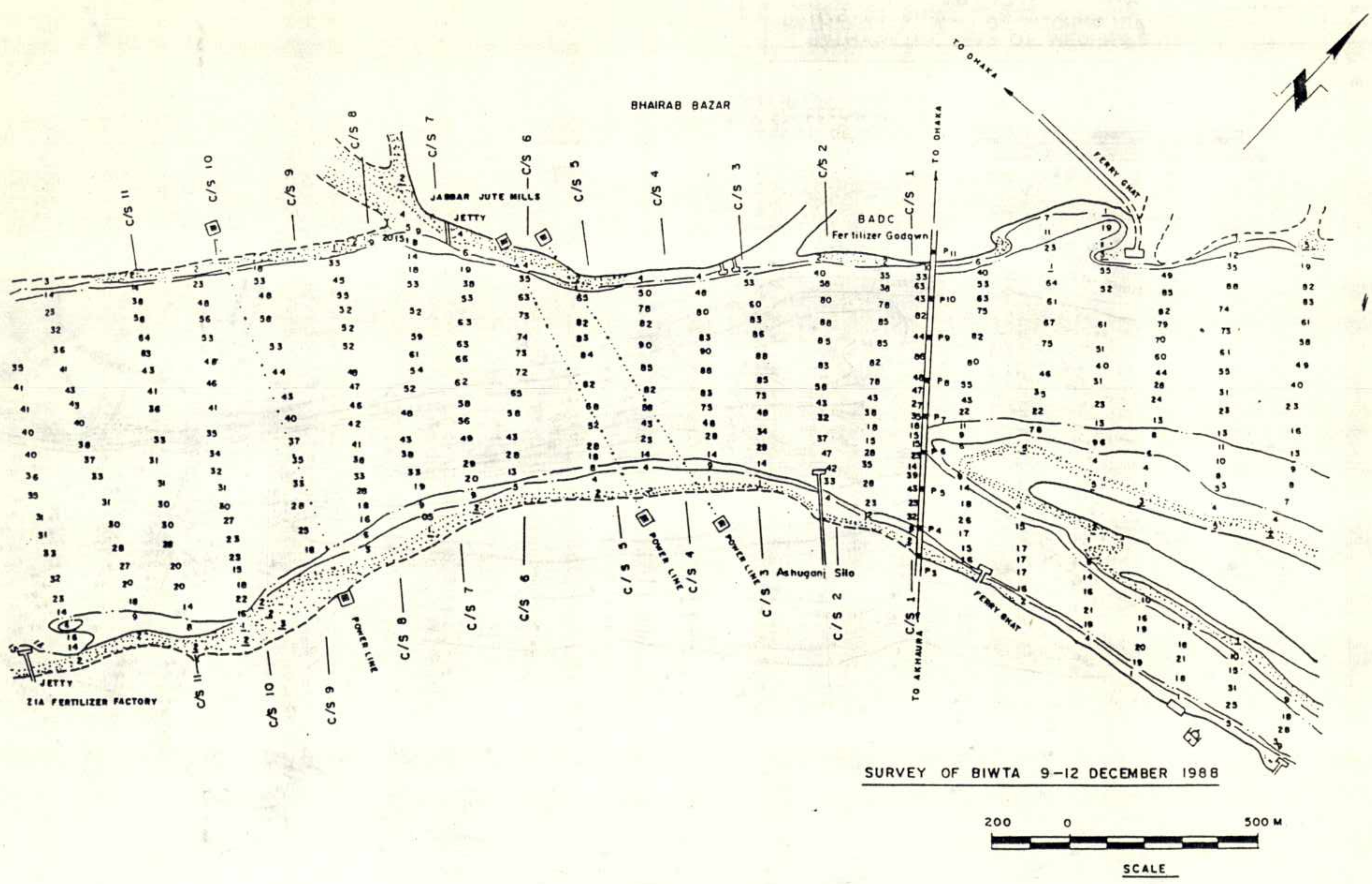


SURVEY OF BIWTA
9 – 12 DECEMBER 1988

SOUNDINGS IN FEET
PWD

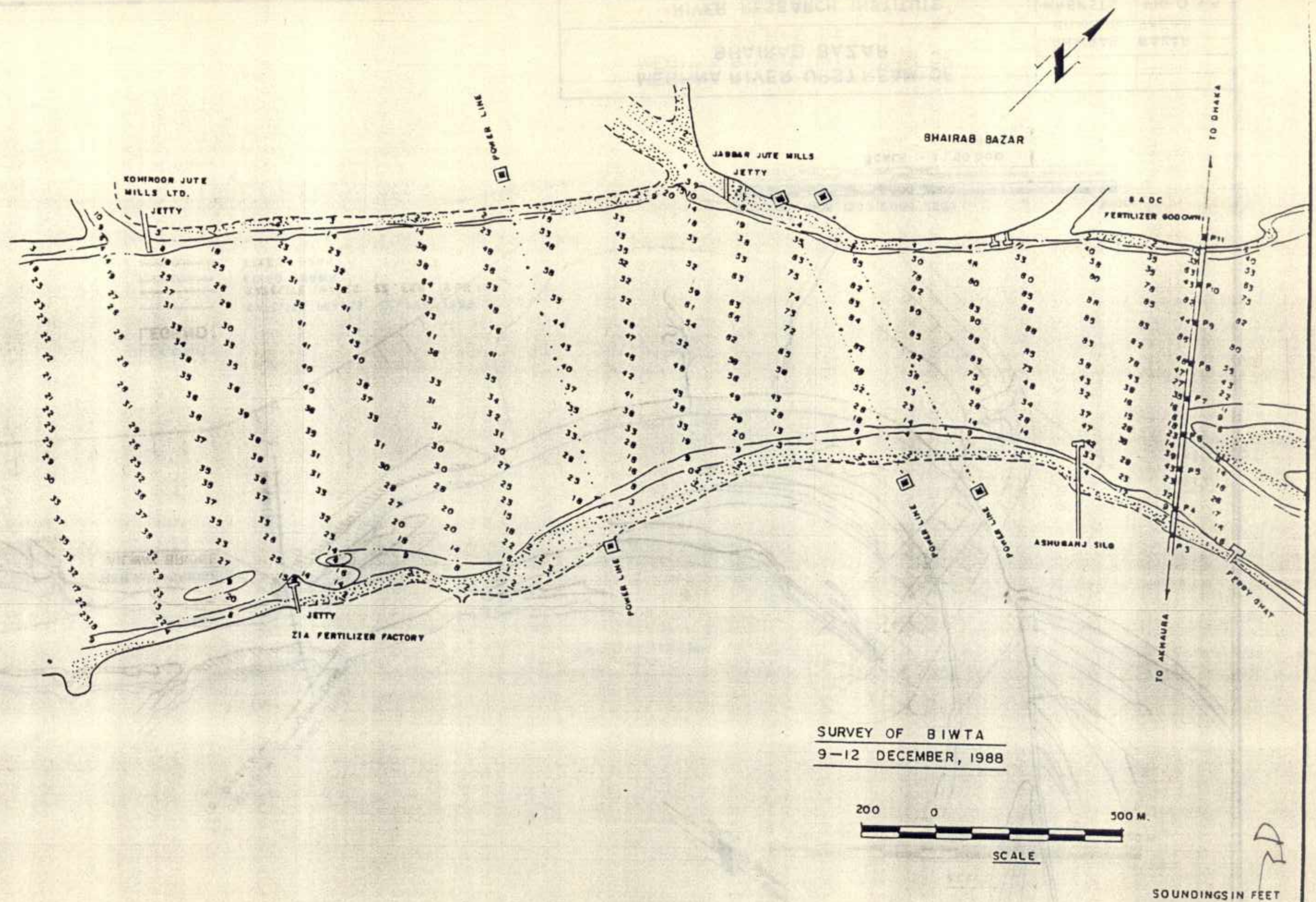
BATHYMETRIC MAP OF MEGHNA RIVER U/S OF BHAIRAB BAZAR RAILWAY BRIDGE	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D-3.2

90



SOUNDINGS IN FEET
PWD

BATHYMETRIC MAP OF MEGHNA RIVER NEAR BHAIRAB BAZAR TOWN	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MPPRSTS FIG. D.33

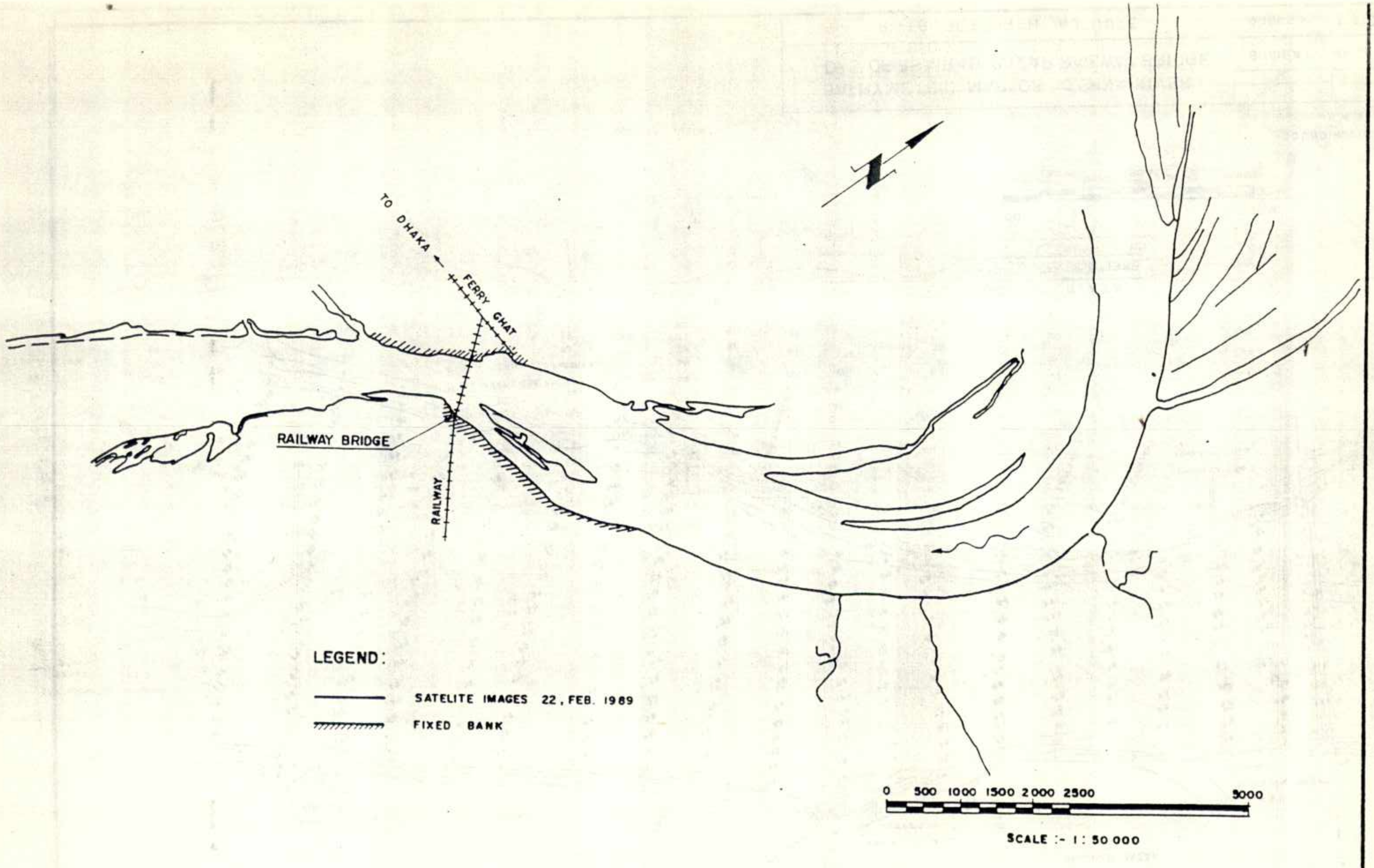


BATHYMETRIC MAP OF MEGHNA RIVER
D/S OF BHAIRAB BAZAR RAILWAY BRIDGE

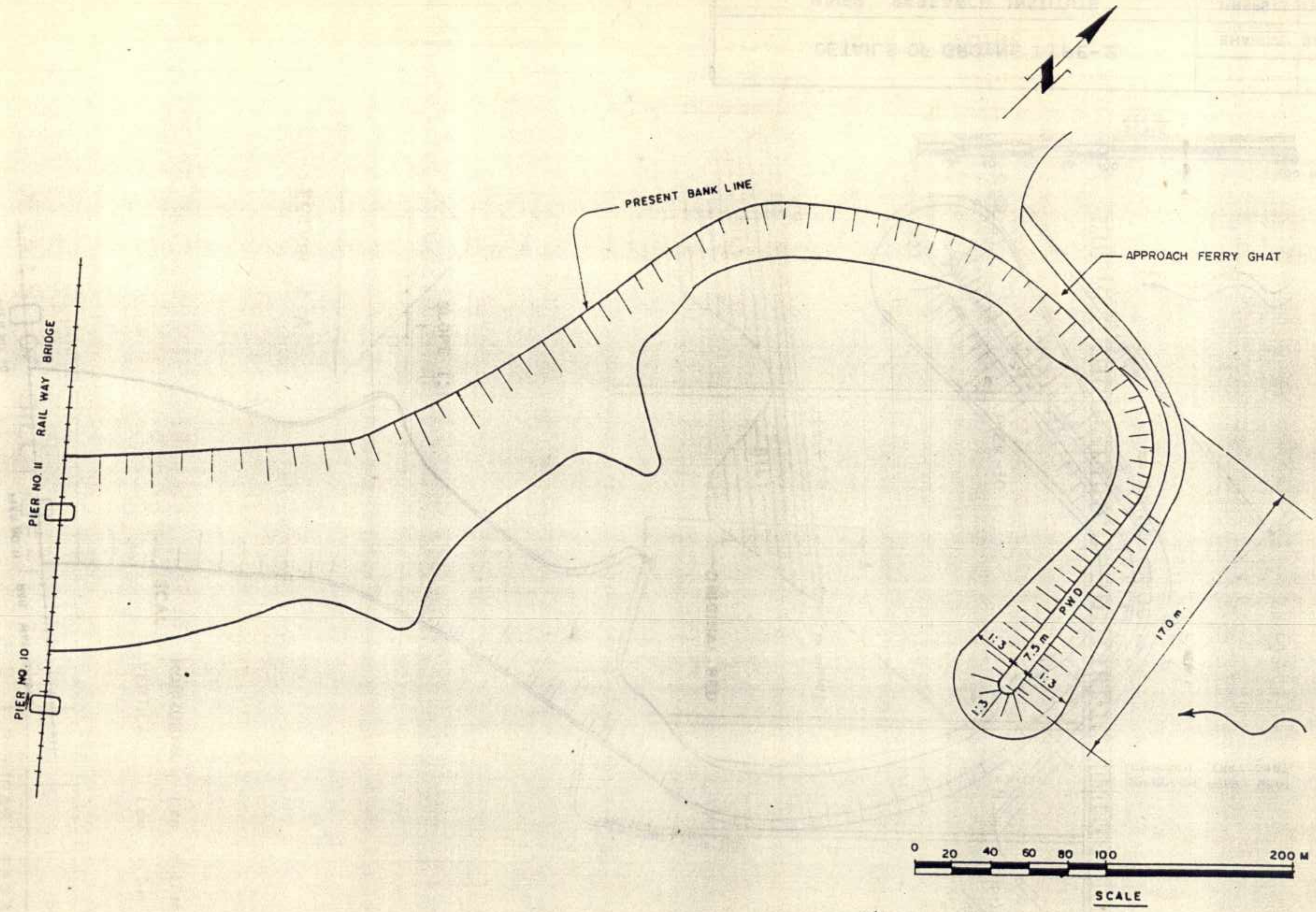
BHAIRAB BAZAR

RIVER RESEARCH INSTITUTE

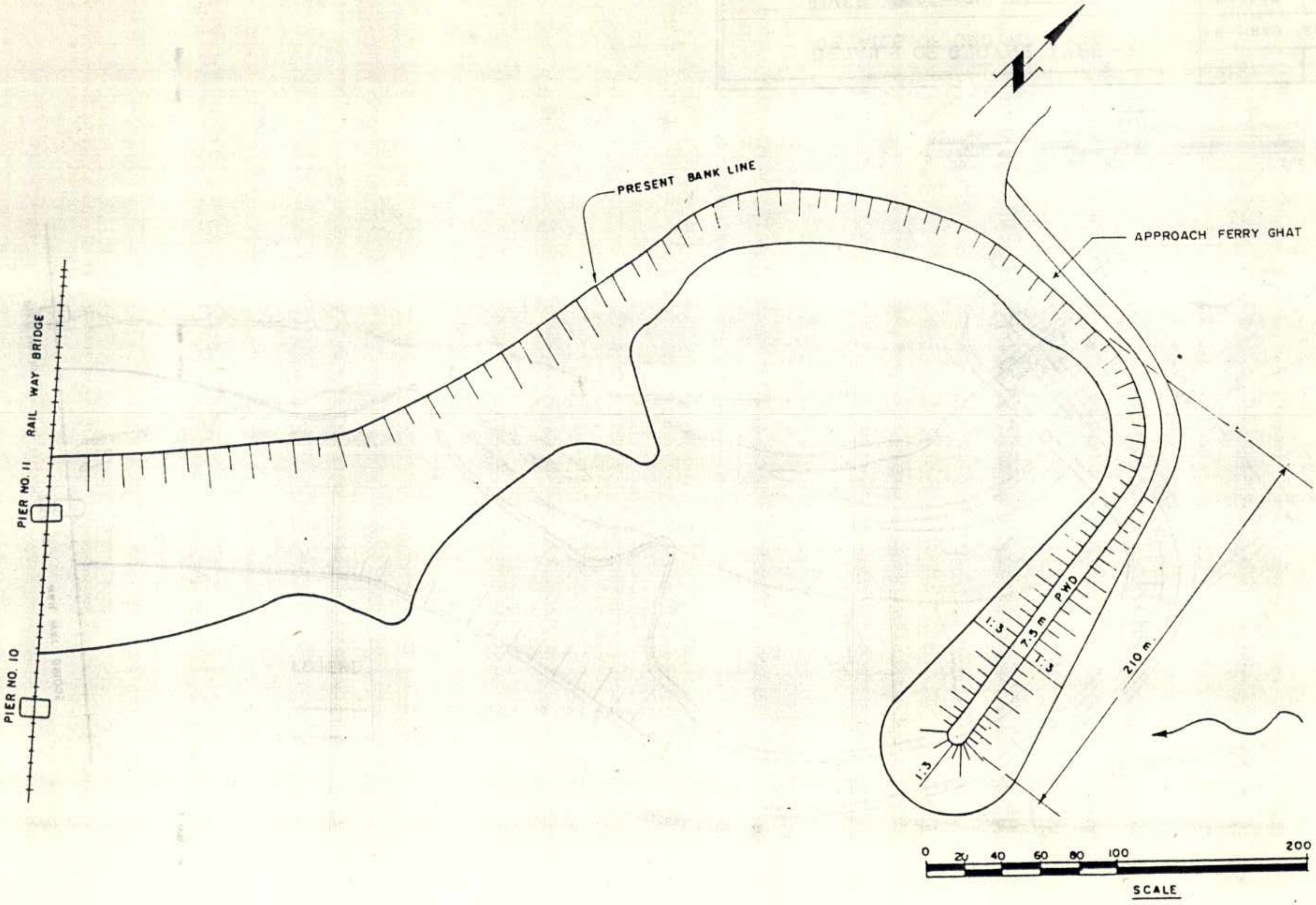
MRBPSTS | FIG D.3.4



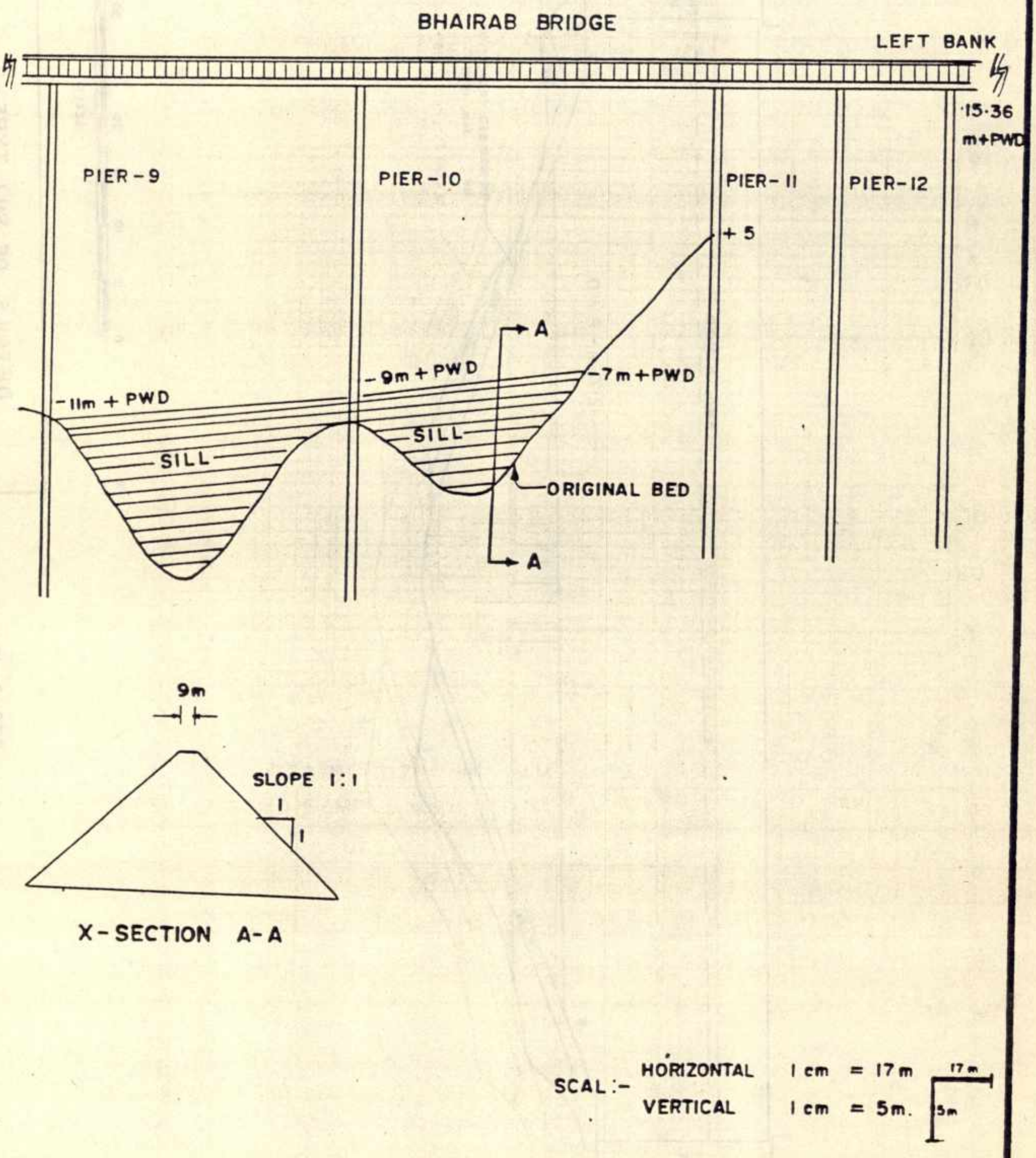
MEGHNA RIVER UPSTREAM OF BHAIRAB BAZAR	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D.3.5



DETAILS OF GROYNE TYPE - I	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D.3.6

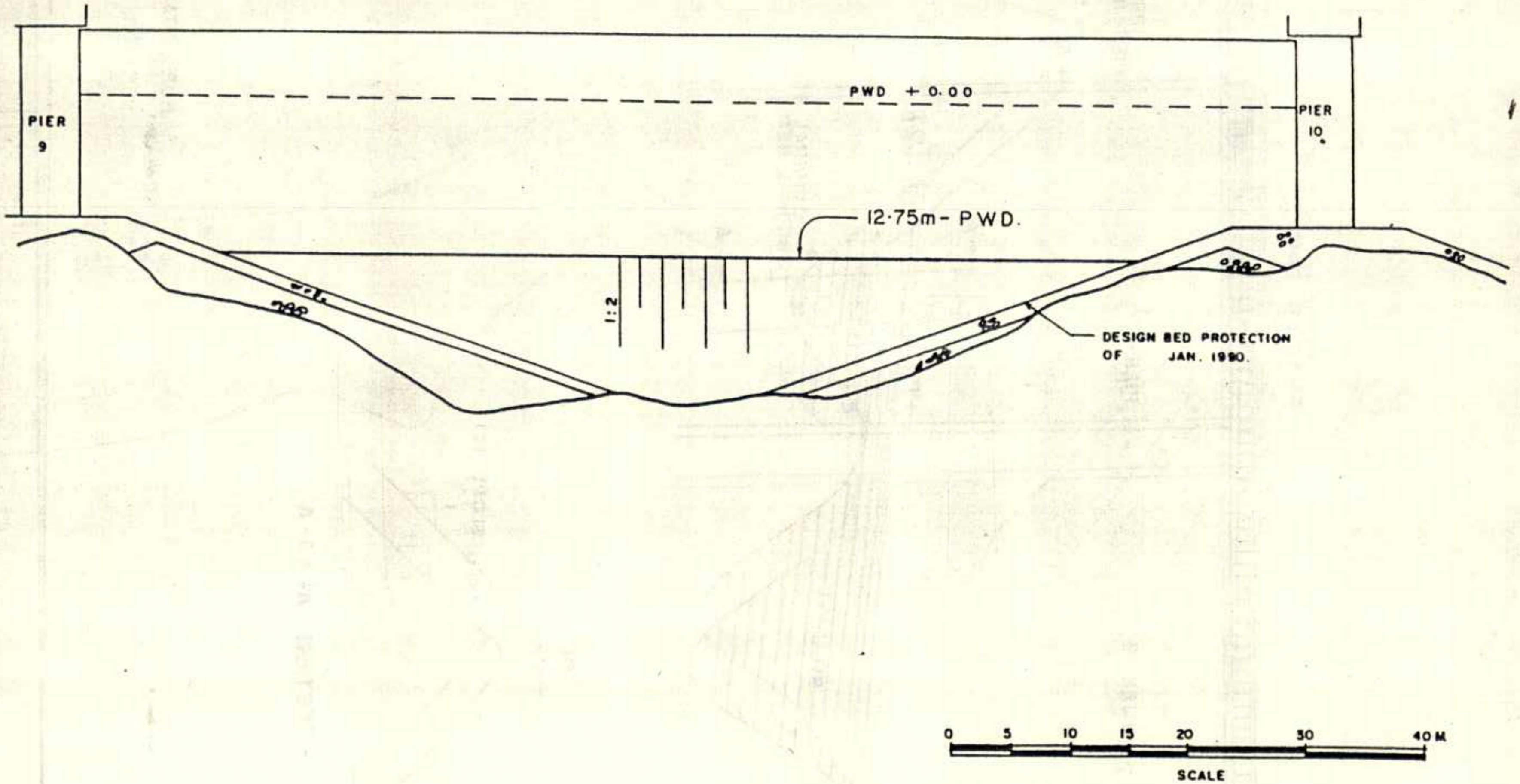


DETAILS OF GROYNE TYPE-2	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MRBPSTS FIG D.3.7



DETAILS OF SILLTYPE-I

RIVER RESEARCH INSTITUTE	BHAIRAB BAZAR	
	MRBPSTS	FIG. D.3.8

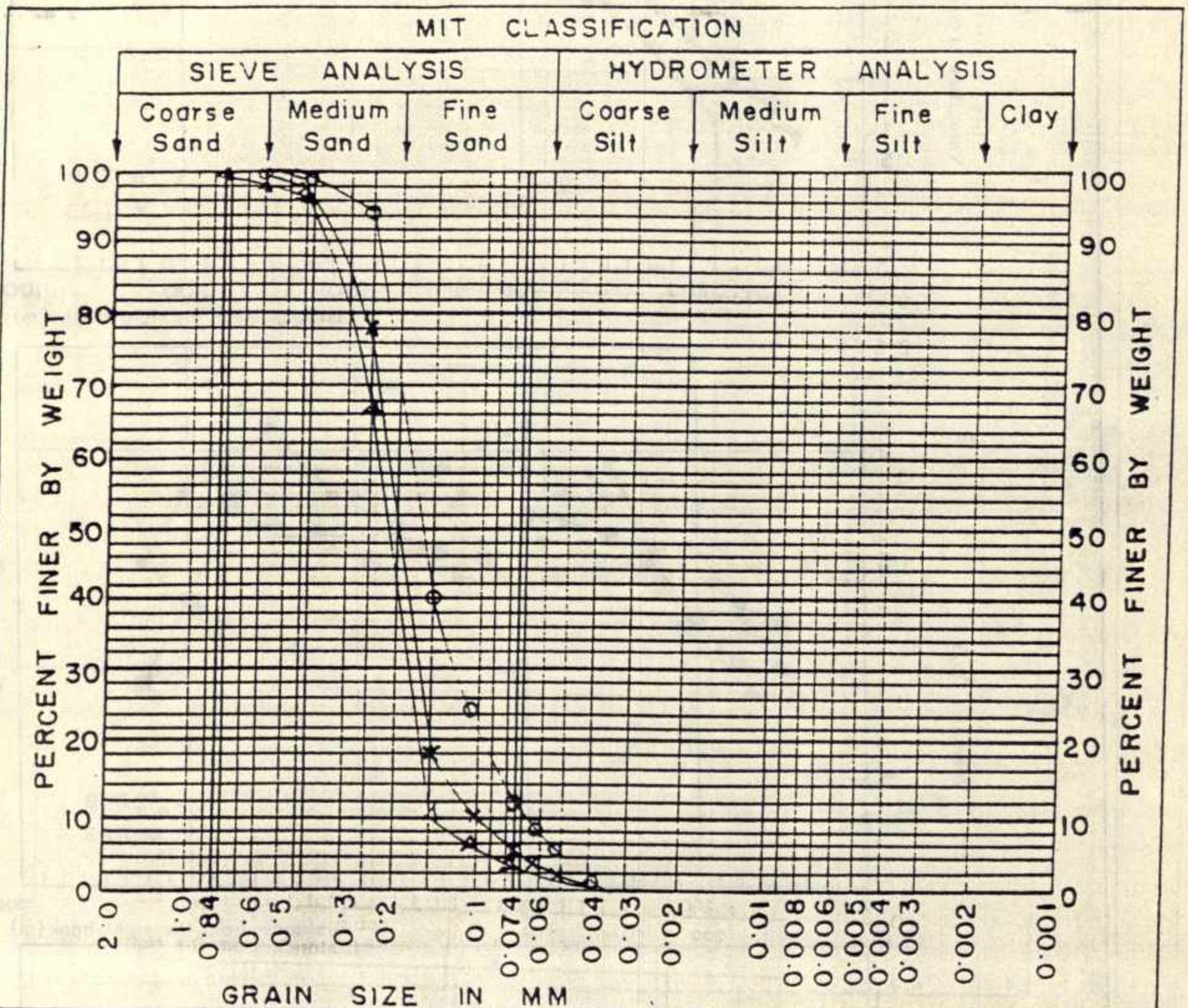


PIER 9 – 10 & PIER 10 – 11 | SAME

DETAILS OF SILL TYPE - 2		
	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D. 3.9

GRADATION CURVES

RIVER :— STATION :—

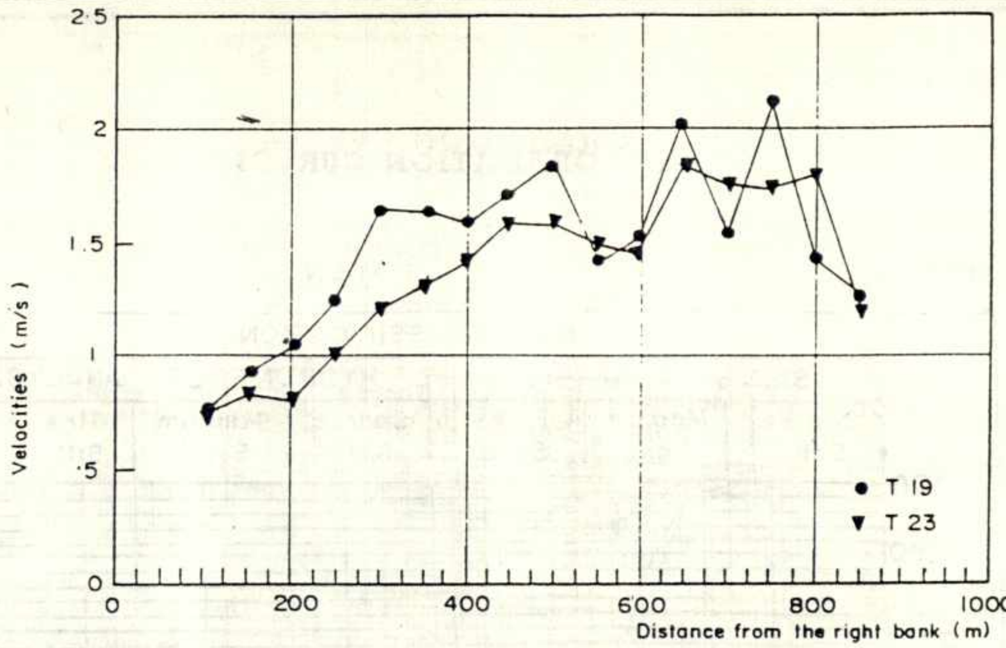


DATE	LEGEND	SAMPLE NO	DEPTH, FT.	D_{65} mm	D_{50} mm	D_{90} mm	Sp. Gr.
17.9.91	⊖ ⊖ ⊖	U/S	C/S-15	—	0.169	0.070	
"	△ △ △	Near Bridge		—	0.219	0.00	
"	× × ×	D/S	C/S-11	—	0.200	0.150	

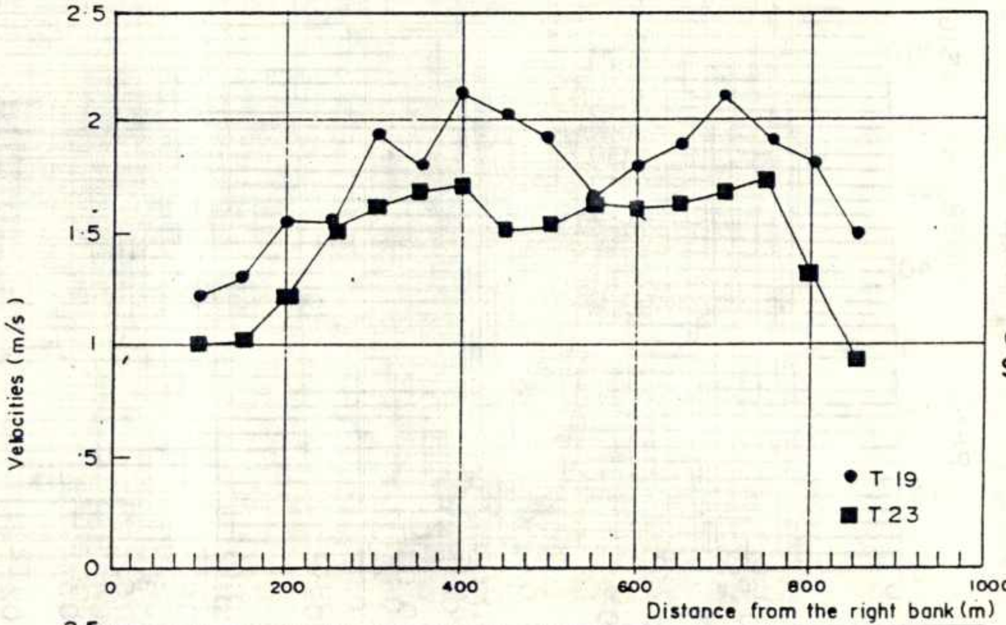
GRADATION CURVES OF BED MATERIAL

BHAIRAB BAZAR

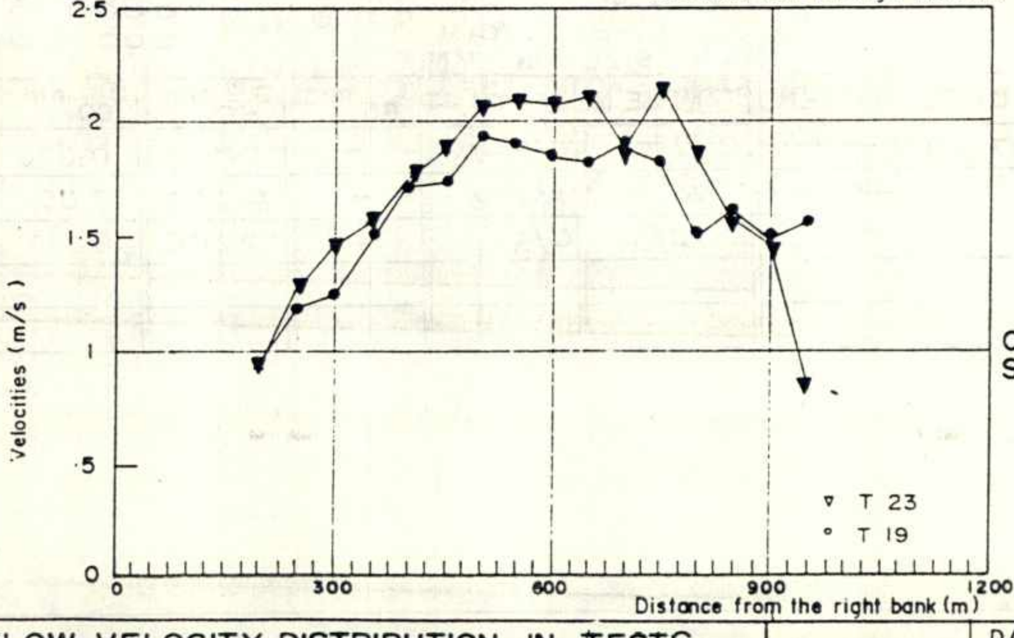
MRBPSTS FIG. D.3.10



CROSS SECTION-3

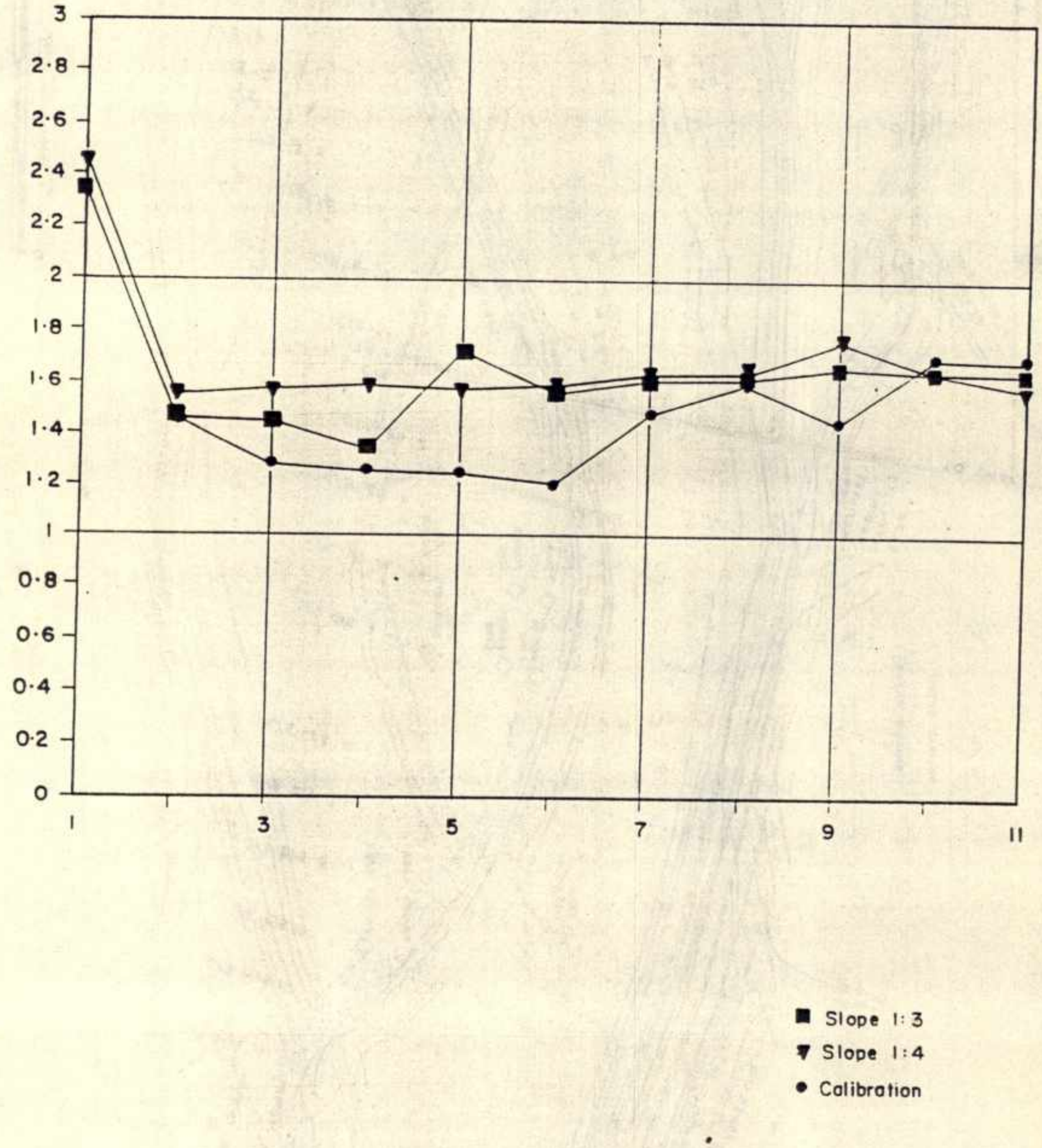


CROSS SECTION-6

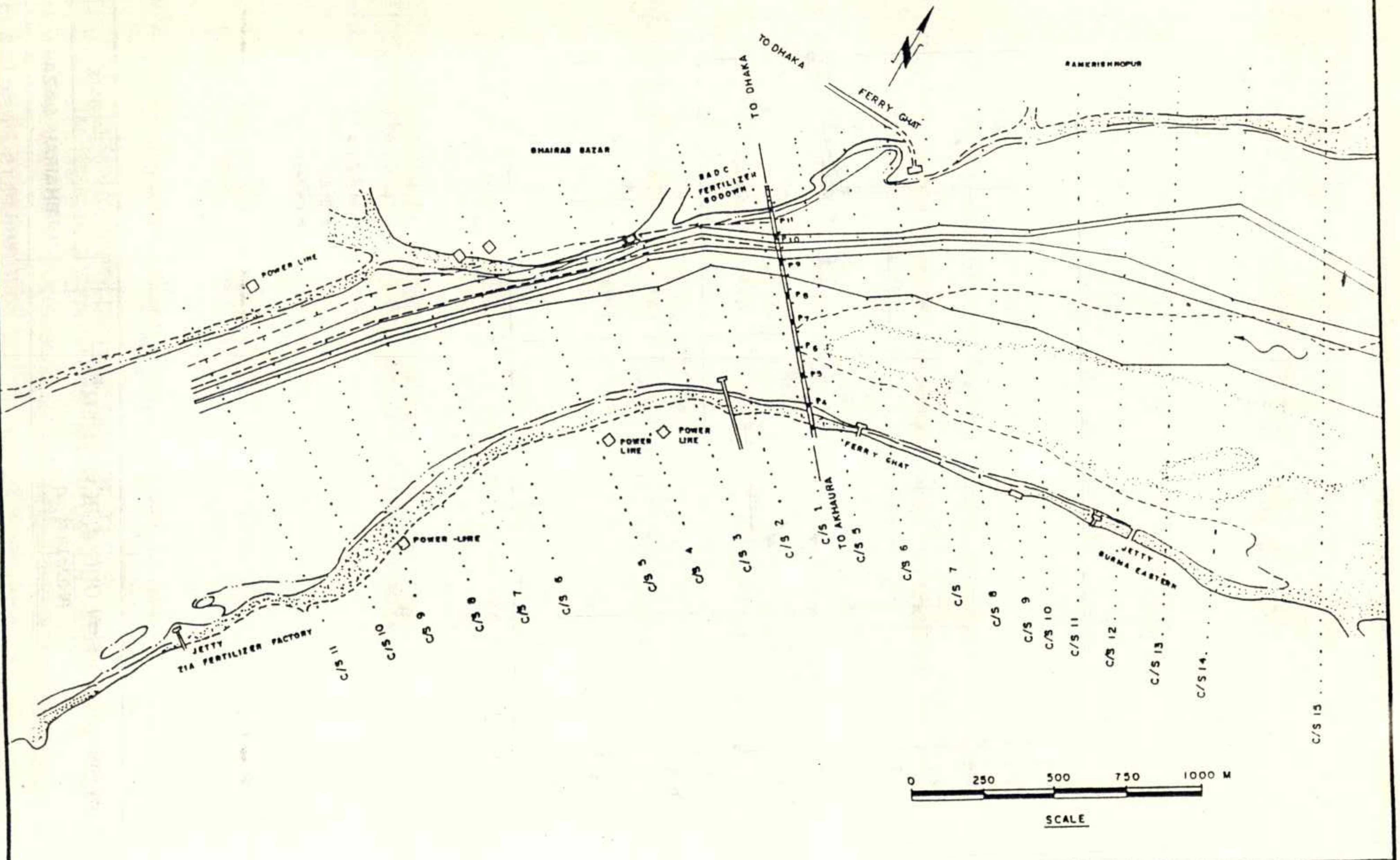


CROSS SECTION-11

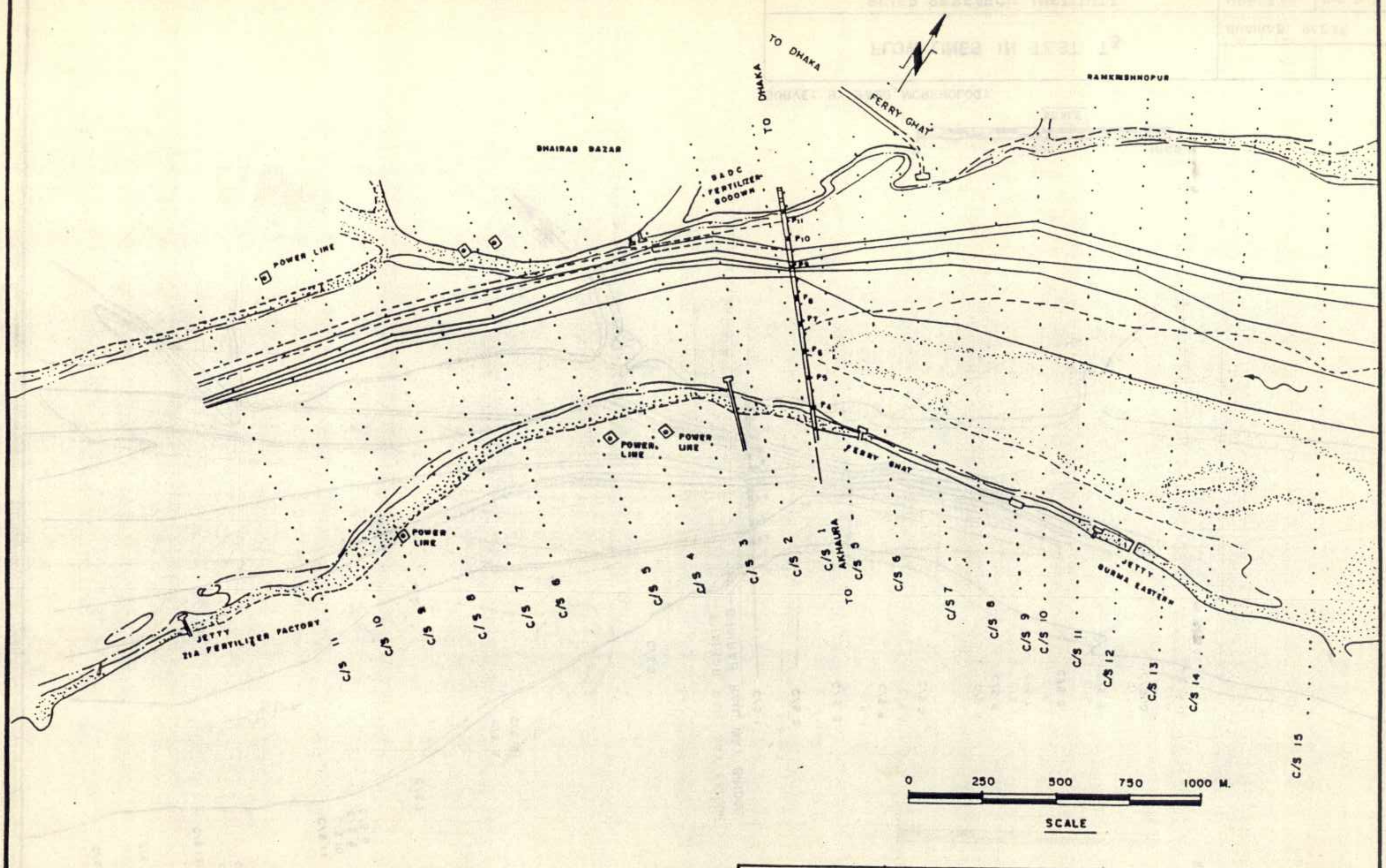
FLOW VELOCITY DISTRIBUTION IN TESTS WITH REVETMENT AND EXISTING SITUATION	DATE	
	BHAIRAB BAZAR	
	MRBPSTS	FIG. D.3.11



FLOW VELOCITIES ALONG A REVETMENT 1:3 AND 1:4		DATE
	BHAIRAB BAZAR	
	MRBPSTS	FIG. D. 3.12



FLOW LINES IN TEST T1, CALIBRATION	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D.3.13

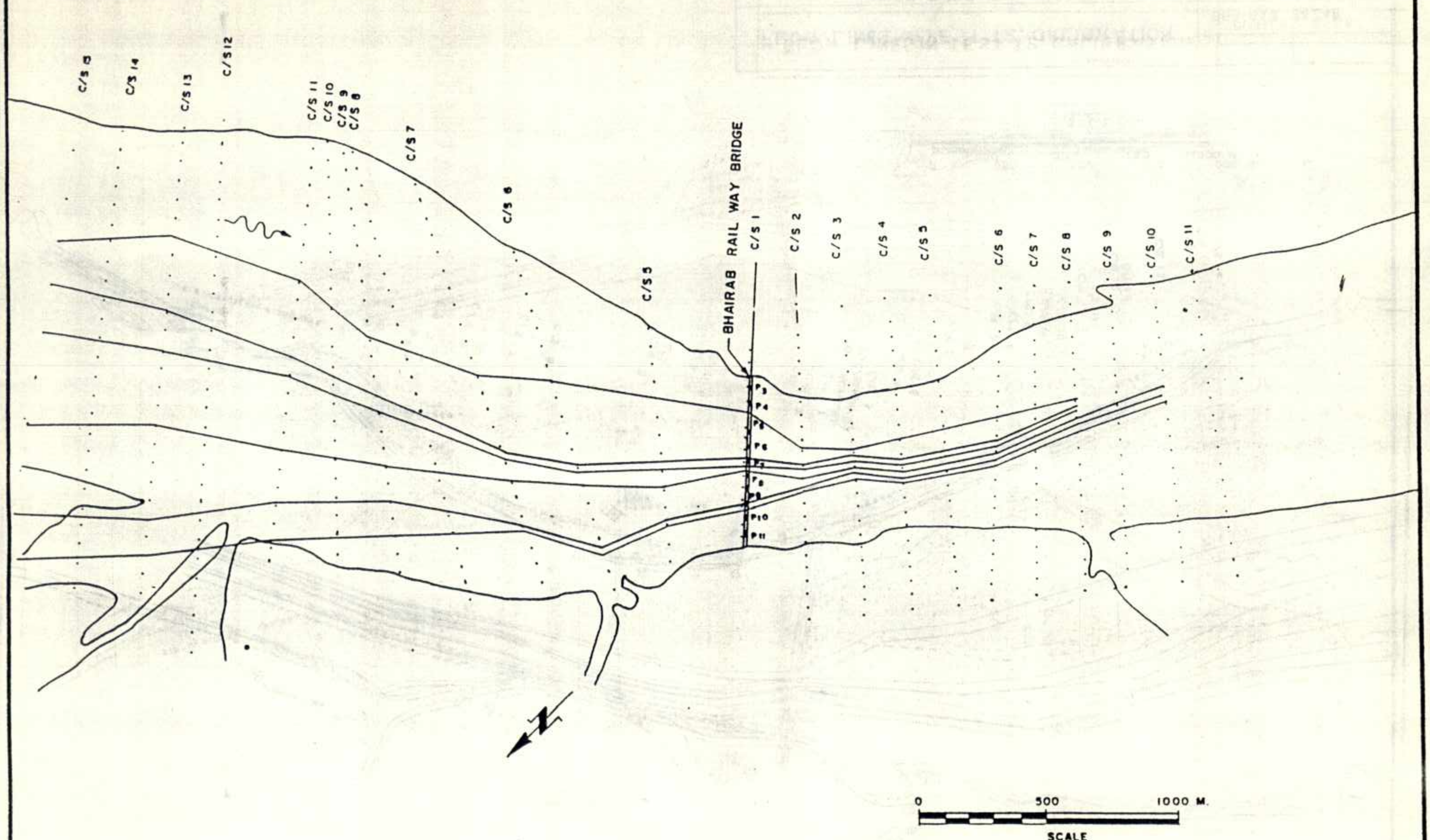


FLOW LINES IN TEST T2, CALIBRATION

RIVER RESEARCH INSTITUTE

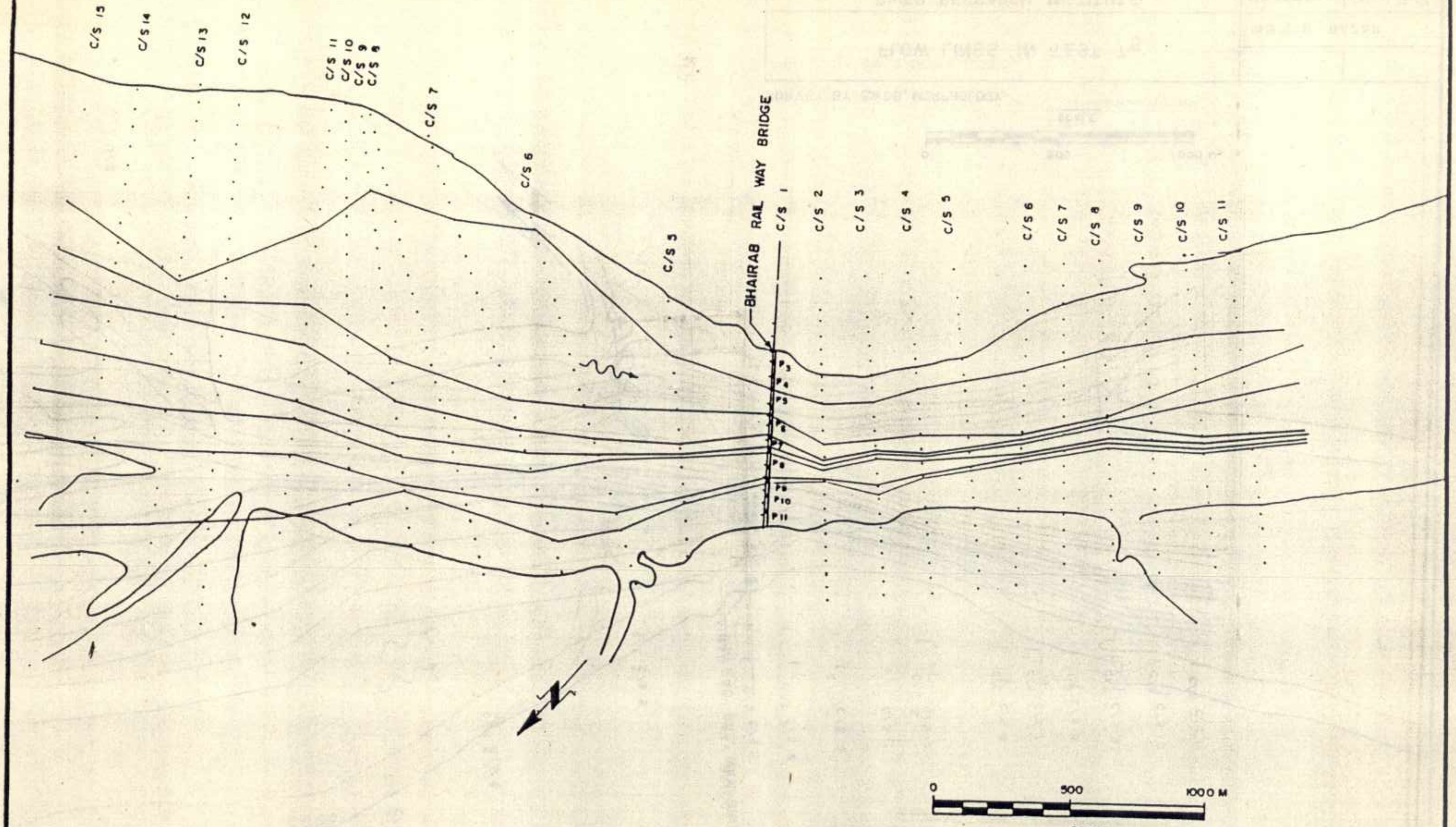
BHAIRAB BAZAR

MRBPSTS | FIG. D.3.14



SURVEY BY BWDB, MORPHOLOGY.

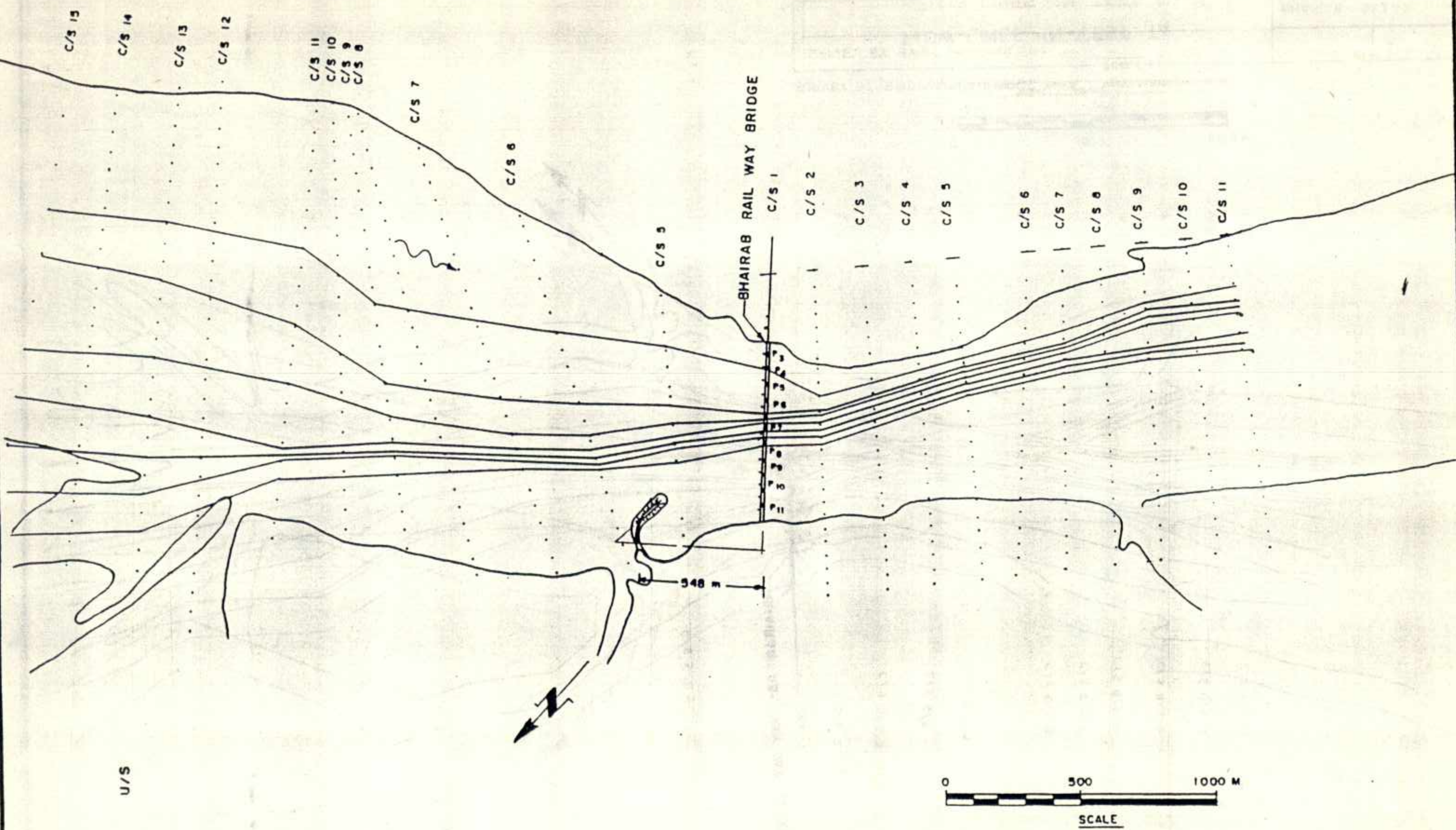
FLOW LINES IN TEST T_3	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MRBPSTS FIG. D.3.15



SURVEY BY BWDB, MORPHOLOGY.

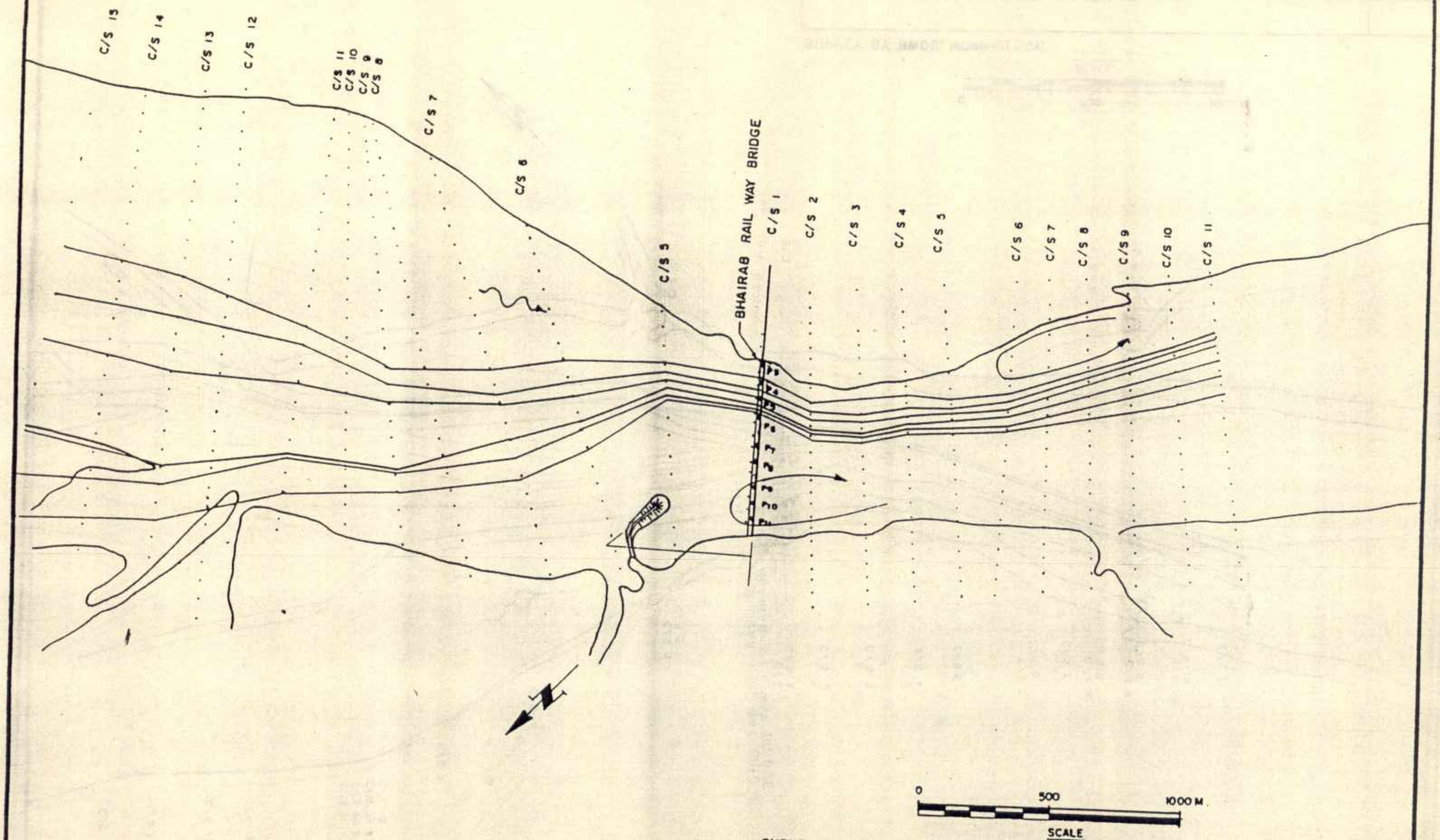
FLOW LINES IN TEST T_4	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG.D.3.16

68



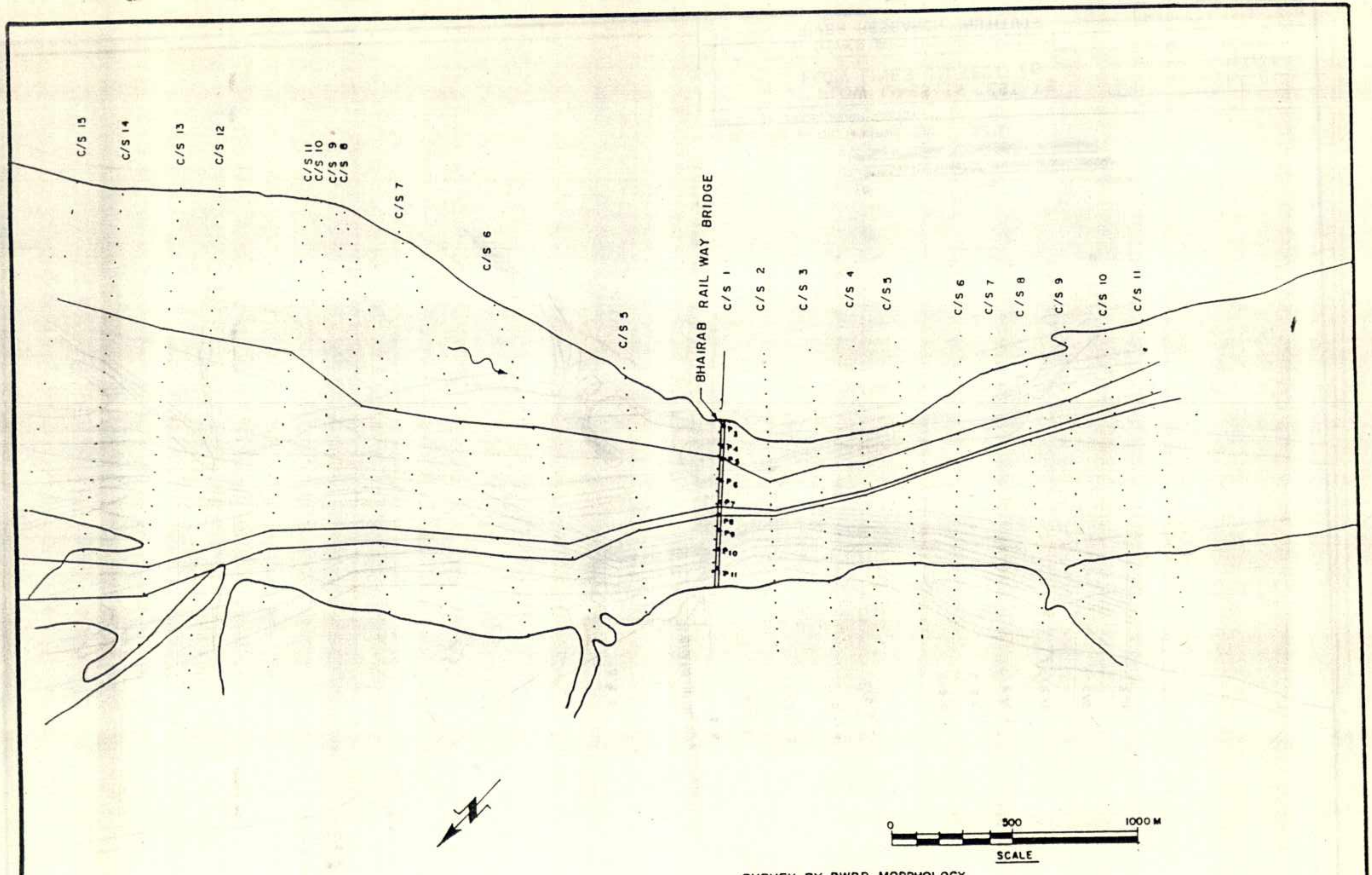
SURVEY BY BWDB, MORPHOLOGY.

FLOW LINES IN TEST T_5	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG.D.3.17



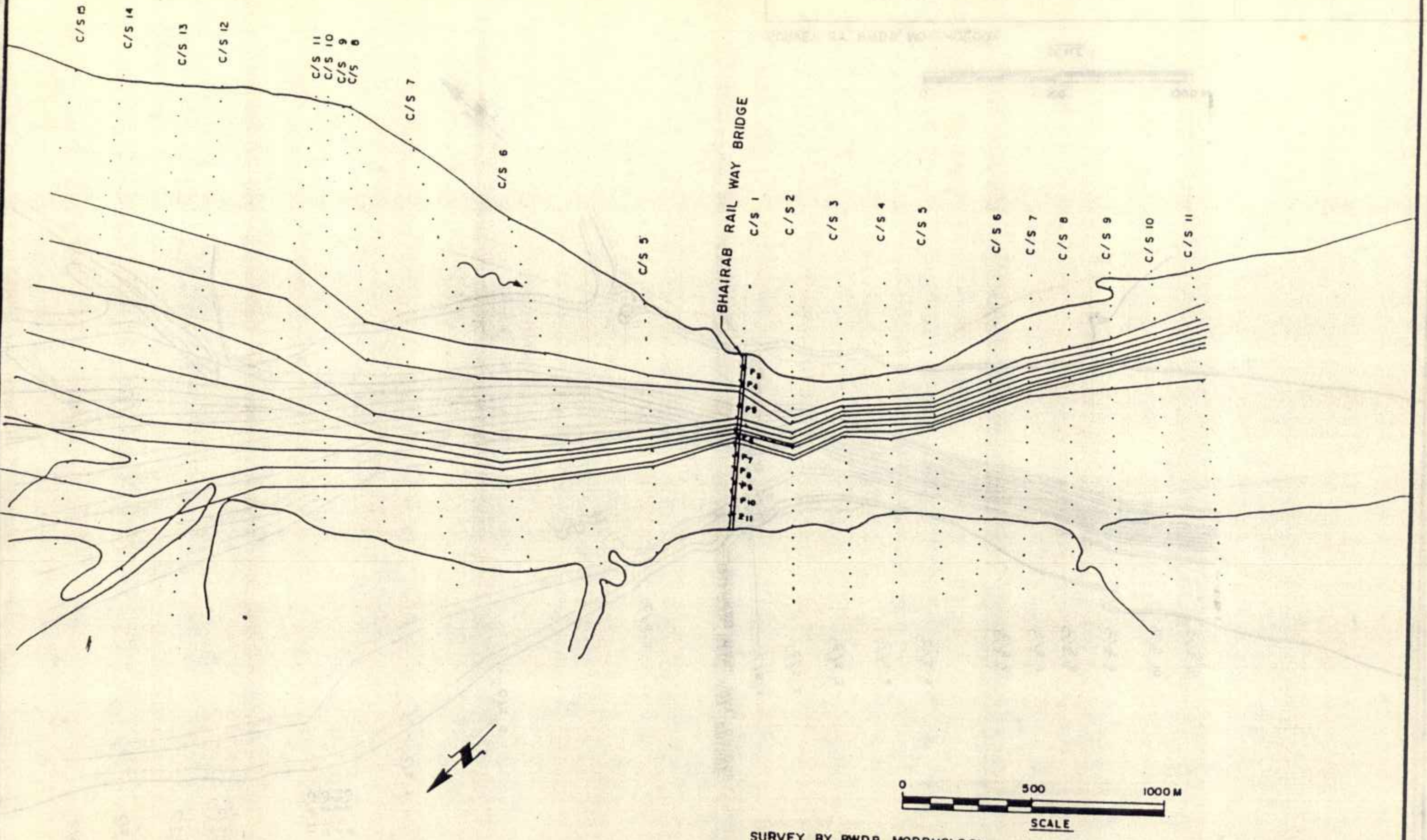
SURVEY BY BWDB, MORPHOLOGY.

FLOW LINES IN TEST T6	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MR BPSTS / FIG. D.3.18



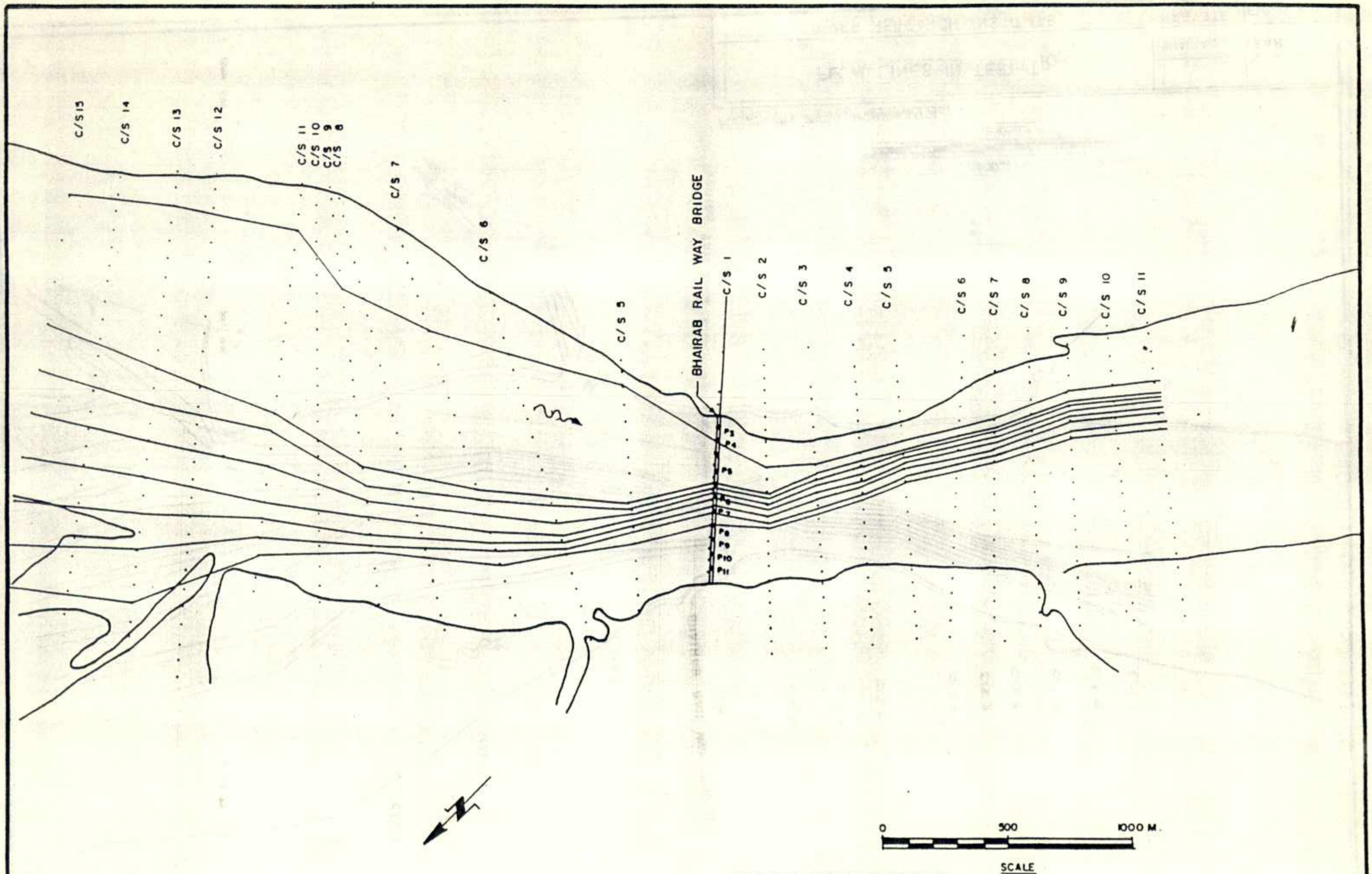
SURVEY BY BWDB, MORPHOLOGY.

FLOW LINES IN TEST T9	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG.D.3.19



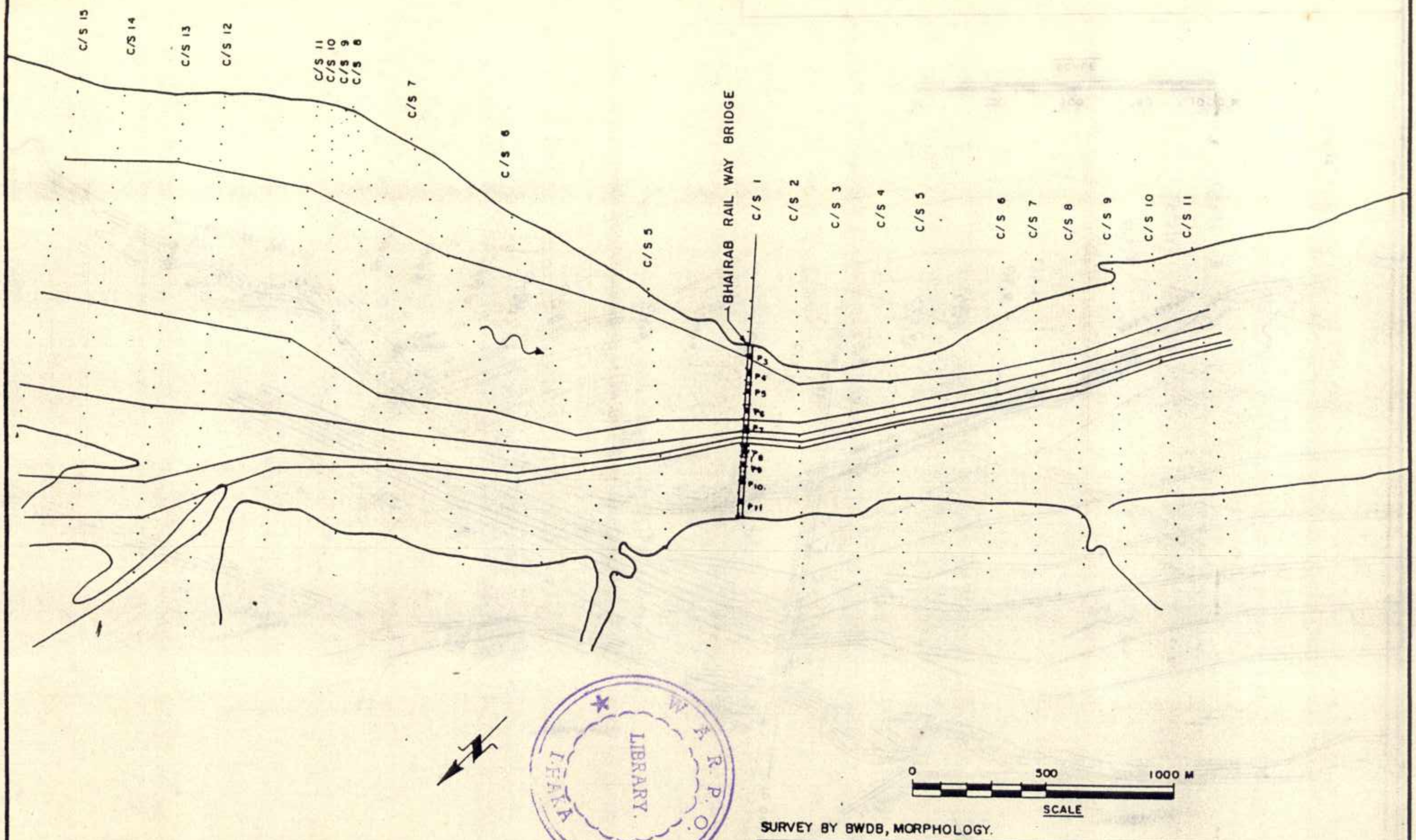
SURVEY BY BWDB, MORPHOLOGI.

FLOW LINES IN TEST T_{10}	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MRBPSTS | FIG. D.3.20



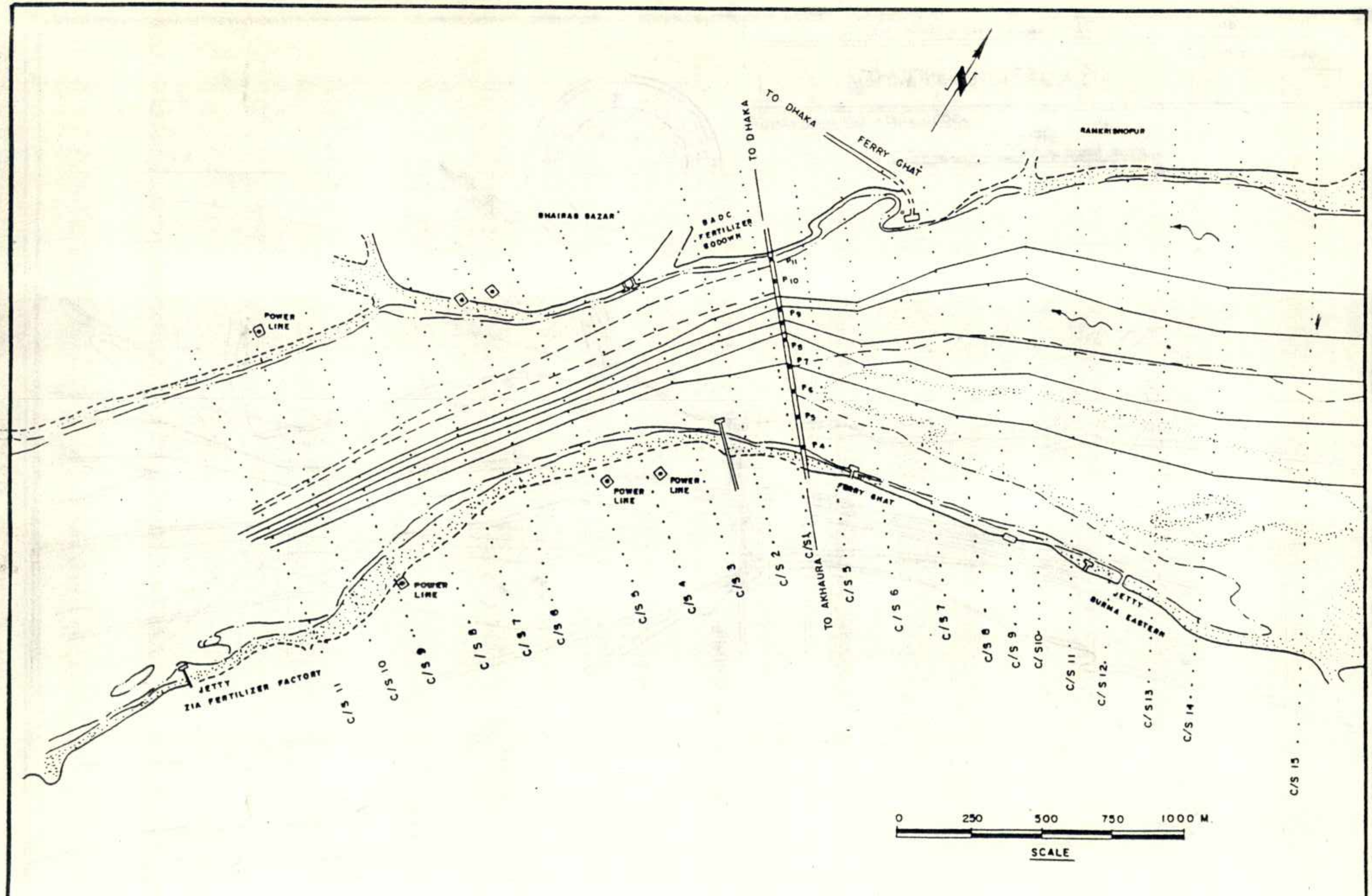
SURVEY BY BWDB, MORPHOLOGY.

FLOW LINES IN TEST T_{II}	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MRBPSTS FIG D.3.21

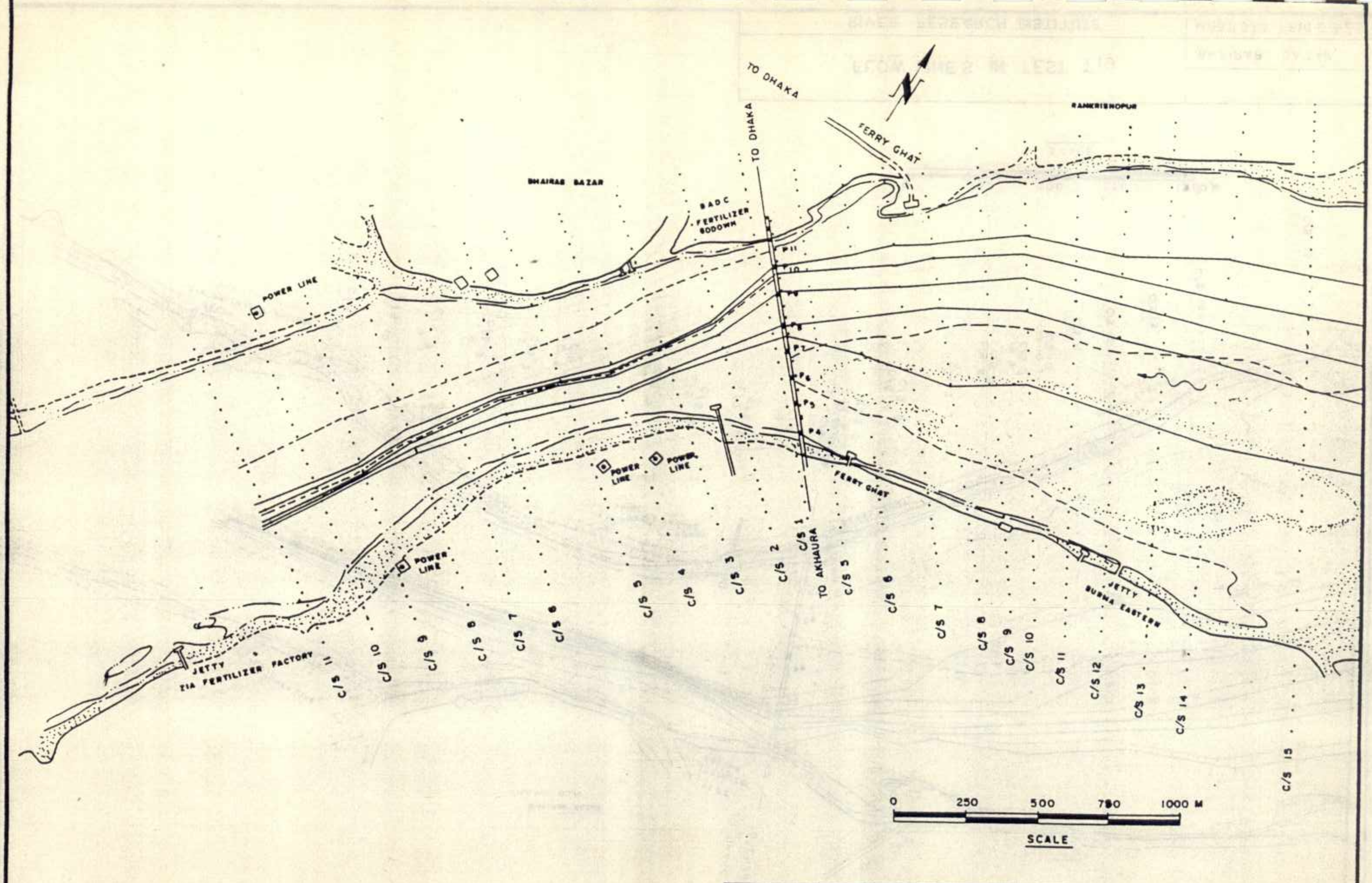


SURVEY BY BWDB, MORPHOLOGY.

FLOW LINES IN TEST T_{12}	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MRBPSTS | FIG. D.3.22

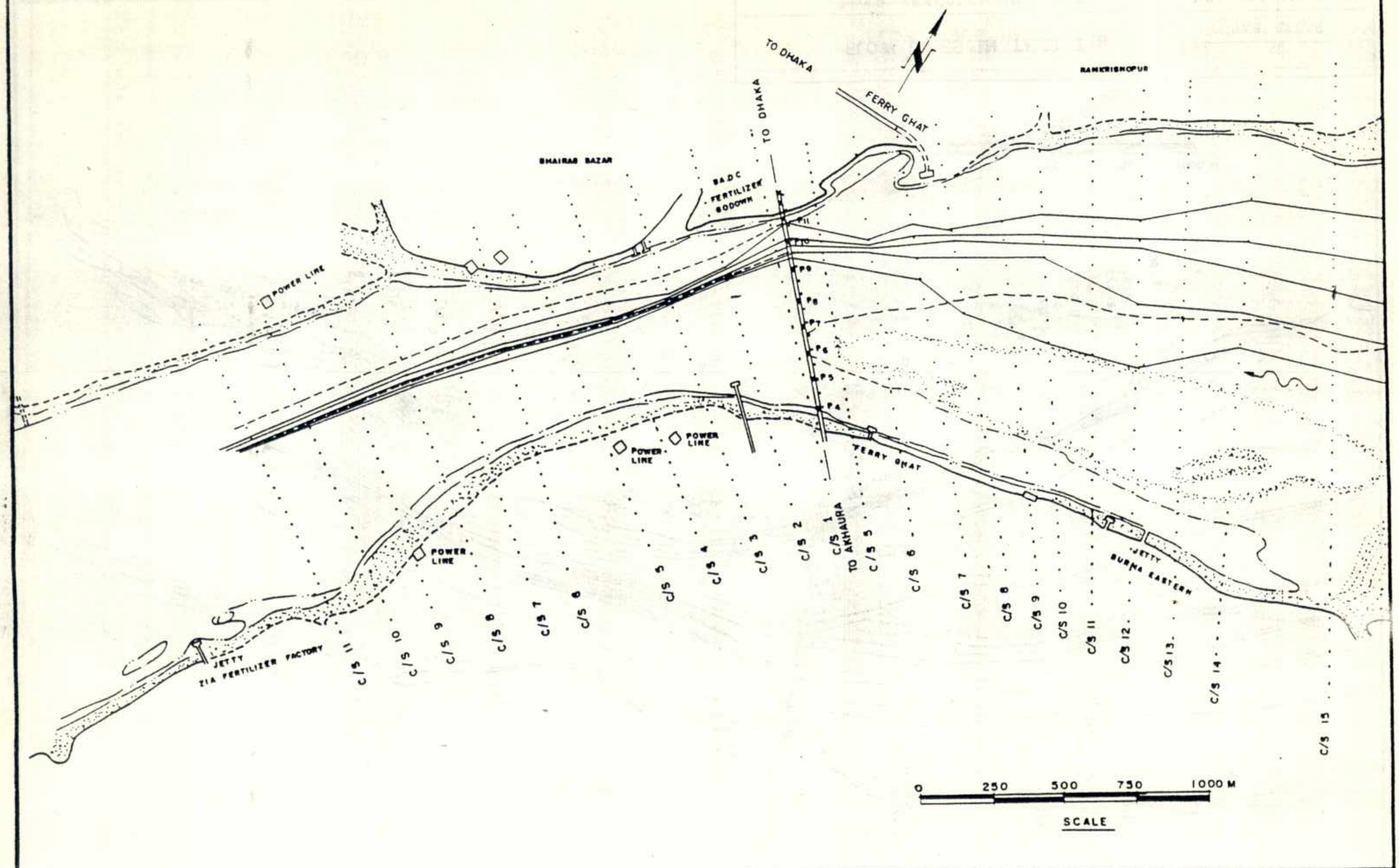


FLOW LINES IN TEST T_{16}		
	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG D 3.23



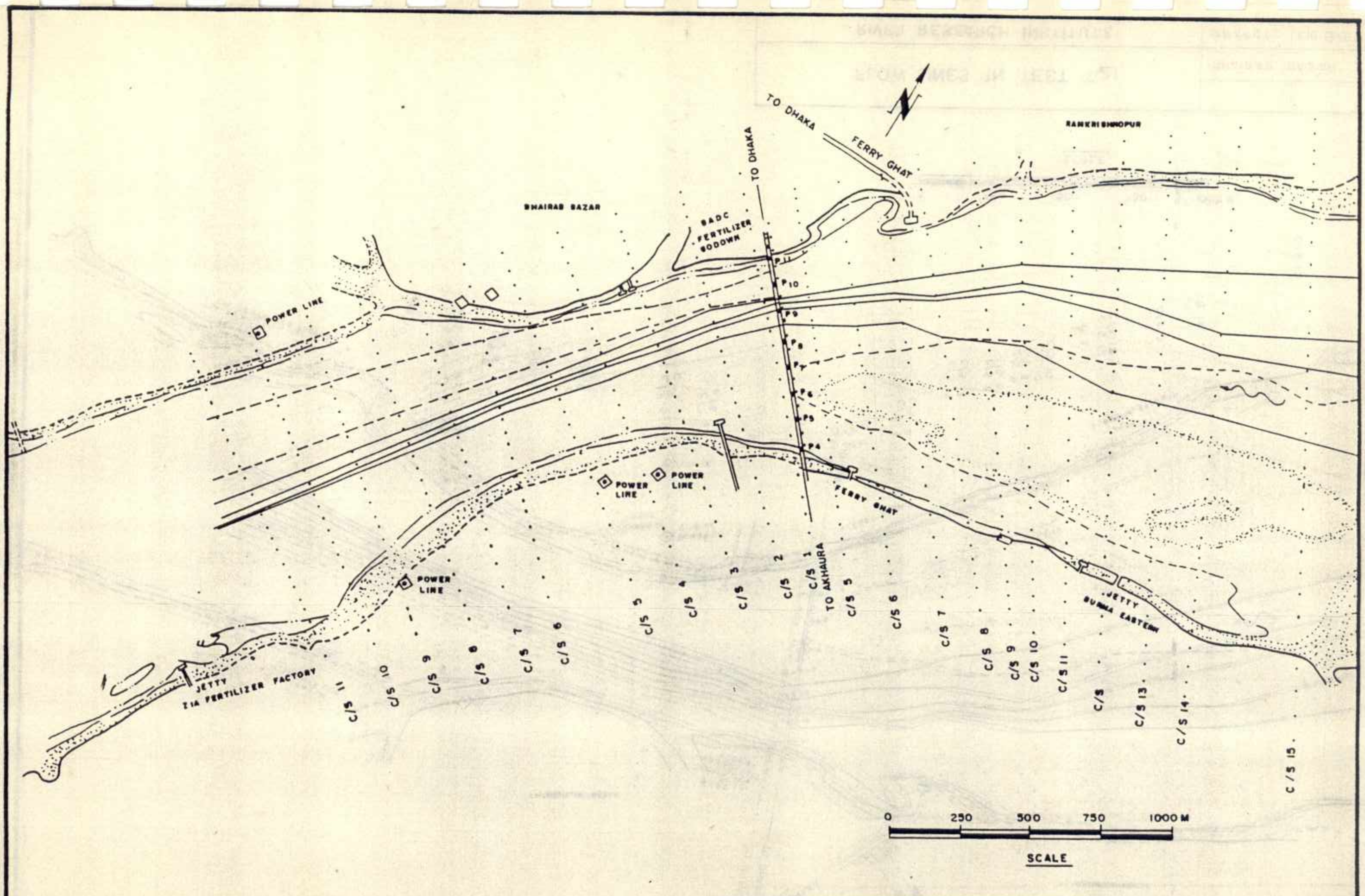
FLOW LINES IN TEST T18	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D.3.24

96

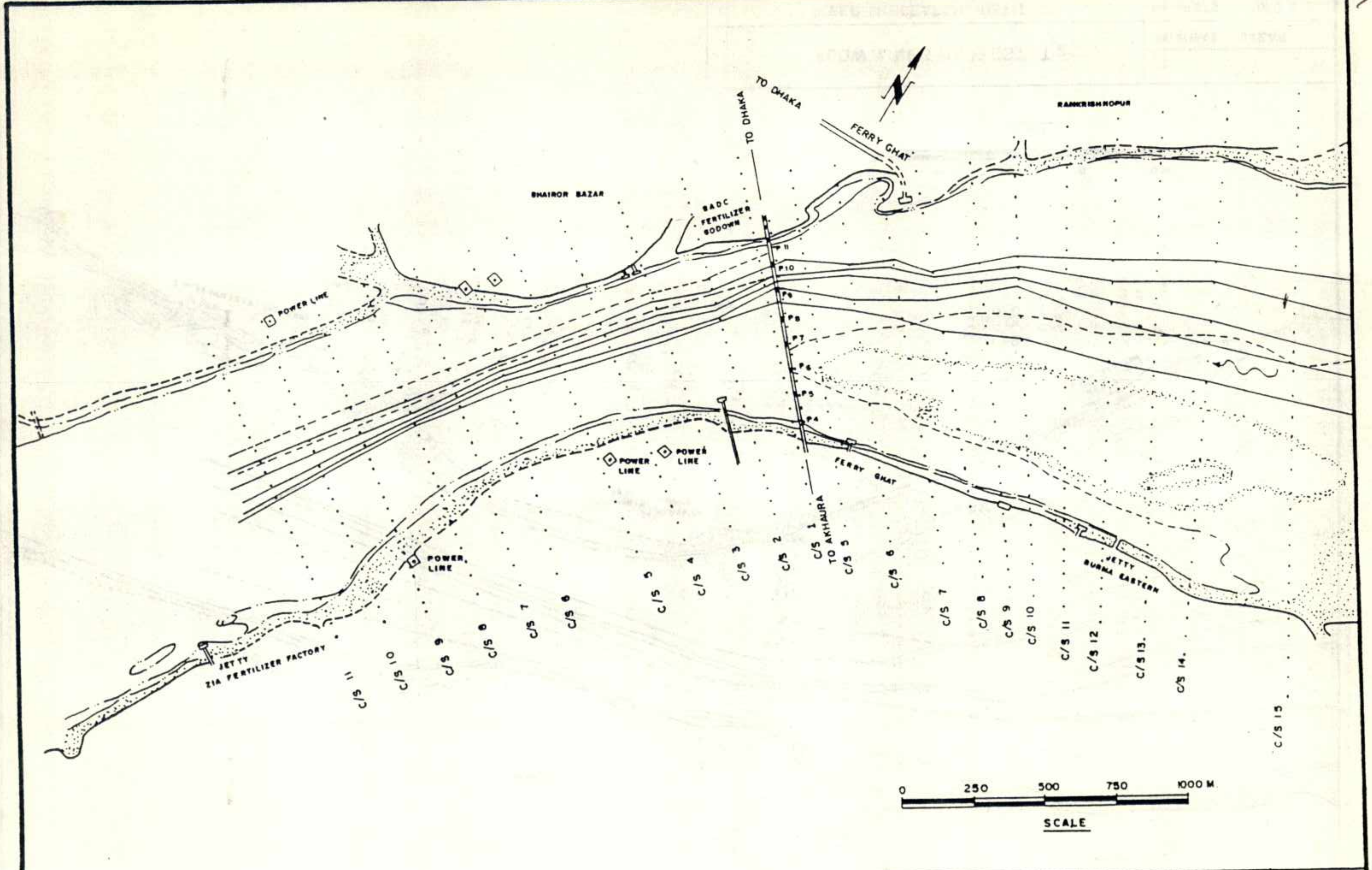


FLOW LINES IN TEST T19	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBP STS	FIG D.3.25

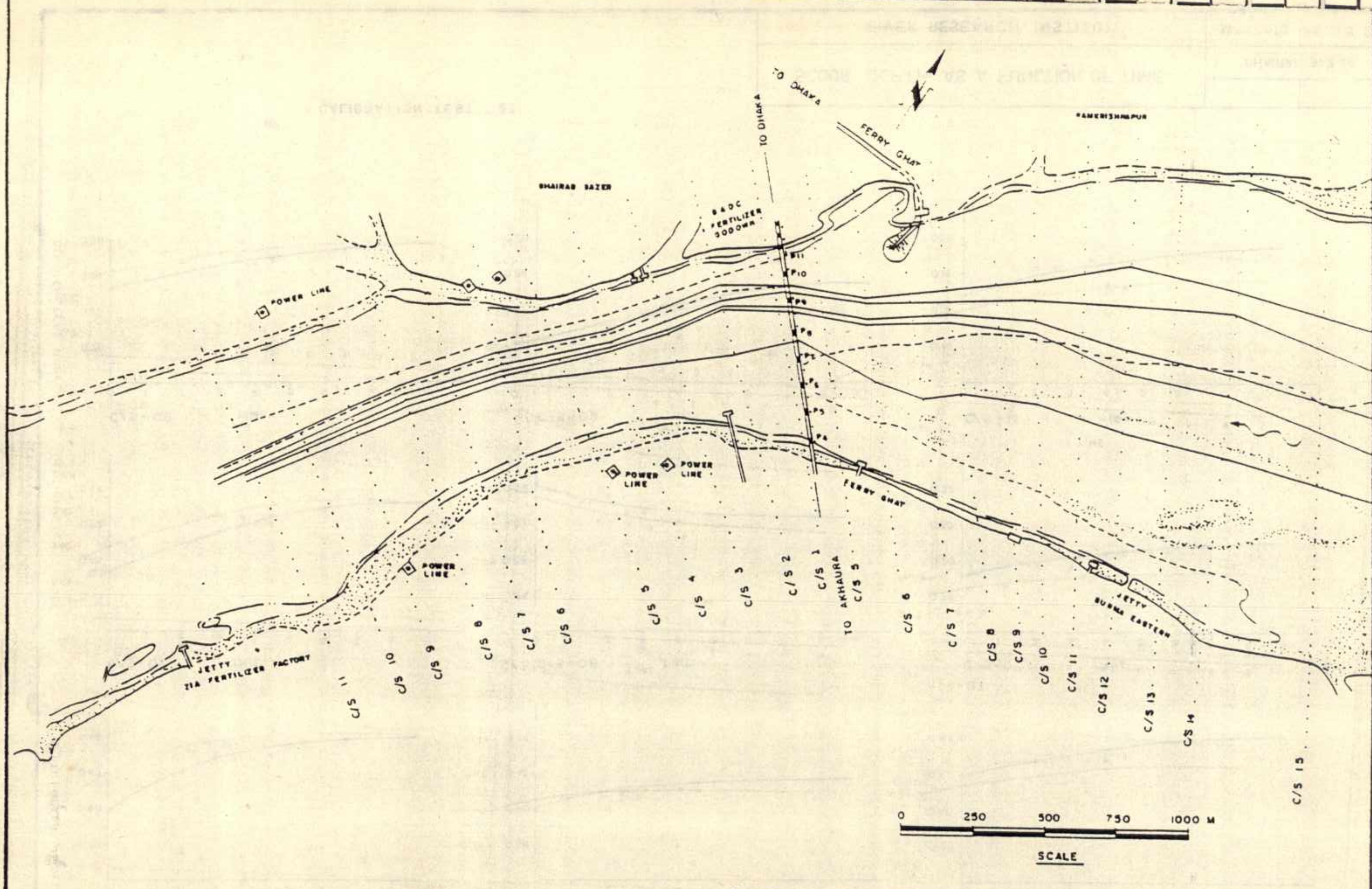
98



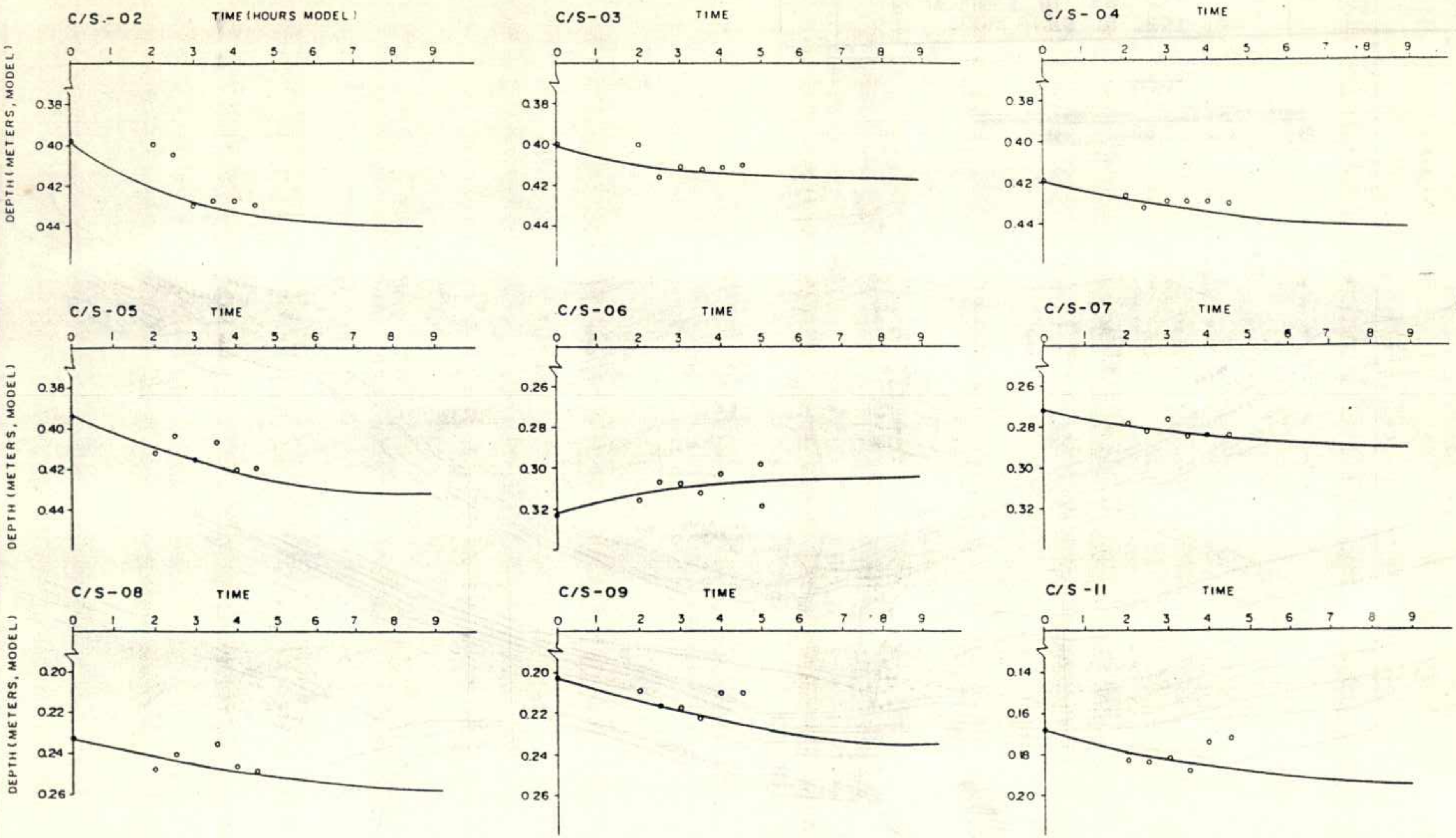
FLOW LINES IN TEST T 20	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D.3.26



FLOW LINES IN TEST T21	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D.3.27

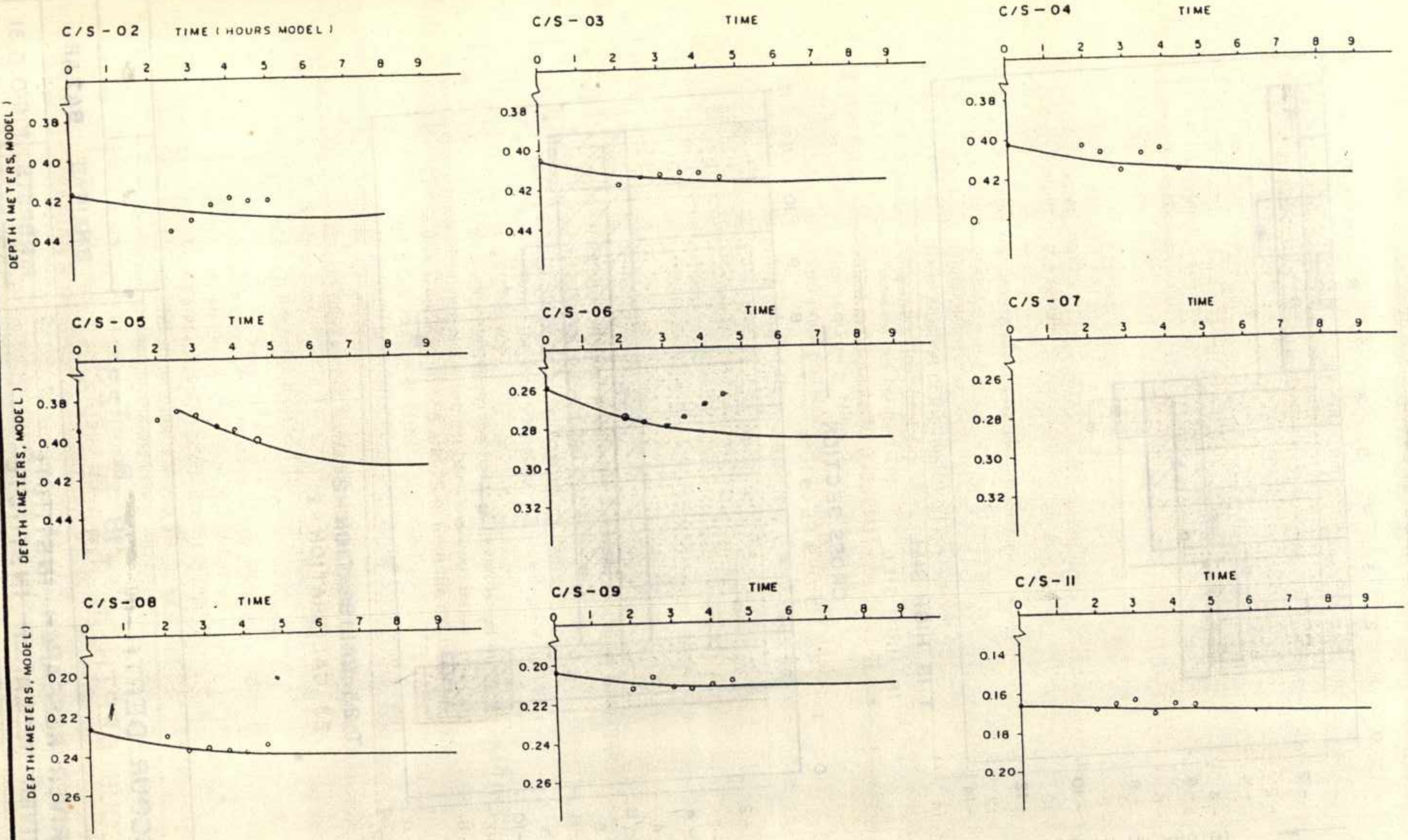


FLOW LINES IN TEST T_{22}	BHAIRAB BAZAR
RIVER RESEARCH INSTITUTE	MRBPSTS FIG. D.3.28



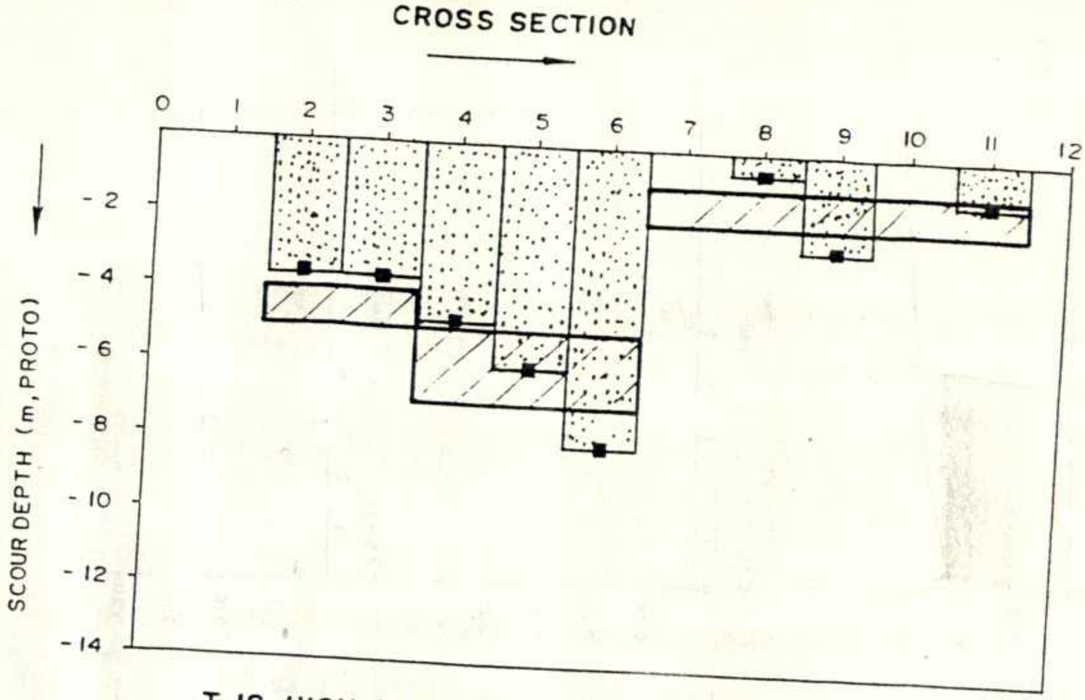
CALIBRATION TEST T23

SCOUR DEPTH AS A FUNCTION OF TIME	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D.3.29

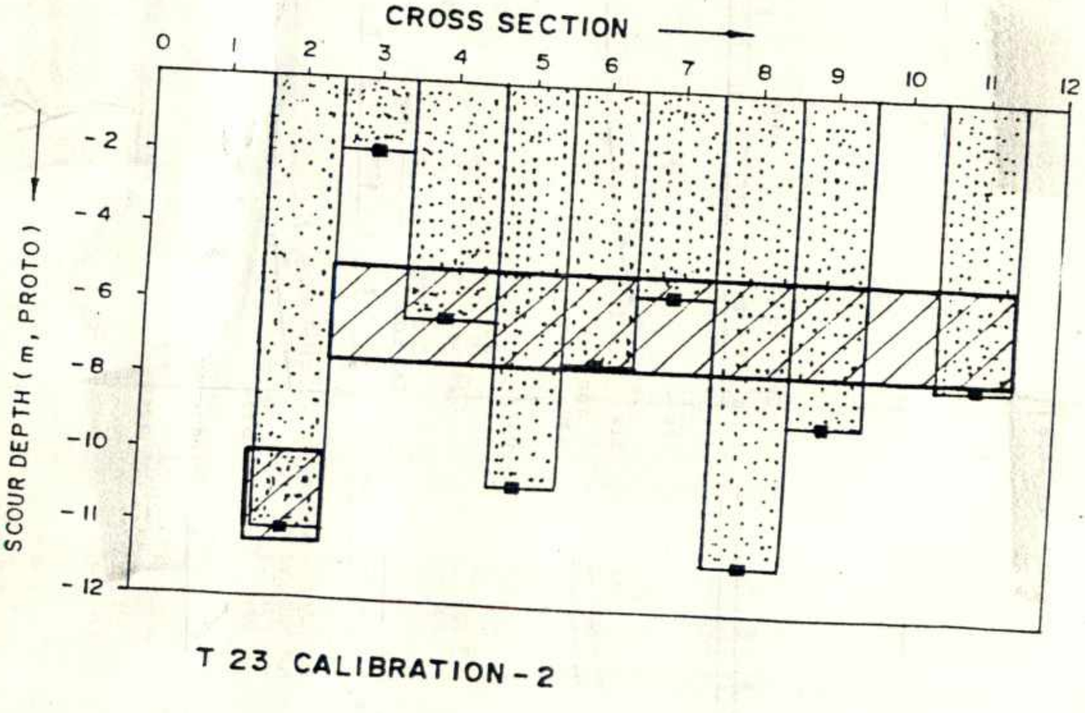


T18 HIGH SILL IN RAILWAY BRIDGE

SCOUR DEPTH AS A FUNCTION OF TIME	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG.D.3.30



T 18 HIGH SILL



T 23 CALIBRATION - 2

MAX. SCOUR DEPTH IN T_{18} & T_{23}		
	BHAIRAB BAZAR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG-D. 3. 31

10

D.4 CHANDPUR MODEL

Contents

TABLES

D.4 CHANDPUR MODEL

D.4.1 Introduction

Within the frame work of the Meghna River Bank Protection Short-Term Study a scale model has been constructed of a part of the Lower Meghna River near Chandpur, a town at the left bank of this river, see Figure 4.1. This scale model has been used to study the existing bank erosion problems along the Lower Meghna River and to test different short-term solutions. Short-term solutions are permanent solutions, which are a part of a long term strategic plan to prevent further bank erosion along the left bank of the Lower Meghna River from Eklashpur to Haimchar. The designed protection of Chandpur is:

(i) at emergency level, to be implemented within about 1 year,
(ii) at medium level, to be implemented within a few years, and
(iii) at final level, to be implemented after 8 to 10 years according to the actual planning.

More specifically, the purpose of the model investigation is to study the present conditions at Chandpur. Chandpur town is divided by the Dakatia River into Puran Bazar south of this river and in Nutan Bazar on the northern bank of this river, (see map in Figure 4.2). The model tests have been used to investigate the causes of both the excessive scour observed in front of the bank protection and of the eddies causing additional bank erosion. Also in the model possible short-term measures to remedy these problems, have been studied (see Table D.4.1).

TABLE D.4.1 THE MEASURES WHICH ARE TESTED IN THE SCALE MODEL FOR CHANDPUR

number	description of the alternatives
1	Strengthening of the existing revetments in Nutan Bazar and Jamuna Oil Terminal with emergency designs, variant 2.
2	A groyne upstream of Chandpur to divert the main flow from Nutan Bazar.
3	An advanced protection or guide bund around Nutan Bazar, which can be extended to include Puran Bazar also.
4	Two groynes perpendicular to the bank at Nutan Bazar to divert the flow locally from the bank.

A groyne upstream of Chandpur would be part of the strategic plan to strengthen the complete left bank of the Lower Meghna River between Eklashpur and Chandpur. The two groynes perpendicular to the bank at Nutan Bazar can be considered as a possible first phase of the construction of a complete guide bund with an "advanced" (as opposed to retired) protection around Nutan Bazar.

From this model investigation the following aspects are treated in this Chapter: the prototype data, which have been used in the scale model (Section D.4.2), the model set-up (Section D.4.3), a description of the test program (Section D.4.4), measurements (Section D.4.5), the interpretation of the results, including flow velocities, flow lines (Section D.4.6) and scour depths (Section D.4.7). A summary with the main conclusions can be found in Section D.4.8.

D.4.2 Prototype data

The data for the river bed geometry in the model have been obtained from the bathymetric survey in February 1991. The water level and the total river discharge have been determined as a function of the recurrence interval from the Hydrological Study (Annex A) of this project. Some preliminary data of the flood season survey in August and the beginning of September 1991 have been used for the calibration of the flow velocities.

The discharge of the Lower Meghna River varies roughly between 10,000 and 130,000 m3/s, whereby about 60,000 m3/s corresponds to bankfull discharge. In Chandpur the Dakatia River flows into the Lower Meghna River. Sufficient discharge data of the Dakatia at Chandpur are not available; therefore the observed flow field in the model has been used to estimate the model discharge of the Dakatia river.

Some characteristic discharges and water levels at Chandpur are presented in Table D.4.2, these data are obtained from the Hydrological Study (Annex B) of this project.

TABLE D.4.2 CHARACTERISTIC WATER LEVELS AND DISCHARGES

Description (m + P.W.D.)	bankfull discharge	flood level discharge (m3/s)
	~4.50	60,000
1 in 5 years	4.97	....
1 in 25 years	5.20	120,000
1 in 100 years	5.40	130,000

The average slope of the river is around 5 cm per km. Some tidal effects are observed in Chandpur having a tidal range of about maximum 1.0 m during the flood season and about maximum 1.6 m during the season with low discharges. These tidal ranges also have some influence on the local flow field around Chandpur. The bed roughness of the Lower Meghna River has been studied in the mathematical, morphological and the hydrological studies of this project. It turned out that no detailed data are available about the bed roughness at this part of the river system.

The bed material in the Lower Meghna River is more or less uniform sand with some silt and a trace of mica. The characteristic diameters are determined as the average of about 7 samples, which have been taken during the survey in February 1991: D_{50} = about 0.09 mm, D_{10} = 0.015 mm, D_{90} = 0.20 mm.

The representative cross sections of the bathymetric survey of February 1991 have been used to model the river bed. The area, which has been represented in the model is about 5 km long and 1.5 km wide, this means a partial reproduction of the total width of the Lower Meghna River only, see Figure 4.2.

D.4.3 Model set-up

The planform of the Lower Meghna River is characterised by a fairly straight river alignment with a braided channel and char pattern, the height of the banks in Chandpur ranging from about 5 to 6 m + P.W.D.

For the verification of the model some results of the survey in February 1991 have been used. During this field survey the discharge was low, therefore also the measured flow velocities were relatively low, and only the flow directions have been used for the calibration of the scale model. For the calibration of the flow velocities the results of the mathematical model and the flood survey in August and September 1991 have been used.

The scale model has one main inflow boundary and a small inflow boundary representing the flow over the char along the left bank upstream of Chandpur, see Figure 4.2, a small inflow from the Dakatia river and one outflow boundary.

In the steady flow model of Chandpur a constant discharge has been pumped around. The influence of the tidal fluctuations on the scour process has been neglected, as is usual in this type of models, because of the conflicting time scales of the tidal flow and the local scour process. The available head difference between the inflow and the outflow of the model is too small for a flexible operation of the model. Since in the model the initiation of motion of the bed material is observed locally, no facilities for the circulation of the sediment are required.

The flow field in the model is guided largely by the banks and the model limits. Therefore a local deviation from the roughness condition for the reproduction of a flow field can be accepted. The discharge distribution along the upstream and the downstream flow boundaries of the scale model have been derived from the mathematical model study.

The bed material of the scale model has the following characteristic diameters: D_{50} = 0.18 mm, D_{10} = 0.15 mm and D_{90} = 0.29 mm. For a local scour model it is rather particular that the grain size distribution of the model sand is coarser than the prototype sand (see Section D.4.2, prototype data). The scale laws require that the size of the model sand should be finer than the prototype sand. This scale is compensated by increasing the model discharge.

The grain size distribution of the sand in the model has been determined from only one sample, see Figure 4.3.

D.4.4 **Test program**

Different alternative solutions for the bank erosion in Chandpur have been tested in the scale model. Apart from the tests for these alternative solutions the test program includes 3 calibration tests with different model discharges.

The background of these calibration tests can be found in Appendix D/1 Scaling Procedures.

Before the removal of the Jamuna Oil Terminal and the severe erosion in Nutan Bazar the strengthening of the existing bank protection with emergency designs, variant 2 has been tested in test T4, see Figure 4.4 (designs at Nutan Bazar) and Figure 4.5 (designs at Puran Bazar). The inflow boundary was not correct in that test. For the alternative measure with a groyne upstream of Chandpur three tests were executed, and the most accurate one, T9, is analyzed here. The lay-out, a cross section and a longitudinal section of this groyne, which is connected with a sand dam to the existing bank, are given in Figures 4.6(a) and 4.6(b).

The alternative with an advanced protection in the shape of a guide bund around Nutan Bazar and Puran Bazar has been tested in test T8, see the lay-out in Figure 4.7. A typical cross section of this advanced protection is presented in Figure 4.8 and as a detail also a cross section of the mouth of the Dakatia river in Figure 4.9. This alternative has been split up in a guide bund around Nutan Bazar only, test T10, see the lay-out in Figure 4.10, in a guide bund around Puran Bazar only, test T11, see the lay-out in Figure 4.11 and as an intermediate construction stage a hard point around Nutan Bazar, test T13, see the lay out in Figure 4.12. In test T11 Nutan Bazar has been supposed to be eroded almost completely, in that case the left bank of the Dakatia River at Puran Bazar needs also to be protected against the flow from the Lower Meghna River, see the lay-out in Figure 4.13. the water depth in the eroded part of Nutan Bazar is assumed to be 10 m-P.W.D.

TABLE D.4.3 TEST PROGRAM FOR PROTECTION WORKS AT CHANDPUR

Test number	Description of the test
1 *	Calibration test with discharge according the Froude law
2 *	Calibration test with the discharge higher than in test T1
3 *	Calibration test with the discharge higher than in test T2
4 *	Emergency designs, variant 2
5 *	Upstream groyne, direction +13 degrees
6 *	Upstream groyne, direction - 7 degrees
7	Repetition of T2 with correct inflow boundary
8	Guide bund around Nutan Bazar and Puran Bazar
9	Upstream groyne, direction -7 degrees
10	Guide bund for Nutan Bazar only
11	Guide bund for Puran Bazar only
12	Two short groynes at Nutan Bazar
13	Hard point protection Nutan Bazar

* test with a not correct inflow boundary

The tests without an asterisk in the table above have been analyzed in this report. In all these tests the discharge, the water levels, the flow velocities and the scour depths have been measured.

D.4.5 **Measurements**

D.4.5.1 General

In each test the discharge, the water level, the flow velocities, the flow lines and the scour depths were measured. The water temperature was measured during each test but the photographs of the flow field were made only a few times, because the temporary shed did not allow to take such photographs with a good quality. No photographs without oblique have been taken of the dry scour hole with contour lines after a test was finished and the model was drained. Some of the photographs of the dry model bed have been presented in this Annex, see Photographs D.4.1 - D.4.4.

D.4.5.2 Discharge

The model discharge has been calculated from the water level upstream of a Rehbock-weir (rectangular weir with a sharp crest with a brass strip, however in the model corrugated weirs were applied with a irregular crest). The upstream water level has been substituted in the standard formula for these type of weirs to determine the model discharge, provided the upstream flow velocity can be neglected:

$$Q = 1.86 \cdot b \cdot h_w^{1.5}$$

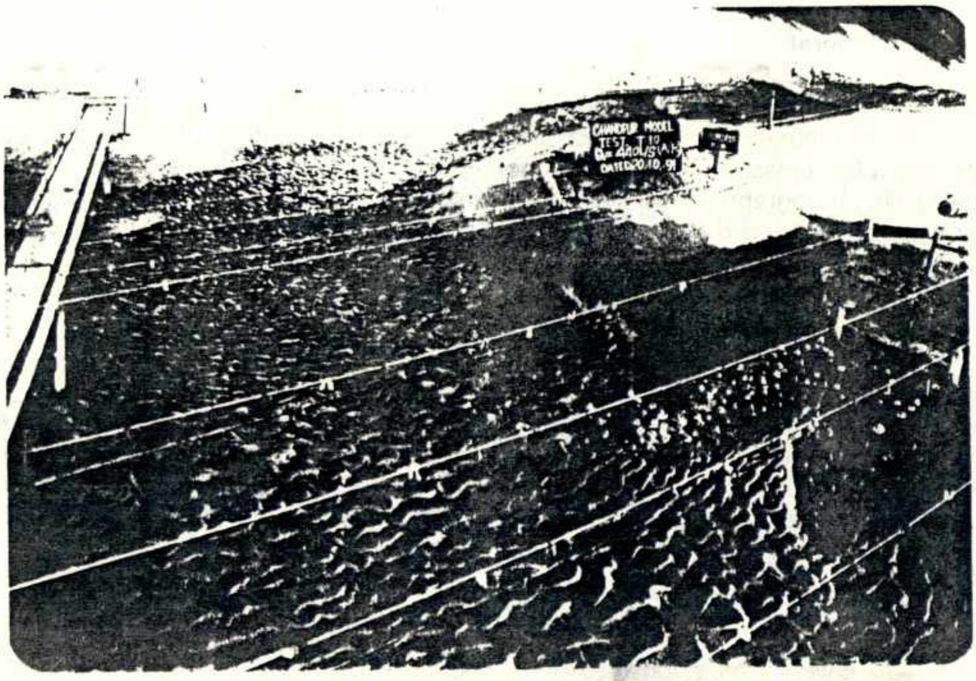
in which Q = discharge (m^3/s), b = width of the weir (m), in this weir b = 1.81 m and h_w = vertical distance between the crest of the weir and the upstream water level (m).
However, in the measuring weir in the model the upstream flow velocity can not be neglected and, consequently, a more detailed formula has to be applied:

$$Q = (0.407 + 0.053 \cdot \frac{h_w}{p}).\sqrt{2.g}.b.[(h_w + \frac{u_w}{2.g})^{1.5} - (\frac{u_w}{2.g})^{1.5}]$$

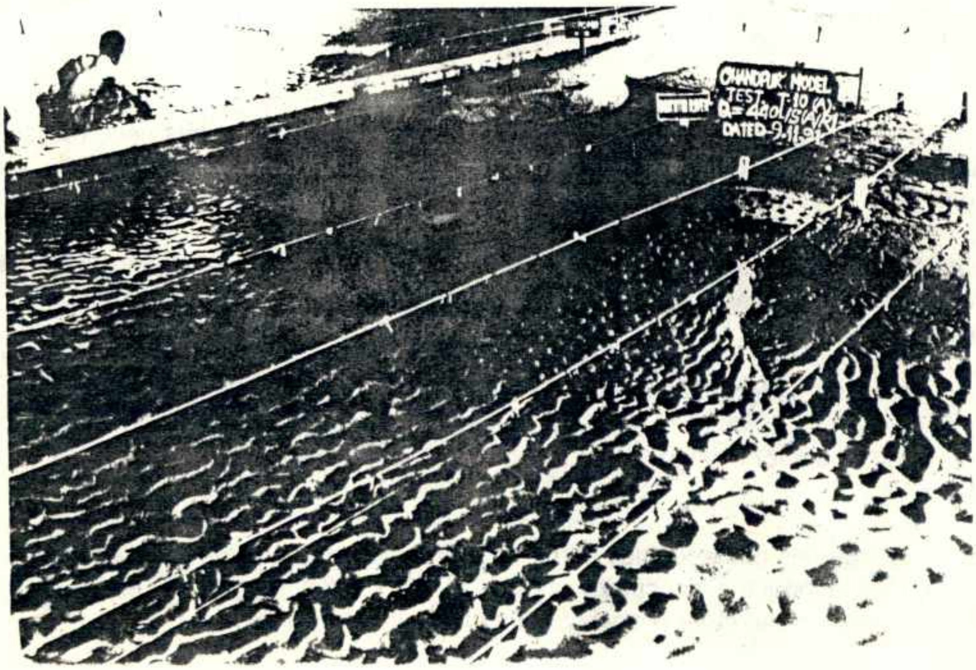
in which u_w = the average flow velocity in the cross section in which h_w has been measured (m/s), p = the height of the crest of the weir above the bottom (m), in this weir p = 0.31 m and g = acceleration by gravity (m/s^2).

No calibration formula has been determined for each weir separately as was required in the Terms of Reference for the model investigation in RRI. The supply of air under the falling jet was not guaranteed as required for a standard weir. The upstream water level showed some fluctuations up to 0.002 m, see Figure 4.17. Therefore the model discharge should be estimated as the time averaged discharge. Because, in that case the influence of these fluctuations on the determined discharge can be reduced to about 1%. It is noted that these fluctuations depend on the model discharge. The inflow structure has probably been designed for discharges up to 0.3 m^3/s and in these tests the discharge varied between 0.4 and 0.5 m^3/s.

One of the other sources of inaccuracy in the determination of the discharge is the corroded crest of the weir, which could induce a inaccuracy of 5 to 7 % if the discharge is low. The final accuracy of the determined discharge is therefore probably not as high as required.

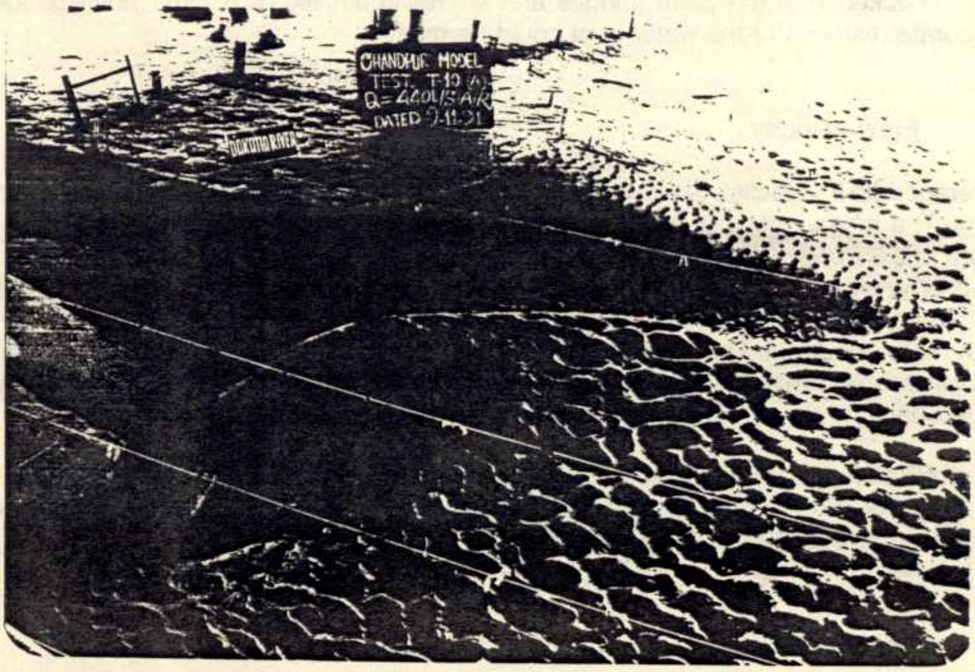


4.1 Dry model bed after test T10 advanced protection in Nutan Bazar

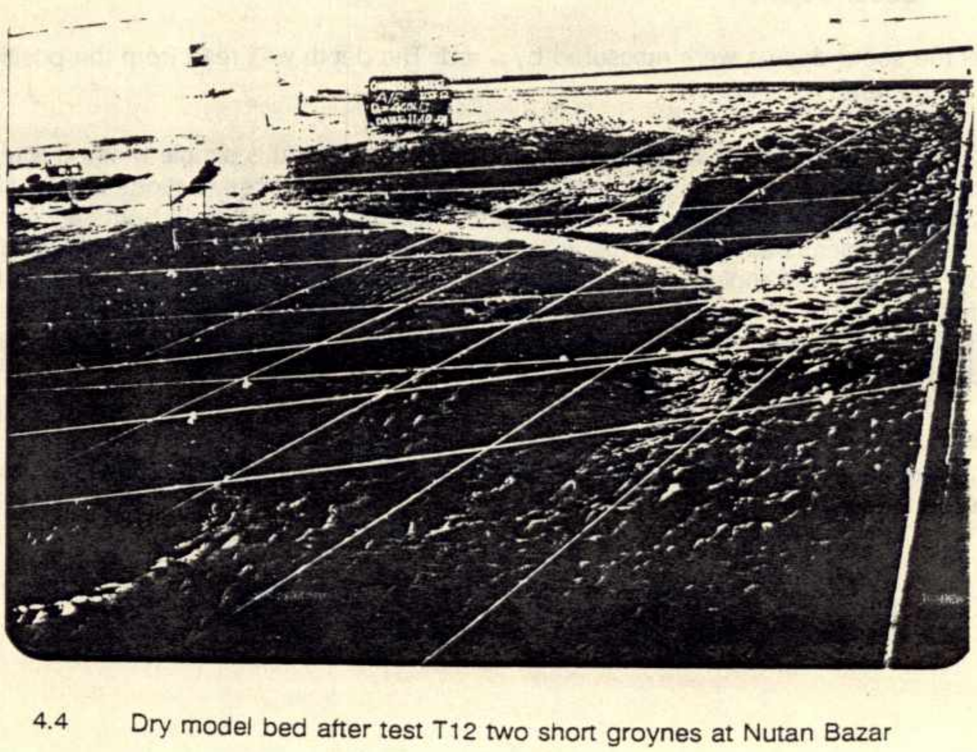


4.2 Dry model bed after test T13 hard point protection in Nutan Bazar

63



4.3 Dry model bed after test T13 hard point protection in Nutan Bazar



4.4 Dry model bed after test T12 two short groynes at Nutan Bazar

D.4.5.3 Water level

In the model the water level can be regulated by the tailgates. After the calibration test the position of the gates should be kept constant during the tests. The positions of each of the five tailgates is only important for the distribution of the discharge in a cross section just upstream of the weirs. The average water level is checked with the point gauges and is well controlled. From the point gauges (if not blocked) accurate readings of the water level could be made.

D.4.5.4 Flow velocity

The time averaged flow velocities have been measured by an Ott flow propeller having a diameter of about 0.03 m. The measurement time for counting the revolutions of the propeller has been reduced from 1 minute to 0.5 minute, and this has resulted in an accuracy of + or - 0.025 m/s in the prototype values. The flow velocities have been measured in cross sections 10, 17, 22, 24, 26, 29, 31, 32, 35, 42 and 50, see Figure 4.18. In these cross sections the measuring points were selected at an interval of 75 m and at half the water depth. For more detailed information about the flow velocities near the left bank the flow velocities were measured in cross sections 21, 22, 23, 24, 26, 28, 29, 30 and 31 from 0 to 350 m from the left bank, and at an interval of 37.5 m, see Figure 4.19.

In the Terms of Reference for this model investigation at RRI a calibrated flow measuring instrument, which also could measure the turbulent fluctuations, was required. The Ott flow propeller was composed of parts of different Ott propellers and the calibration was not repeated.

D.4.5.5 Flow direction

The flow direction has been measured with flow lines, which have been determined by floats. These floats have been made from small bottles, which float with a depth of approximately 0.04 m in the water. The position of the float was determined at the moment the float passes the line of a cross section. The accuracy of this method to locate the float was about 0.05 m.

D.4.5.6 Scour depths

In the model the scour depths were measured by a rod. The depth was read from the position of the rod relative to the reference line.

This line was also used to fix the location relative to the bank. With this simple method a reasonable accuracy could be maintained. The scour depths were measured in cross sections 22, 24, 26, 28, 29, 30, 31 and 32, see Figure 4.36. and at an interval of 0.5 hour between 4 and 16 hours after the start of the test. From these measurements and the initial bed level the equilibrium maximum scour depth was estimated by graphical extrapolation. This procedure was applied to the deepest point in the cross section of the initial scour hole and to the point in that cross section, where the maximum increase in scour depth was determined. The procedure was followed because the location of the deepest point in a cross section can shift during a test.

As an additional measurement the temperature of the water has been measured in each test. The average temperature was about 31 degrees Celsius, the minimum 29.5 degrees and the maximum 32 degrees Celsius.

D.4.6 Interpretation of the results

D.4.6.1 Introduction

All tests have been done in the actual bed geometry and flow pattern based on the data of the field survey of February 1991. For design purposes the results of these tests have been analyzed elsewhere (see Annex B) together with other information about the morphological processes in the Lower Meghna River in order to obtain the design parameters (flow velocities and scour depths).

The model discharge has been selected on the condition that the flow velocity is high enough to create a scour hole at the depth scale, and on the condition that the deviations between the flow field according to the Froude scale and the flow field in a test with relatively high discharge are small.

The initial model bed has been moulded by interpolating between the head of bamboo picks, which were placed in the cross sections. The height of the picks has been determined from the bathymetric map. After each test the height of the picks was checked and the average deviation was between 1 and 2 mm. Therefore it is concluded that the reproduction of the initial model bed is sufficiently accurate.

The following results are discussed in this section: the hydraulic gradient and the model roughness in Section D.4.6.2, the flow field, maximum flow velocities and the flow lines in Section D.4.6.3 and the main aspect of the scouring process, the equilibrium maximum scour depth, in Section D.4.6.4.

D.4.6.2 Model roughness and hydraulic gradient

For the preparation of the test program the gradient, i_w, in the water level has been estimated at (see Appendix D/1):

i_w = 0.00003 with a model discharge of 0.225 m^3/s,
i_w = 0.00007 with a model discharge of 0.410 m^3/s,
i_w = 0.00005 with a model discharge of 0.510 m^3/s.

In the Chandpur model the flow field can be schematized as a wide upstream part followed by a more narrow middle part near Nutan Bazar, where the flow accelerates, and an outflow section in which the flow again decelerates. Therefore the schematization to a uniform flow to apply the Chezy equation is rather crude. For a more detailed analysis a simple 1 dimensional flow routing model is recommended.

The main tests have been conducted with a discharge of 0.410 m3/s, 0.440 and 0.450 m3/s respectively, and from the Chezy equation the following values of the roughness coefficients have been calculated:

discharge :	0.410	0.440 m^3/s
hydraulic radius:	0.22	0.22 m
average velocity:	0.24	0.26 m/s
average gradient:	0.00007	0.00008
Chezy coefficient:	61	62 $m^{0.5}s^{-1}$

These calculated values of the Chezy coefficient are quite similar to the values assumed during the design of the model (see Appendix D/1). This should be interpreted as a tentative conclusion if one considers the rather crude schematization.

D.4.6.3 Flow velocities

D.4.6.3.1 General

In all situations the analysis of the flow velocities has been concentrated on the maximum flow velocities along the banks, because these flow velocities are the important parameter for the maximum hydraulic load on the bank.

It is emphasized that if the flow velocities in different tests are to be compared, than the model discharges should be the same in these tests, because the flow velocities are proportional to the model discharge.

The following aspects are discussed below: the reproduction of the measured flow velocities, the flow velocities in the calibration test, the maximum flow velocities in the tests with a upstream groyne, a guide bund and the two short groynes at Nutan Bazar.

D.4.6.3.2 Reproduction of the measured flow velocity

If in a steady flow the flow velocities are measured two times at the same location, then the differences between these flow velocities vary between 1.5 and 3.5 % of the flow velocity.
In the calibration test T2 the differences in the measured velocities in cross sections 24 and 29 are between 0 and 0.2 m/s, In the test with the complete guide bund in the same cross sections these differences are 0.04 to 0.3 m/s, and in the test with the upstream groyne in the same cross sections these differences are 0 to 0.1 m/s, see Table D.4.12. This information concerns the reproduction of the flow velocities in one test, the reproduction of the flow velocities in different tests shows a wider range in these differences.

It is should be pointed out that the aforementioned differences are caused by at random fluctuations in the flow field. These fluctuations are caused partially by the turbulence in the flow, and possible variations in the discharge by the pump and the fluctuations in the voltage of the electricity. Another source is the morphological variation in the local bed level during the time elapsing between the two measurements.

The measured flow velocity can be influenced, if the position of the propeller meter makes an angle with the main local flow direction, or if the propeller is not exactly at the correct position in the vertical. These are not-systematic errors. However, also systematic errors exist, for example if the calibration curve of the instrument is not reproduced with sufficient accuracy by the instrument. It is recommended in this respect to repeat the calibration of the Ott-current meters.

D.4.6.3.3 Calibration test (test T2):

The distribution of the flow velocities along the upstream boundary (cross section 42) of the mathematical model has been reproduced accurately in the model by adjusting the brick stones at the model inflow section.

As a next step the flow lines of the mathematical model have been compared with the measured flow lines in the model, see Figures 4.20 and 4.21. Due to the relatively high bed roughness the radius of the curvature of the flow lines is too small in the model. This can not be avoided in this type of local scour model and as a part of the interpretation the consequences of this scale effect on the main results and conclusions of the model should be estimated. In this model these consequences are estimated to be small.

Characteristic flow velocities along the existing bank of Nutan Bazar and Puran Bazar are given below:

TABLE D.4.4 COMPARISON OF THE FLOW VELOCITIES IN THE CALIBRATION TEST AND IN THE FLOOD SURVEY

cross section number	calibration test distance from left bank		flood survey distance from left bank	
	37.5 m m/s	75 m m/s	37.5 m m/s	75 m m/s
32		1.2		
31	2.6	2.7		
30	0.0	3.3		
29	2.8	3.2 to 3.3	2.6	3.1
26	0.8	2.3		2.1
24	1.5	1.6 to 1.8		
23	0.0	0.0		1.0
22	0.0	0.0		
21	0.0	0.0		
19				
17		0.0		0.4

From Table D.4.4 it can be concluded that in the calibration test the maximum flow velocities along the bank are almost the same as the maximum flow velocities which have been measured during the flood survey. In the scale model downstream of Puran Bazar an eddy with very low flow velocities existed. Due to the scale effects in the Reynolds-number these flow velocities are too low in the model and the Ott propeller can not measure these velocities properly, because they are outside the calibration range of this instrument. However, these low velocities are not important for the overall flow field and for the design of bank protection works, and therefore these deviations in the model can be accepted.

It is noted that the direction of the maximum flow velocities in the prototype does not coincide with the direction of the maximum flow velocities in the scale model, probably because of tidal effects.

From the tests T1, T2 and T3 the model discharge has been selected. In T1 the flow velocities have been scaled as per Froude condition to reproduce the flow field accurately. However in that test the flow did not have sufficient capacity to erode the bed. Therefore, the model discharge has been increased and the maximum possible discharge has been selected. Subsequently, it was checked whether the flow field changes with an increase in model discharge, (see flow lines of T1, T2 and T3 in Figures 4.22, 4.23 and 4.24). In these tests the discharge distribution was wrong, but in all these tests in the same way. By introducing the increased capacity in order to erode the model bed also bed forms were generated. This resulted in a relatively too rough model bed and in a deviation in the curvature of the flow lines. This local scale effect has been accepted and has no consequences for the main results of this study.

D.4.6.3.4 Upstream groyne (test T9):

The maximum flow velocity near the head of the groyne is 3.9 m/s just near the flow separation point. Along the upstream side of the shaft of the groyne the maximum flow velocities are 2 to 2.5 m/s for the first 950 m and 2.5 to 3 m/s for the most exposed 600 m of the shaft.

The main effect of this groyne on the flow field near Nutan and Puran Bazar is a modest reduction of the flow velocities near the bank: in cross section 29 at Nutan Bazar about 20 % up to 400 m from the left bank, gradually changing up to cross section 17 downstream of Puran Bazar to about 10 % from the bank line to 600 m from the bank, see Figure 4.25. Along the bank of Nutan Bazar the maximum flow velocity is reduced by 20 %, i.e. from 3.2 m/s to 2.6 m/s. Along the bank of Puran Bazar the maximum flow velocity is reduced by about 10 %, i.e. from 2.3 to 2.1 m/s at 75 m from the bank and from 3.05 to 2.9 m/s at 150 m from the bank (cross sections 24 and 26). The flow lines in this test are presented in Figure 4.26.

D.4.6.3.5 Guide bund (tests T8, T10, T11):

For the guide bund (i.e. the advanced protection) three tests have been executed:

- the complete guide bund (test T8)
- Nutan Bazar guide bund (test T10)
- Puran Bazar guide bund (test T11)

The maximum flow velocity for the bank protection along Nutan Bazar:

TABLE D.4.5 THE FLOW VELOCITIES ALONG THE BANK OF NUTAN BAZAR IN T8 AND T10

cross section -	T8 m/s	T10 m/s
29	1.9 to 3.4	2.8, 2.8
30	2.9	3.25
31	3.8	3.4
32	3.0	2.8

If it is planned to extend the guide bund in Nutan Bazar to a complete guide bund then the maximum flow velocity near the bank will be 3.8 m/s, otherwise 3.5 m/s.

If a guide bund is only constructed along Nutan Bazar, the bank of Puran Bazar near the mouth of the Dakatia River has to resist a maximum flow velocity of 1.5 m/s. More downstream the flow velocity is less than 0.9 m/s.

The maximum flow velocities at Puran Bazar are 2.0 m/s for the complete guide bund. For a guide bund only at Puran bazar and Nutan Bazar almost being completely eroded the maximum flow velocity is 3.45 m/s only near the mouth of the Dakatia River in the Meghna River. More downstream the maximum flow velocities will be < 0.7 m/s, see Figure 4.27.

The relatively small influence of the complete guide bund on the flow velocities far away from the bank is observed in Figure 4.28 with the flow velocity distribution in cross sections 17, 29 and 32.

TABLE D.4.6 FLOW VELOCITIES ALONG THE LEFT BANK AT PURAN BAZAR

cross section -	T8 m/s	T11 m/s
22	1.9	-0.3
23	1.7	-0.5
24	1.7	-0.2
26	1.2	3.4

In case of an advanced protection at Nutan Bazar the flow velocities along the bank at Puran Bazar are reduced as follows:

TABLE D.4.7 THE FLOW VELOCITIES AT 37.5 AND 75 M FROM THE LEFT BANK OF PURAN BAZAR IN T2 AND T10

cross section -	T2 m/s	T10 m/s
22	0.0	0.0
23	0.0	0.0
24	1.5 to 1.8	0.3 to 0.9
26	0.8 to 2.3	0.3 to 1.6

The guide bund at Nutan Bazar reduces the flow velocities by about 0.5 m/s to 0.9 m/s along the bank protection at Puran Bazar.

From the comparison of the flow lines in T8, see Figure 4.29 with the flow lines in T2 it follows that the flow in T8 is a little bit more concentrated along the left bank. Because of the influence of the too high bed roughness this tendency in the flowlines is not reliable. The flow velocity distribution in cross section 17 in tests T2 and T8 indicate that the main flow has been shifted about 100 m to the middle of the river and will have an effect also more downstream of Nutan and Puran Bazar. The flow lines in tests T10 and T11 are presented in Figures 4.30 and 4.31.

D.4.6.3.6 Two groynes at Nutan Bazar (test T12):

The maximum flow velocity near the head of groyne I is 1.9 m/s just downstream of the flow separation point. Along the shaft of that groyne a maximum flow velocity of 1.5 m/s is to be expected.

The maximum flow velocity near the head of groyne II is 3.75 m/s in the axis of the groyne, and 2.3 to 3.6 m/s just downstream of the flow separation point.

Between these two groynes the flow velocities vary between 0.3 m/s to about 1.4 m/s, and the maximum flow velocity along the existing bank upstream of groyne I is about 0.8 m/s.

Along the existing bank of Puran Bazar the maximum flow velocities at 40 m from the bank vary between 0.25 and 0.6 m/s in upstream direction (in the calibration test 0.8 to 1.6 m/s), because the flow along that bank from cross section 23 to 26 is part of a big eddy. Downstream of cross section 23 up to cross section 19 the flow velocities are < 0.25 m/s up to 300 m from the bank, because the flow in that area is also part of that eddy (in the calibration test it is also <0.25 m/s).

The flow velocities along the existing bank in the alternative with two groynes are reduced considerable, if these velocities are compared with the flow velocities in the calibration test.
Downstream of these groynes the flow velocities are also reduced up to 300 to 600 m from the bank as can be seen from the flow velocity distribution in cross sections 17, 24 and 29, see Figure 4.32.
The flow lines measured in this test have been presented in Figure 4.33.

From these flow measurements one tends to conclude that the length of these short groynes can be reduced somewhat.

D.4.6.3.7 Hard point protection at Nutan Bazar (test T13)

The measured maximum flow velocities along the left bank at a distance of 75 and 75 m from the bank have been presented in Table D.4.8. It is mentioned that the slope protection extends to about 130 m from the bank, if the waterdepth is about 40 m.

In cross section 32 a decisive maximum flow velocity of 3.3 m/s is measured.

TABLE D.4.8 THE MAXIMUM FLOW VELOCITIES ALONG THE LEFT BANK WITH A HARD POINT PROTECTION

cross section	distance from the bank	
number	37.5 m	75 m
32	...	2.8
31	2.0	3.3
29	2.1	2.6
28	1.0	2.1
26	1.2	2.0
25	2.0	2.7
24	1.6	1.7
22	0.3	0.3
21	0.3	0.4

The flow lines are presented in Figure 4.34.

2c

D.4.6.4 Scour

The scour depths in front of Nutan Bazar and Puran Bazar are important for the stability of the existing banks and for the design of the toe of the bank protection works in the alternative solutions. In addition the local scour around the head of the groynes in the various alternatives has been determined for the design of the toe level of the protection of the groyne head.

In some alternatives a falling apron has been designed for the lower part of the bank protection. In the model this falling apron has not been modelled, because the presence of bed forms, e.g. small ripples, disturbs the presence of a falling apron in the model. A falling apron will limit the slope gradient of the scour hole to approximately 1 in 2 and probably a falling apron will not reduce the maximum scour depths very much.

In a cross section, where the scour depths have been measured, the maximum increase of scour depth and the scour depths in the deepest point of that cross section have been given in Tables D.4.13 to D.4.18 (placed at end of this Chapter) for the calibration test T2 and the tests T9, ...,T12. In these Tables also the location of these points and the initial bed levels can be found.
In the calibration test T2 the development of the river bed can be determined in the actual situation when the discharge is increased up to the design discharge.

From Table D.4.13 the following can be concluded:

- The scour hole is deepened relatively uniformly by 4 to 7 m. The extra deepening by 8 m in cross section 32 can be explained as a tendency to extend the scour hole in upstream direction.

- The initial bed level has been measured during the bathymetric survey in February 1991. The scour depths are strongly related to the geometry at Chandpur and to the discharge distribution in the upstream channels. But also the scour depths depend to a certain extent on the specific discharge (or the flow velocity, see the formula of Ahmad).

- In the calibration test it has been determined that the flow velocities in the model are high enough to reproduce the local scour process. This means roughly that the average flow velocity is twice as large as the critical flow velocity, (see Appendix D/1). The measured flow velocities in the scour hole and just upstream of the scour hole are between 0.35 and 0.4 m/s. The calculated critical flow velocity is 0.16 to 0.19 m/s. This condition is just fulfilled and it is concluded that the maximum scour depths are reproduced rather accurately in the model.

The graphical extrapolation of the measured bed levels as a function of time made in order to estimate the equilibrium scour depths is illustrated with some examples of different tests in Figure 4.35.

From the aforementioned Tables the lowest bed level, which is decisive for the design of the toe, has been selected and these values are summarised in the Table below.

The shift in the point with the deepest scour in a cross section has been in the range of 200 to 500 m.

28

TABLE D.4.9 LOWEST EQUILIBRIUM BED LEVELS IN DIFFERENT CROSS SECTIONS AND IN DIFFERENT TESTS

test	cross sect. 22	cross sect. 24	cross sect. 26	cross sect. 28	cross sect. 29	cross sect. 30	cross sect. 31	cross sect. 32
-	m	m	m	m	m	m	m	m
T2	-37	-42 *		-53		-50	-55	-29
T8	-39.5	-49	-48	-58.4	-53	-47	-49.5	-50
T9	-43	-45	-47	-52	-53	-49	-53	-50
T10	-37.5	-42.5	-46	-56.5	-51.5	-52	-51.5	-45
T11	-36	-38	-43.5	-39		-41.8		
T12	-45.5	-48	-49	-45				

* all depths in meters below P.W.D.

From the foregoing Table it can be seen that the bed levels in the alternatives with a guide bund around (i.e. an advanced protection) Nutan Bazar, the complete guide bund, the upstream groyne and two groynes at Nutan Bazar are almost the same and these scour depths show a slight increase over the scour depths in the calibration test.

The equilibrium scour depths in the test with the upstream groyne are almost the same as the equilibrium scour depths in the calibration test. An upstream groyne does not result in a reduction in the scour depths along Nutan and Puran Bazar, compared with the other solutions. It should be added that in this test the discharge was 2 to 10 % higher than in the other tests, as a consequence thereof also the scour depths in this test can be slightly deeper.

In case of a guide bund along Puran Bazar a reduction of 2 to 10 m in the equilibrium scour depths can be observed if compared with the scour depths in the calibration test.

In cross section 28 the deepest scour depths are measured in case of a guide bund along Nutan Bazar and the complete guide bund.

Compared with the calibration test a reduction of the slope of the bank from 1 in 2 to 1 in 3.5 has as effect the reduction of the scour depths.

In T9 the scour depths around the head of the long upstream groyne has been measured at different points, see Figure 4.6a. Just at the edge of the falling apron a equilibrium scour depth of 38.5 m-P.W.D. follows from measurements. About 300 m downstream of the head of the groyne the deepest point of the scour hole has an equilibrium scour depth of 53 m-P.W.D. It is recommended to optimize the geometry of this groyne in order to reduce the scour depths and to reduce the costs of the protection of this groyne.

Initially this groyne was designed as a submerged groyne during the 1 in 100 year flood. The overflowing water caused considerable erosion at the downstream side of the groyne, therefore the crest of this groyne was raised until the crest had a higher level than the water level.

In test T12 the scour depths around the head of the two small groynes I and II have been measured. Just downstream of groyne II major scour depths have been recorded: a maximum equilibrium scour depth of 65 m - P.W.D., see Photograph D.4.4. It is recommended to optimize the geometry of this groyne in order to reduce these scour depths and to reduce the costs of the protection of this groyne.

Downstream of this scour hole the eroded material has been deposited. The local flow field has been influenced probably by this sedimentation, because during a test this material was systematically not removed.

The scour depth can be calculated with the formula of Ahmad:

$$y_s = k \cdot (u_0 \cdot h_0)^{0.67} - h_0$$

in which

u_0	=	depth averaged flow velocity just upstream of the scour hole	(m/s)
h_0	=	water depth just upstream of the scour hole, or the initial water depth without a scour hole	(m)
y_s	=	scour depth	(m)

For Chandpur as a first estimate the following values are assumed:

u_0 = 3.3 m/s
h_0 = 25 m

TABLE D.4.10 K-FACTOR IN THE CALIBRATION TEST T2

cross sect.	$y_s + h_0$	k
-	m	-
22	37 + 5.4 = 42.4	2.2
24	42 + 5.4 = 47.4	2.5
26	47 + 5.4 = 52.4	2.7
28	53 + 5.4 = 58.4	3.0
30	50 + 5.4 = 55.4	2.9
31	55 + 5.4 = 60.4	3.1
32	29 + 5.4 = 34.4	1.8

The calculated values of k in the calibration test are in the range of the values recommended by Breusers (i.e. 1.8 - 3.2). Therefore these values seem to be reasonable.

TABLE D.4.11 K VALUES AS A FUNCTION OF $h_0 + y_s$

ys + h0	k
m	-
45 + 5.4 = 50.4	2.6
50 + 5.4 = 55.4	2.9
55 + 5.4 = 60.4	3.1
60 + 5.4 = 65.4	3.4
65 + 5.4 = 70.4	3.7

In case of the complete guide bund in T8 the maximum scour depth of 58 m -P.W.D., results in a maximum value of k: k = 3.3.

In scour depths in the test with the complete guide bund two influences can be distinguished, if compared with scour depths in the calibration test:

- the reduction in the steepness of the slope from 1 in 2.0 to 1 in 3.5 results in a reduction of the scour depth,

- but also in the model the guide bund results in a reduction of the cross section for the discharge to pass. This results in constriction scour and an increase of the scour depths.

The combination of these two effects results in a modest increase of the scour depths and of the k-values (3.1 to 3.3).

D.4.8 Summary and conclusions

D.4.8.1 General

Within the frame work of the Meghna River Bank Protection Short-Term Study a scale model has been constructed of a part of the Lower Meghna River near Chandpur, a town at the left bank of this river. This scale model has been used to study the existing bank erosion problems along the Lower Meghna River and to test different short-term solutions.

A part of the Lower Meghna River near Chandpur having a length of about 5 km and a width of about 1.5 km has been modelled at a length scale of 1 in 150.

The non-distorted scale model has one main upstream inflow boundary, a small inflow boundary representing the flow over the char along the left bank upstream of Chandpur, a small inflow from the Dakatia river and one outflow boundary some kilometres downstream of Chandpur.

In the steady flow model of Chandpur a constant discharge has been pumped around. The influence of the tidal fluctuations on the scour process has been neglected, as is usual in this type of models, because of the conflicting time scales of the tidal flow and the local scour process. Since in the model the initiation of motion of the bed material is observed locally, no facilities for the circulation of the sediment are required. The discharge distribution at the upstream boundary of the model is based on the results of the mathematical model study of this project.

For the alternative solution with a groyne upstream of Chandpur three tests were executed, and the most accurate one, T9, has been analyzed.

The alternative solution with an advanced protection in the shape of a guide bund around Nutan Bazar and Puran Bazar has been tested in T8. This alternative solution has been split up in a guide bund around Nutan Bazar only: test T10, in a guide bund around Puran Bazar only: test T11 and (during construction) as a partial advanced protection (hard point) around Nutan Bazar: test T13. In test T11 Nutan Bazar has been supposed to be eroded almost completely, in that case also the left bank of the Dakatia River at Puran Bazar needs to be protected against the flow from the Lower Meghna River.

See Table D.4.3 for a summary of the test programme.

The tests without an asterisk in this Table have been analyzed in this Report. In all these tests the discharge, the water levels, the flow velocities and the scour depths have been measured.

D.4.8.2 Flow velocities

(a) *Calibration:*

The present situation of the bank lines and the bed geometry in February 1991 has been modelled in the calibration test. For calibration purposes the flow velocities in the model have been compared with the flow velocities in the field survey. The maximum flow velocities, which have been measured in the model along the bank of Nutan Bazar, are 0.1 to 0.2 m/s higher than the flow velocities, which have been measured in the flood survey of August and September 1991.

Some flow lines in the model were compared with the flow lines in the mathematical model. At the inflow and outflow boundaries of the model these flow lines coincide, but in the middle of the model the flow lines in the model show a stronger curvature than the flow lines in the mathematical model, because of the relatively inaccurate modelling of the scour hole in the mathematical model and because of a scale effect in the bed roughness. It can be concluded that the flow field in the model has been reproduced with sufficient accuracy.

(b) *Upstream groyne with a length of 1600 m:*

The maximum flow velocities around the groyne are 3.9 m/s near the head, 2.5 to 3.0 m/s at the most exposed part of the body and 2 to 2.5 m/s along the sand dam of the groyne near the bank. Along the bank of Nutan Bazar the maximum flow velocity is reduced with 20% from 3.2 m/s to 2.6 m/s. Along the bank of Puran Bazar the maximum flow velocity is reduced by about 10% from 2.3 to 2.1 m/s at 75 m from the bank and 3.05 to 2.9 m/s at 150 m from the bank.

(c) *Guide bund with advanced protection:*

The maximum flow velocity along the complete guide bund with advanced protection is 3.8 m/s, in case of a guide bund only at Nutan Bazar 3.5 m/s.

With a guide bund only at Nutan Bazar the maximum flow velocity near the existing bank of Puran Bazar is 1.5 m/s (in the existing situation it is 2.3 m/s).

If Nutan Bazar has been eroded almost completely, than a guide bund at Puran Bazar will induce a maximum flow velocity of 3.2 m/s.

(d) *Two groynes at Nutan Bazar:*

The maximum flow velocity near the head of groyne I is 1.9 m/s just downstream of the flow separation point. Along the body or the shaft of that groyne a maximum flow velocity of 1.5 m/s is to be expected.

The maximum flow velocity near the head of groyne II is 3.75 m/s in the axis of the groyne, and 2.3 to 3.6 m/s just downstream of the flow separation point.

Between these two groynes the flow velocities vary between 0.3 m/s and 1.4 m/s, and the maximum flow velocity along the existing bank upstream of groyne I will be about 0.8 m/s.

Along the existing bank of Puran Bazar the maximum flow velocities at 40 m from the bank vary from 0.25 to 0.6 m/s in upstream direction (in the calibration test: 0.8 to 1.6 m/s), because the flow along that bank from cross section 23 to 26 is part of a big eddy.

All the above mentioned flow velocities are time averaged flow velocities, which do not include the variations in the flow velocities by the eddies and the turbulence.

D.4.8.3 Scour depths

(a) *General:*

The scour depths are important parameters for the design of the toe level of the bank protection and the size of a falling apron.

In the calibration test the maximum depth of the scour hole increased from 51 m-P.W.D. down to 55 m-P.W.D. The flow velocities in the model are just high enough to reproduce the scour at the depth scale in the model.

(b) *Upstream groyne having a length of 1600 m:*

The maximum measured depth of the scour hole in front of Nutan Bazar is 53 m -P.W.D.
Near the head of the upstream groyne the maximum measured depth is 39 m-P.W.D., and in the scour hole 300 m downstream of this groyne the maximum measured depth is 53 m-P.W.D.

ঽ৫

(c) *Guide bund with advanced protection:*

In the test with the complete guide bund the maximum measured depth of the scour hole in front of Nutan Bazar is 58 m-P.W.D.

A guide bund only at Nutan Bazar results in almost the same depth: a maximum measured depth of 57 m-P.W.D. However, during a phased construction (in two seasons) of this guide bund at Nutan Bazar a hard point functioning as a partial guide bund induces a deep scour hole with the deepest point at 65 m-P.W.D. in the mouth of the Dakatia River. Due to the poor guidance of the flow the flow separates near the mosque and the strong vortex street causes the deep local scour hole.

When Nutan Bazar has been eroded almost completely the maximum measured scour depth in front of the advanced protection around Puran Bazar is 43 m -P.W.D.

(d) *Two groynes at Nutan bazar:*

Downstream of groyne II at Nutan Bazar a maximum depth of the scour hole of 65 m-P.W.D. has been measured. Along the existing bank of Puran Bazar this maximum depth is 48 m-P.W.D.

(d) *Comparison of alternative solutions*:

From the comparison of the different alternative solutions the following conclusions can be drawn:

- The upstream groyne has induced a small reduction of the scour depths near Nutan Bazar.
- The groynes at Nutan Bazar and also a partial guide bund (as a hard point) result in a deep scour hole downstream of these groynes.
- The complete guide bund creates a small increase in scour depth.
- The advanced protection or guide bund at Nutan Bazar is comparable with the complete guide bund and gives some protection to Puran Bazar as well.
  Therefore this alternative seems to be optimal from a hydraulic, morphological and economical point of view. However, during a phased construction of this advanced protection a deep local scour hole can be generated.

The formula of Ahmad has been applied to analyze the results in the calibration test. The value of an empirical coefficient in that formula has been determined from the measured data. These values are within the range recommended for this coefficient.

It is emphasized that in the model no falling apron has been modelled, therefore the measured scour depths are on the safe side.

TABLE D.4.12 THE REPRODUCTION OF THE MEASURED FLOW VELOCITIES IN SOME TESTS

test	cross section	distance from L.B.	velocity	velocity	difference (4)-(5)	(6)/(4)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
-	-	m	m/s	m/s	m/s	%
T2	29	75	3.336	3.225	0.111	3.3
		150	3.715	3.861	-0.146	3.8
		225	3.865	3.978	-0.113	2.8
		300	3.720	3.697	0.023	0.6
	24	75	1.770	1.630	0.14	7.9
		150	3.207	3.029	0.18	5.6
		225	3.871	3.861	0.01	0.3
		300	3.620	3.660	-0.04	1.1
T8	29	75	0.0	0.0	0	0
		150	3.442	3.754	-0.312	8.3
		225	3.596	3.843	-0.247	6.4
		300	3.877	3.703	0.174	4.5
	24	75	1.810	1.803	0.007	0.3
		150	2.902	2.979	-0.077	2.6
		225	3.278	3.319	-0.041	1.2
		300	3.257	3.313	0.056	1.7
T9	29	75	2.918	2.879	0.039	1.3
		150	3.263	3.375	-0.112	3.3
		225	3.503	3.492	0.011	0.3
		300	3.765	3.826	-0.061	1.5
	24	75	1.803	1.753	0.05	2.7
		150	3.063	3.152	-0.089	2.8
		225	3.391	3.369	0.022	0.6
		300	3.703	3.703	0	0

202

TABLE D.4.13 BED LEVELS AND SCOUR DEPTHS IN CALIBRATION TEST T2

cross section	location, type	initial bed level	equilibrium bed level	scour depth
-	-	m	m	m
22	,max	-32	-37	-5
22	,deepest			
24	,max	-36	-42	-6
24	,deepest			
28	,max	-46	-53	-7
28	,deepest			
30	,max	-46	-50	-4
30	,deepest	-46	-50	-4
31	,max	-51	-55	-4
31	,deepest			
32	,max	-21	-29	-8
32	,deepest			

* all levels below P.W.D.

TABLE D.4.14 BED LEVELS AND MAXIMUM SCOUR DEPTHS IN TEST T8

cross section	location, type	initial bed level	equilibrium bed level	scour depth
-	-	m	m	m
22	7 ,max	-32	-39.5	-7.5
22	10 ,deepest	-38.7	-32	+6.7
24	8 ,max/deep	-37.8	-49	-11.2
26	6 ,max	-38.3	-48	-9.7
26	4 ,deepest	-41.8	-41.5	+0.3
28	5 ,max/deep	-47	-58.4	-11.4
29	5 ,max	-43	-53	-10.0
29	- ,deepest	-48	-48.5	-0.5
30	8 ,max	-38	-47	-9.0
30	5 ,deepest	-46.5	-46.5	0
31	14 ,max/deep	-43	49.5	-6.5
32	16 ,max	-36.5	-50	-13.5
32	17 ,deepest	-38.5	-49.5	-11

* all levels below P.W.D.

TABLE D.4.15 BED LEVELS AND SCOUR DEPTHS IN TEST T9

cross section	location, type	initial bed level	equilibrium bed level	scour depth
-	-	m	m	m
22	7 ,max	-32.2	-41	-8.8
22	10 ,deepest	-38.5	-43	-4.5
24	5 ,max	-32.1	-40.8	-8.7
24	8 ,deepest	-37.8	-45	-7.2
26	8 ,max	-35.8	-47	-11.2
26	4 ,déepest	-41.8	-41.6	+ 0.2
28	7 ,max	-40.8	-51.5	-10.7
28	5 ,deepest	-47	-52	- 5.0
29	7 ,max	-41	-52	-11
29	3 ,deepest	-48.1	-53	- 4.9
30	9 ,max	-38.3	-49	-10.7
30	4 ,deepest	-46.7	-45.5	+ 1.2
31	17 ,max	-39.3	-49.8	-10.5
31	12 ,deepest	-48.5	-53	- 4.5
32	17 ,max	-38.3	-50	-11.7
32	14 ,deepest	-44.0	-46.5	- 2.5
groyne head	22 ,max	-27.8	-40	-12.2
groyne head	3 ,deepest	-43.5	-52	- 8.5

* all levels below P.W.D.

TABLE D.4.16 BED LEVELS AND SCOUR DEPTHS IN TEST T10

cross section	location, type	initial bed level	equilibrium bed level	scour depth
-	-	m	m	m
22	7 ,max	-32.4	-37.5	-5.1
22	10 ,deepest	-38.6	-35.5	+3.1
24	7 ,max	-37.5	-42.5	-5.0
24	8 ,deepest	-37.8	-42.5	-4.7
26	4 ,max	-41.8	-40.8	+1.0
26	7 ,deepest	-37.7	-46.0	-8.3
28	5 ,max, deepest	-47	-56.5	-9.5
29	6 ,max	-43.8	-51.5	-7.7
29	4 ,deepest	-48	-48	0
30	7 ,max	-43.0	-52.0	-9.0
30	4 ,deepest	-46.5	-46.5	0
31	14 ,max	-43.0	-51.5	-8.5
31	12 ,deepest	-48.2	-48.2	0
32	16 ,max	-36.8	-45	-7.2
32	14 ,deepest	-44.2	-44.5	-0.3

* all levels below P.W.D.

TABLE D.4.17 BED LEVELS AND SCOUR DEPTHS IN TEST T11

cross section	location, type	initial bed level *	equilibrium bed level *	scour depth
-	-	m	m	m
22	6 ,max	-26.5	-36	- 9.5
22	10 ,deepest	-38.5	-32	+ 6.5
24	5 ,max	-32	-38	- 6
24	8 ,deepest	-37.8	-37.5	+ 0.3
26	6 ,max	-38.4	-43.5	- 5.1
26	4 ,deepest	-41.8	-41.8	0
28	2 ,max	-27.7	-35.5	- 7.8
28	5 ,deepest	-47.0	-39	+ 8.0
30	4 ,max	-46.5	-41.8	+ 4.7
30	1 ,deepest	-10	-16	- 6.0
add.	27c ,max	-33.5	-37	- 3.5
add.	27a ,deepest	-10	-14.5	- 4.5

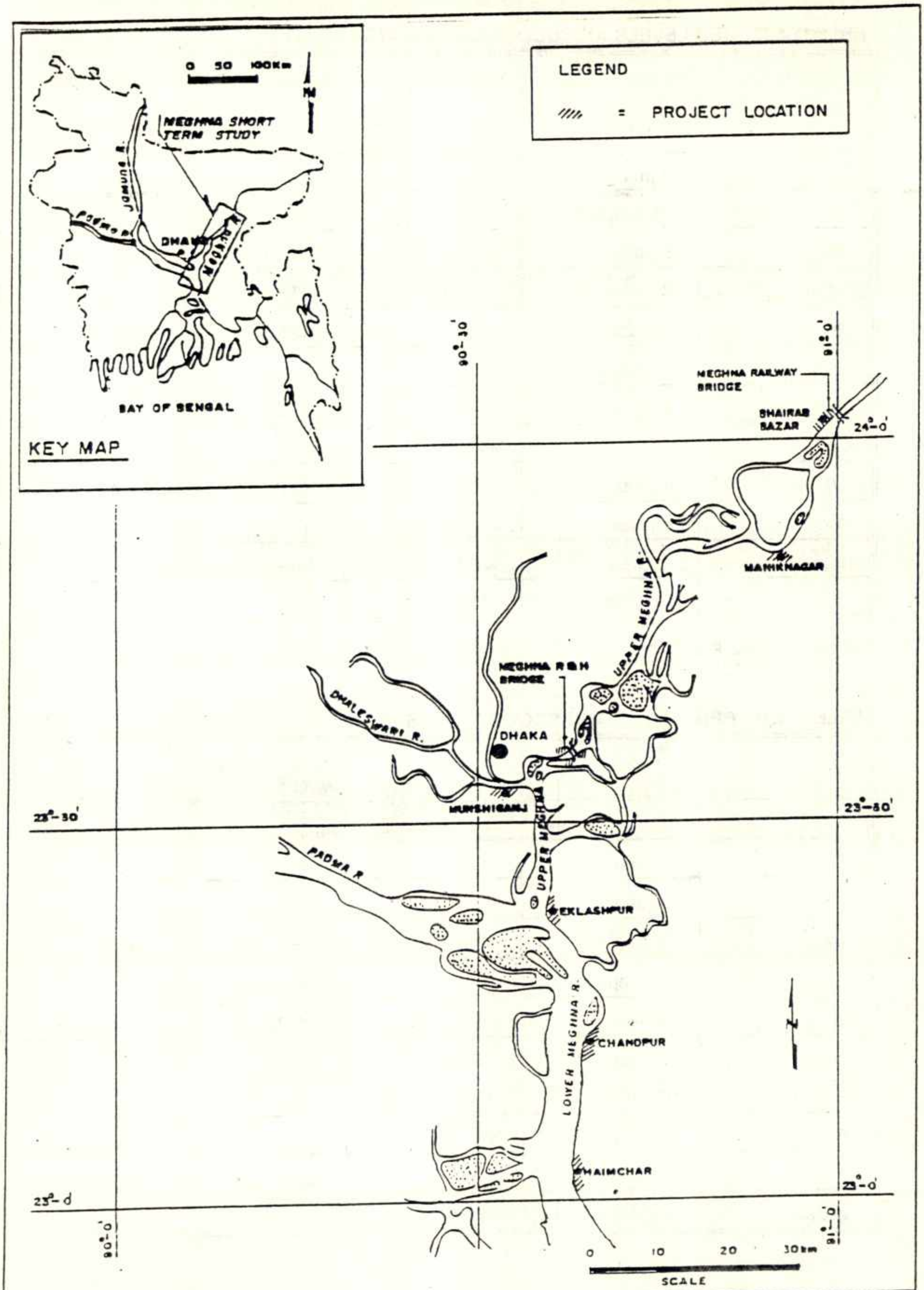
* all levels below P.W.D.

TABLE D.4.18 BED LEVELS AND SCOUR DEPTHS IN T12

cross section	location, type	initial bed level *	equilibrium bed level *	scour depth
-	-	m	m	m
22	11 ,max	-36.5	-45.5	-9.0
22	10 ,deepest	-38.25	-45.5	-7.25
24	9 ,max	-37.7	-48	-10.3
24	8 ,deepest	-37.8	-40	-2.2
26	9 ,max	-34.25	-49.25	-15.0
26	4 ,deepest	-41.8	-41.8	0
28	10 ,max	-36.5	-45	-8.5
28	6 ,deepest	-47.	-27.5	+19.5
groyne	13a,max	-40	-65	-25
groyne	5 ,deepest	-40.2	-48	-7.8

* all levels below P.W.D.

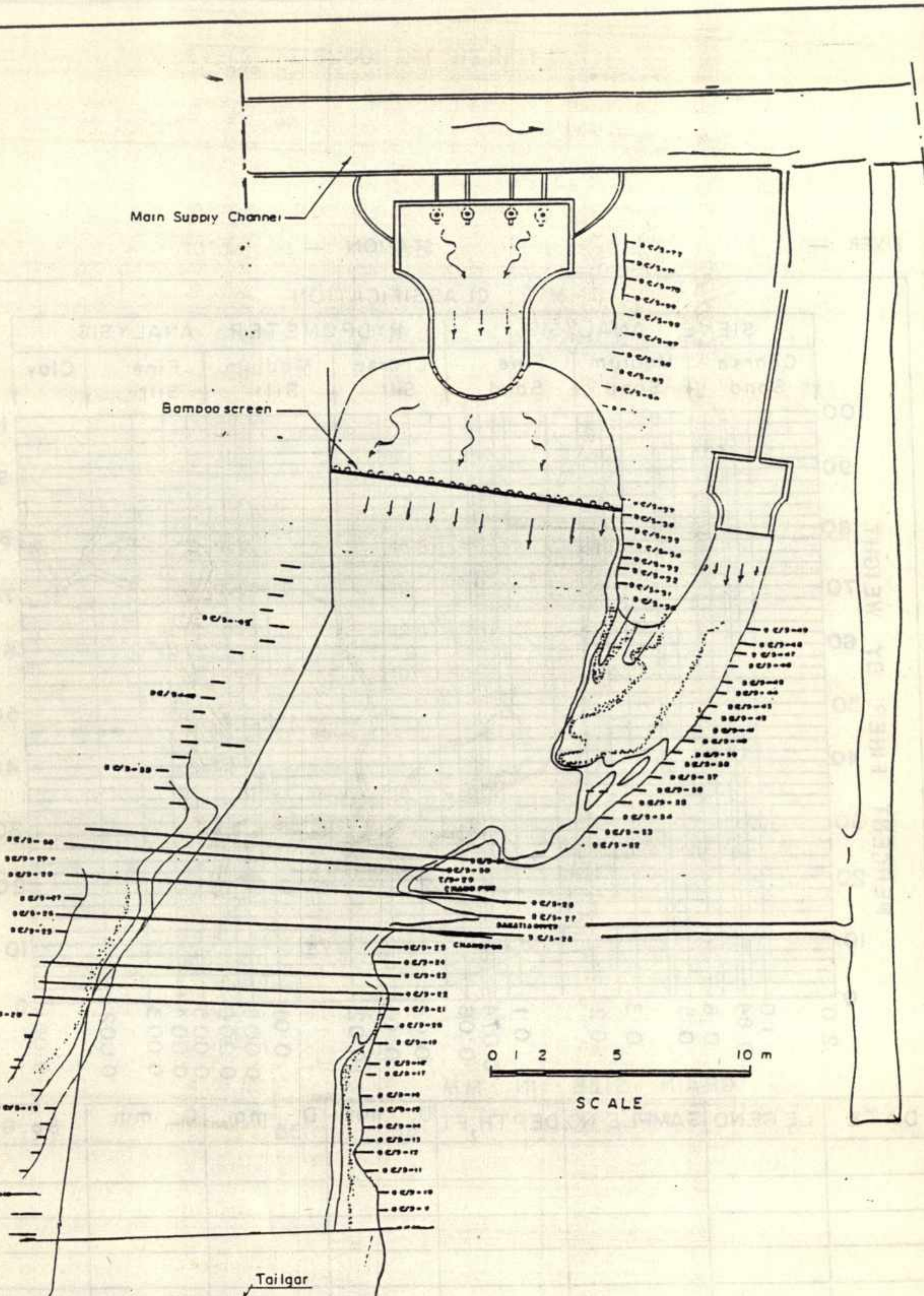
200



LOWER MEGHNA RIVER

CHANDPUR

MRBPSTS | Fig. D. 4.1



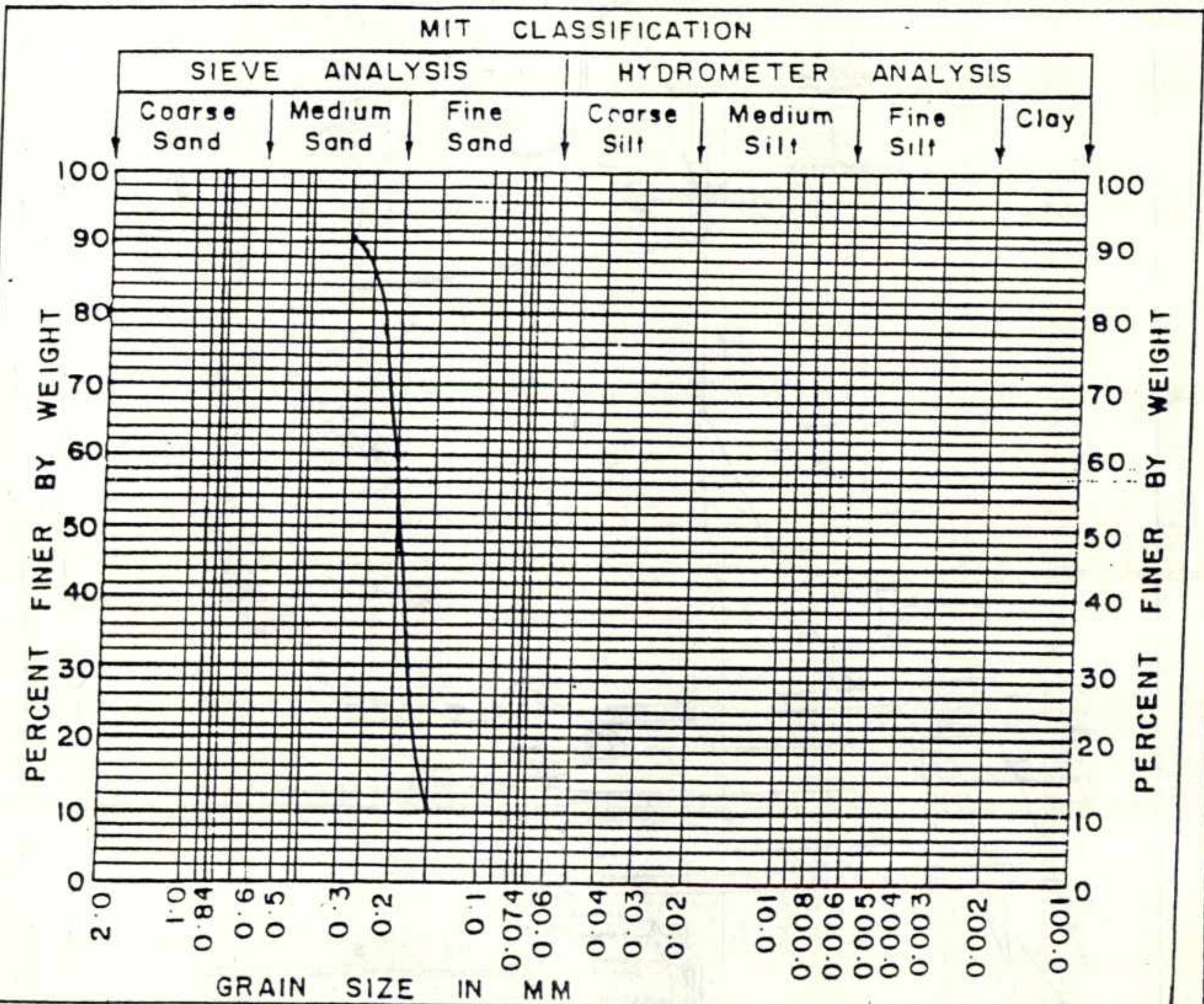
LAYOUT OF THE MODEL

CHANDPUR

MRBPSTS

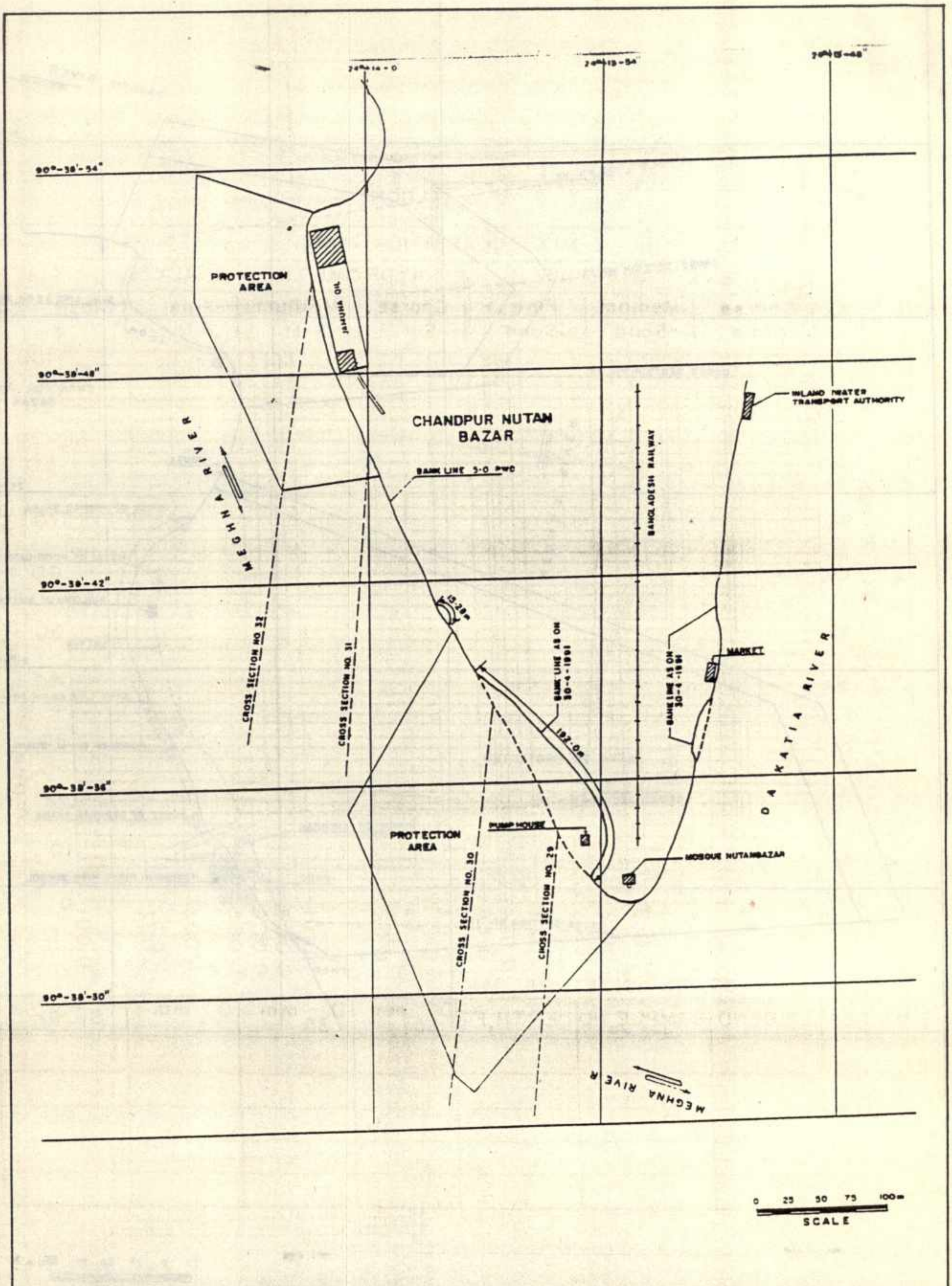
FIG. D.4.2

RIVER :— STATION :—

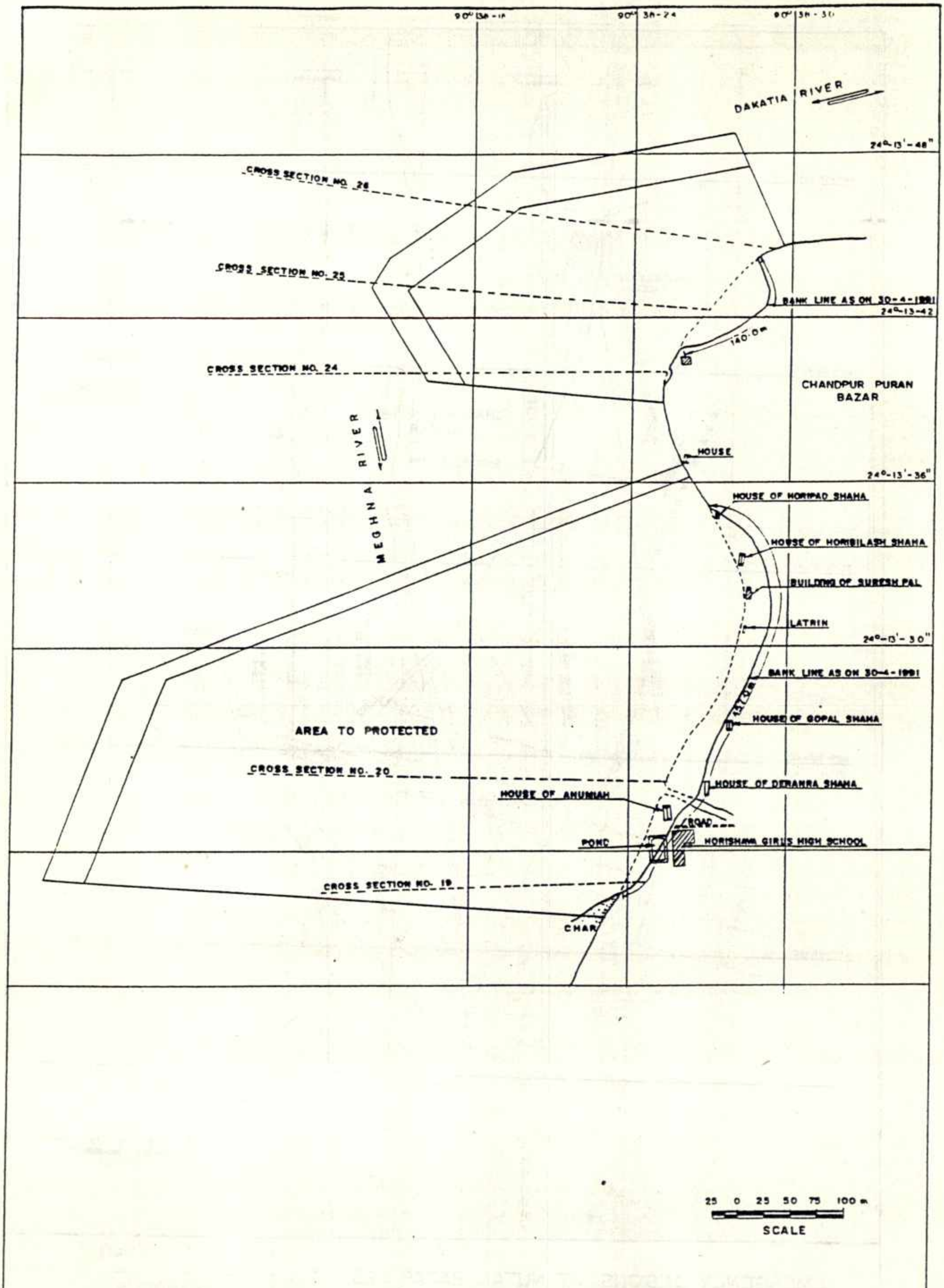


DATE	LEGEND	SAMPLE NO	DEPTH, FT	D_{65} mm	D_{35} mm	D_m mm	Sp. Gr.

GRADATION CURVE OF BED MATERIAL IN THE MODEL	CHANDPUR	
RIVER RESEARCH INSTITUTE	MRBPSTS	FIG. D.4.3

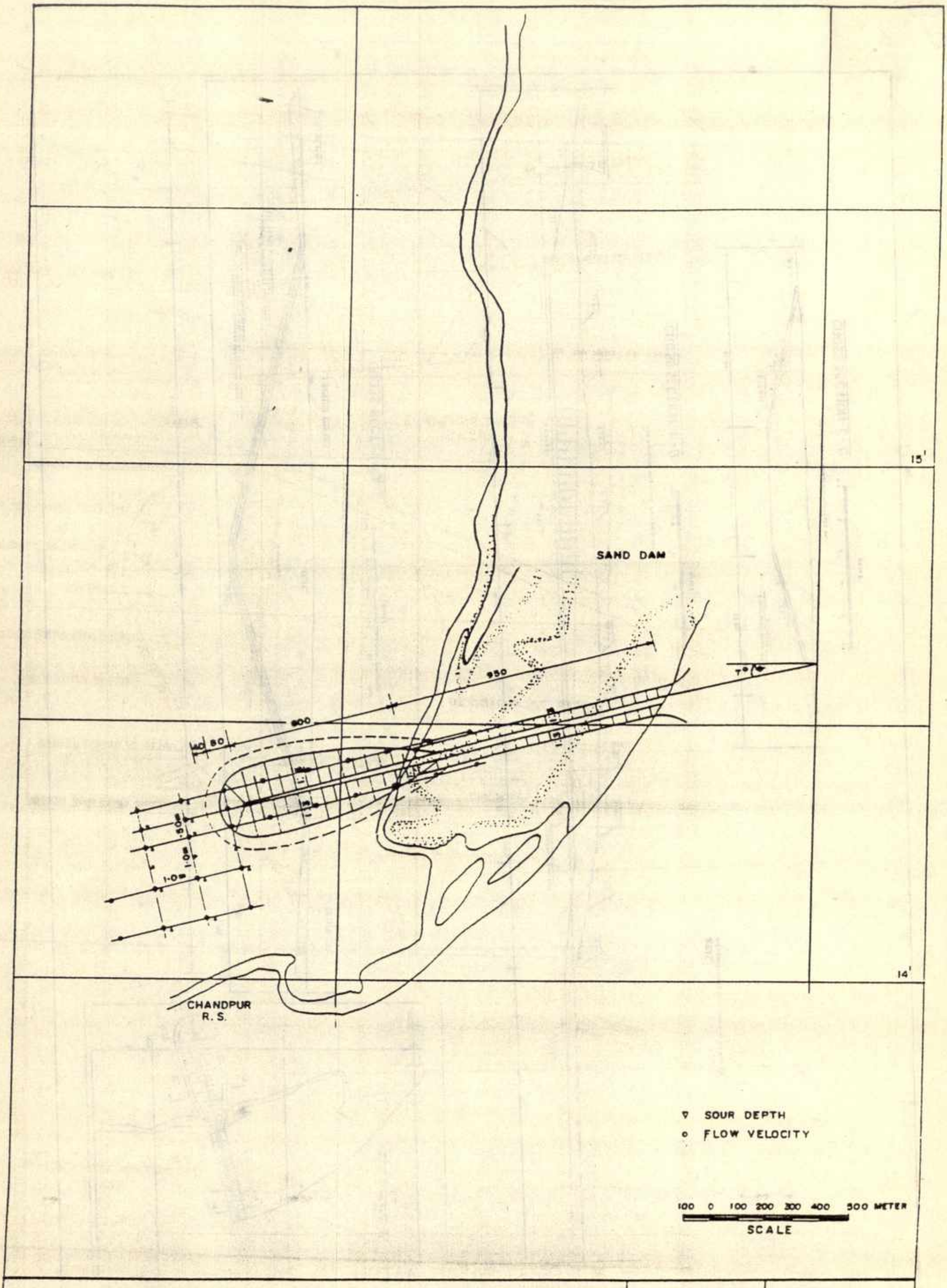


EMERGENCY DESIGNS AT NUTAN BAZAR TEST T3		DATE
	CHANDPUR	
	MRBPSTS	FIG. D.4.4

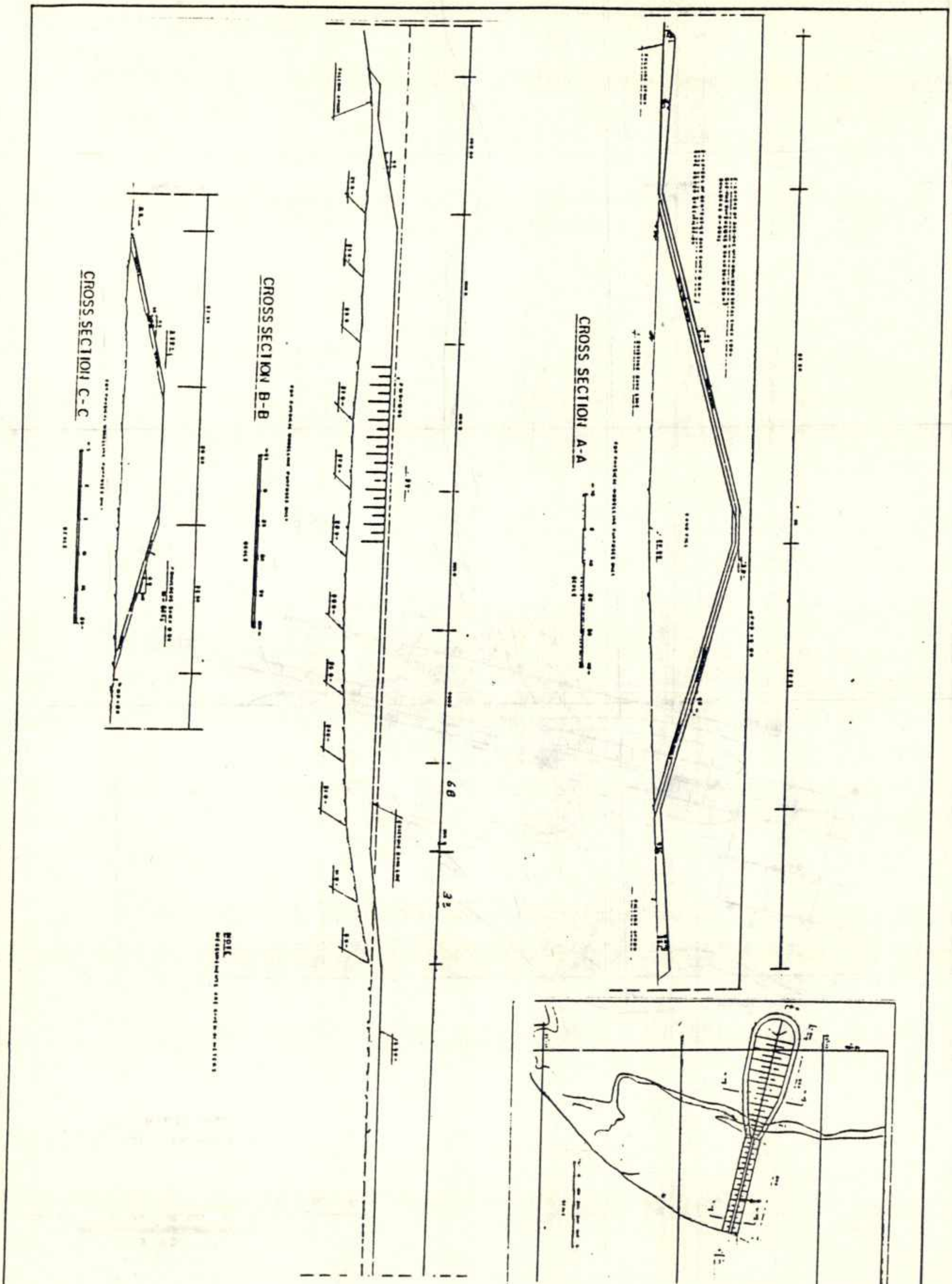


EMERGENCY DESIGNS AT PURAN BAZAR		DATE
	CHANDPUR	
	MRBPSTS	FIG. D.4.5

OSO

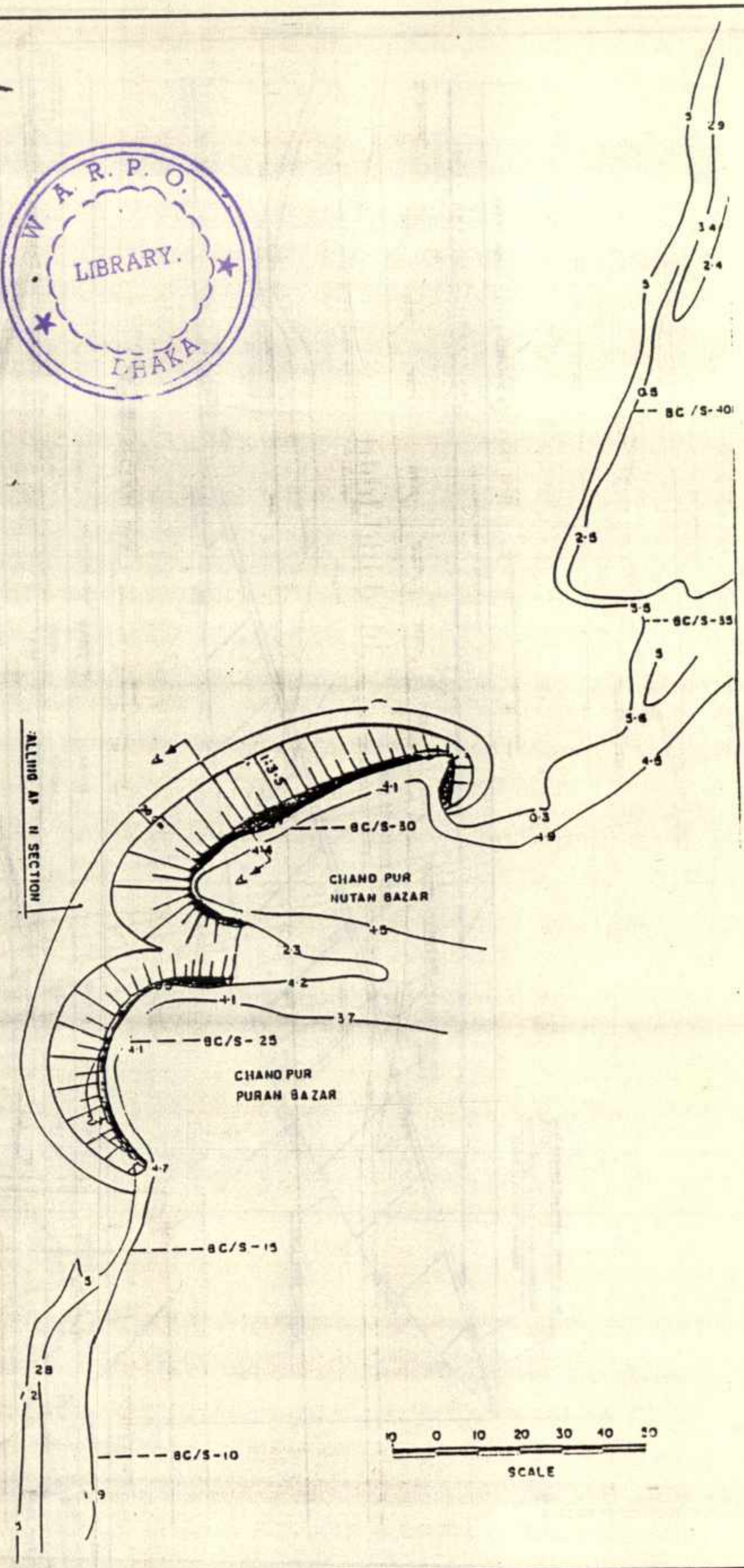


LAY-OUT GROYNE IN TEST T 7		DATE
	CHANDPUR	
	MRBPSTS	FIG. D.4.6(a)

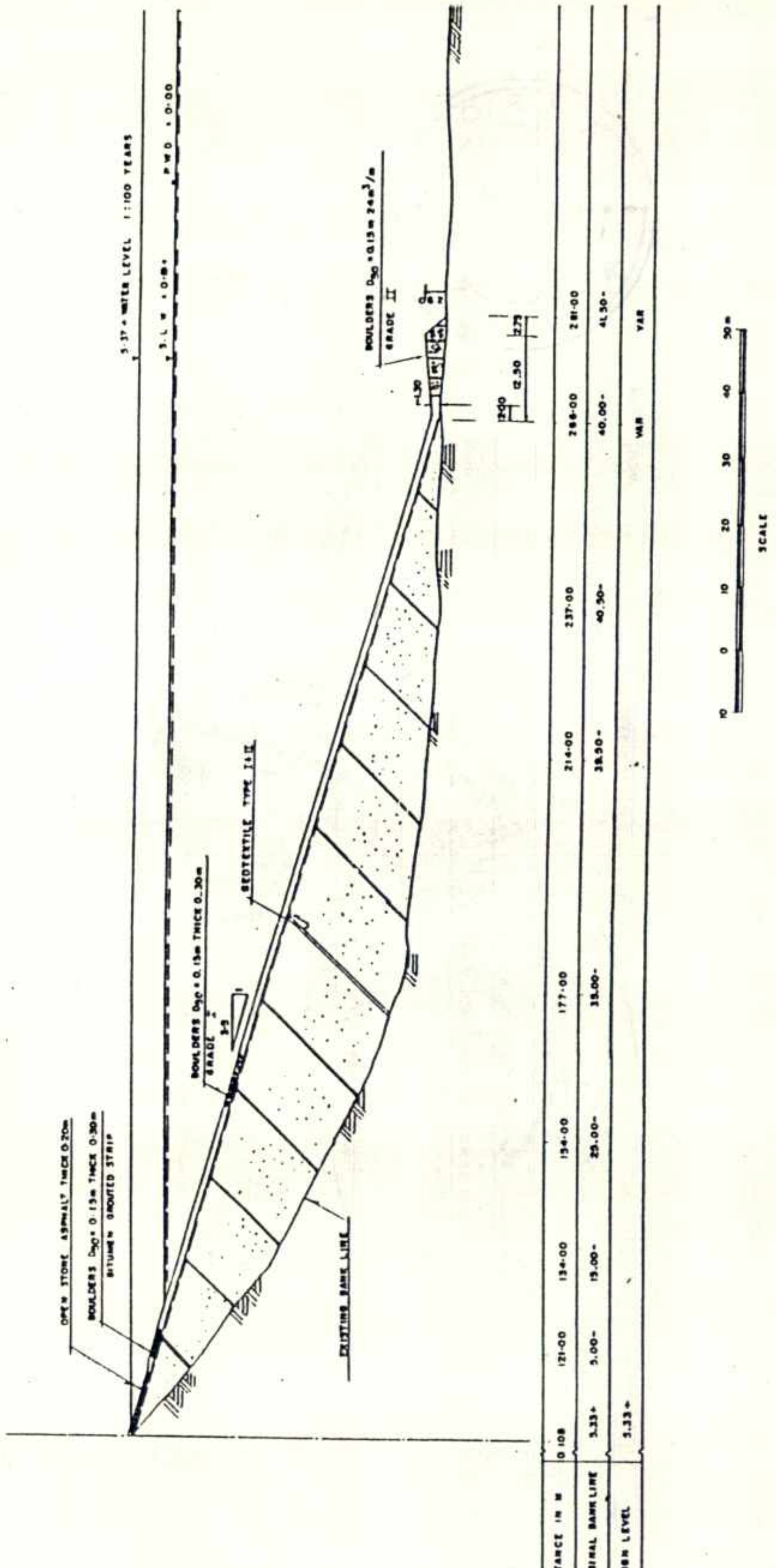


LAYOUT GROYNE. CHANDPUR	FIGURE. D. 4.6 (b)

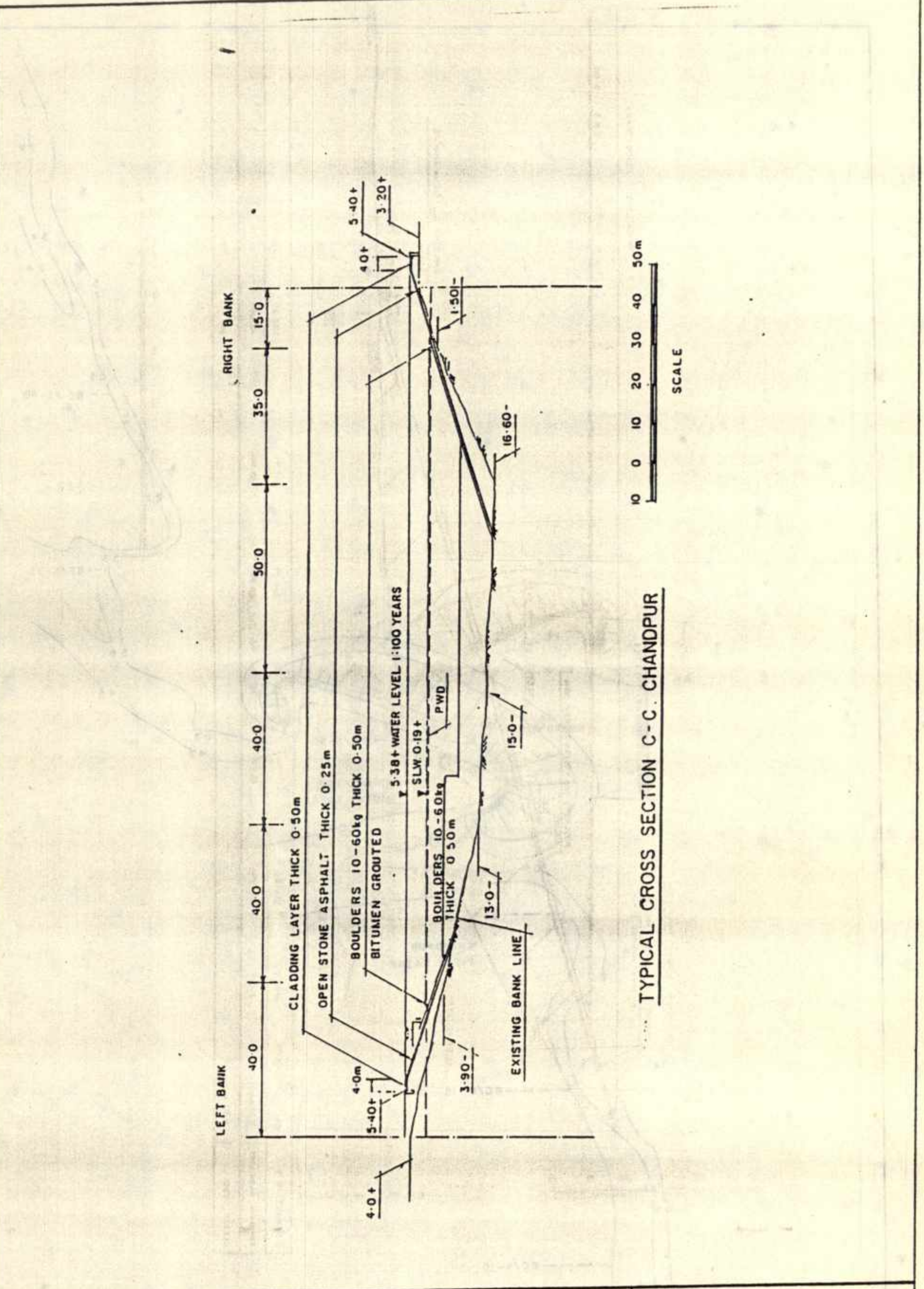
282



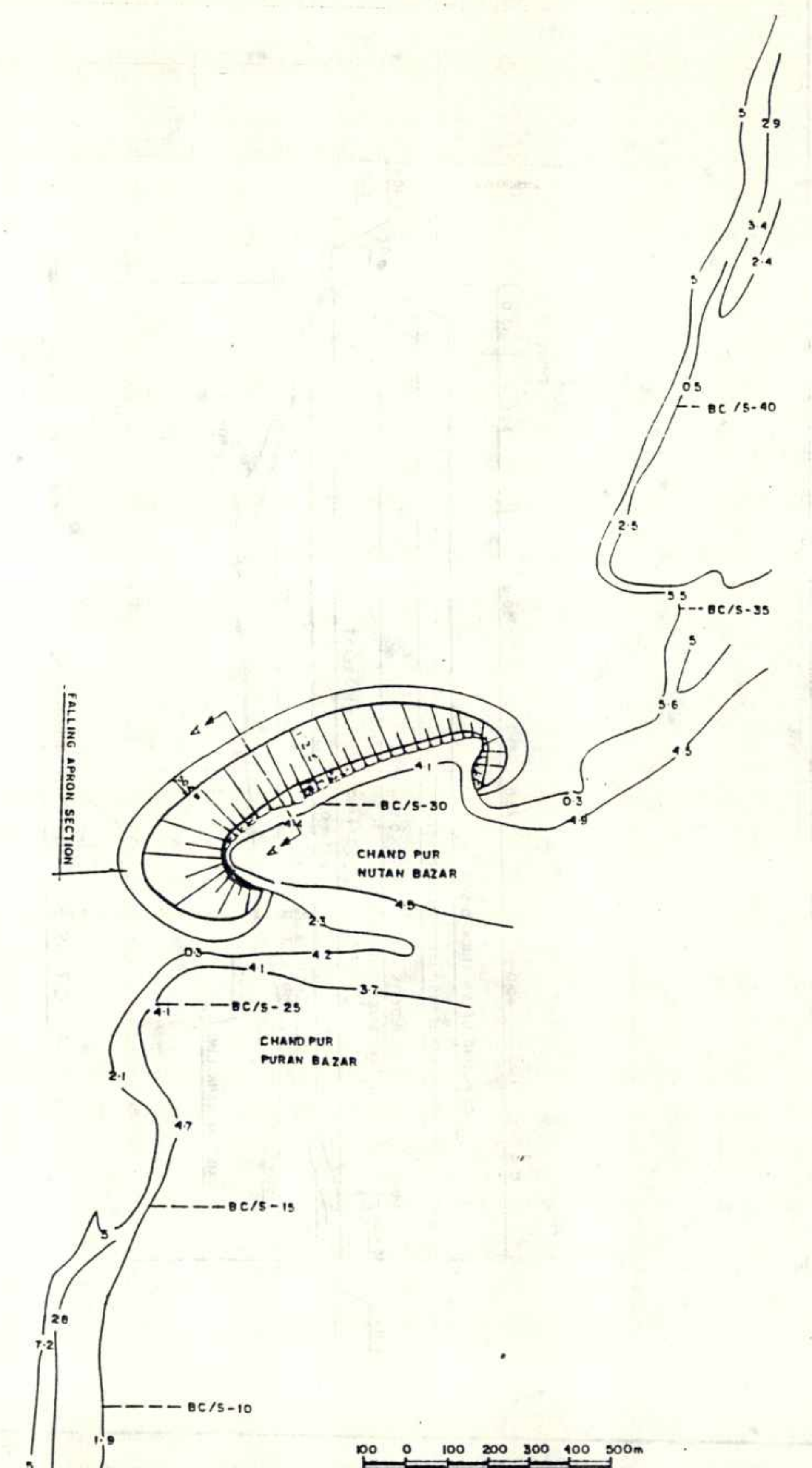
LAYOUT ALTERNATIVE WITH COMPLETE ADVANCED PROTECTION TEST T 8. CHANDPUR	FIGURE. D. 4.7



CROSS SECTION ADVANCED PROTECTION, CHANDPUR — FIGURE. D. 4.8

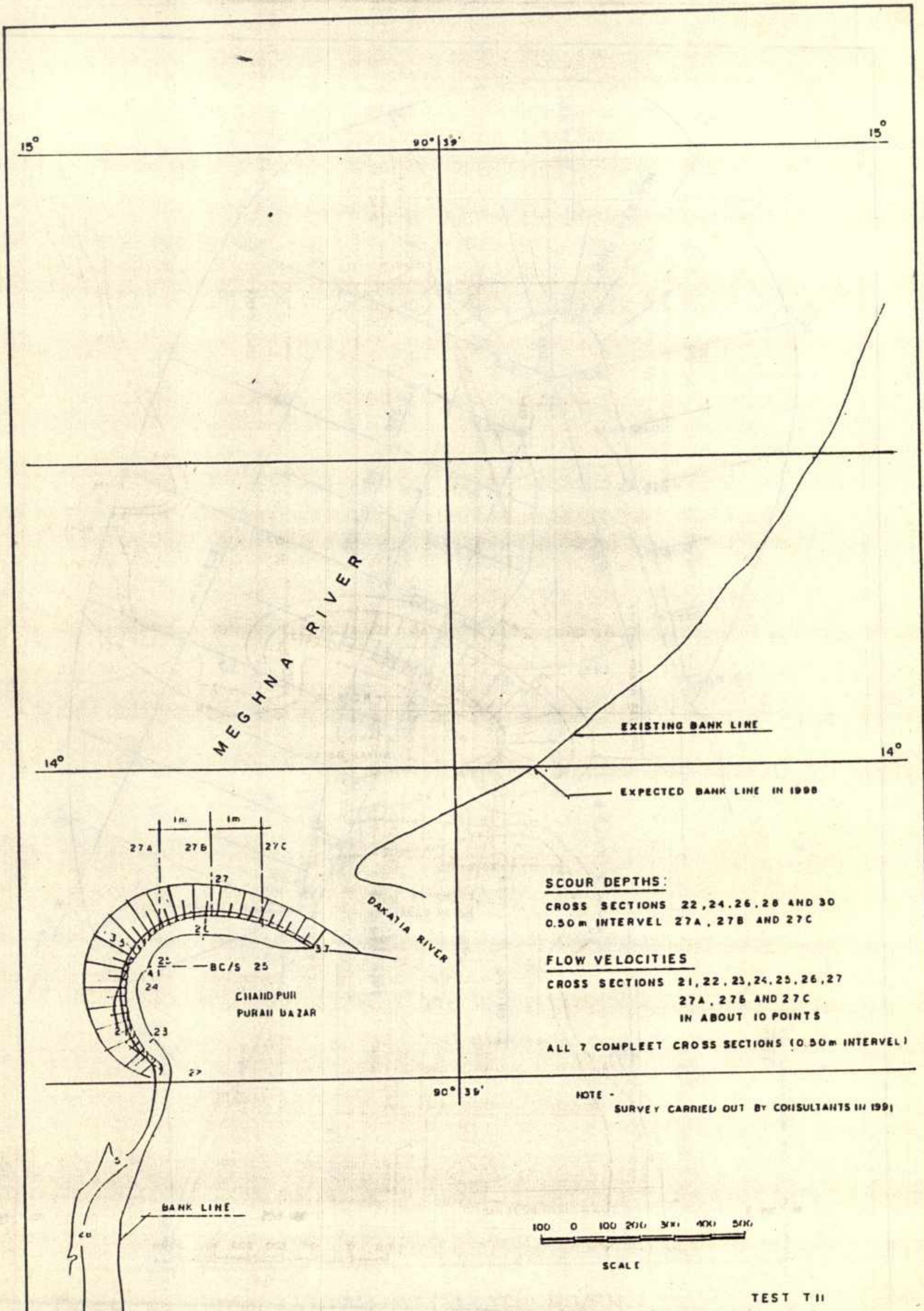


CROSS SECTION MOUTH OF DAKATIA RIVER	CHANDPUR	
	MRBPSTS	FIG. D. 4.9



LAYOUT ALTERNATIVE WITH ADVANCED PROTECTION IN NUTAN BAZAR CHANDPUR	FIGURE D. 4.10

224



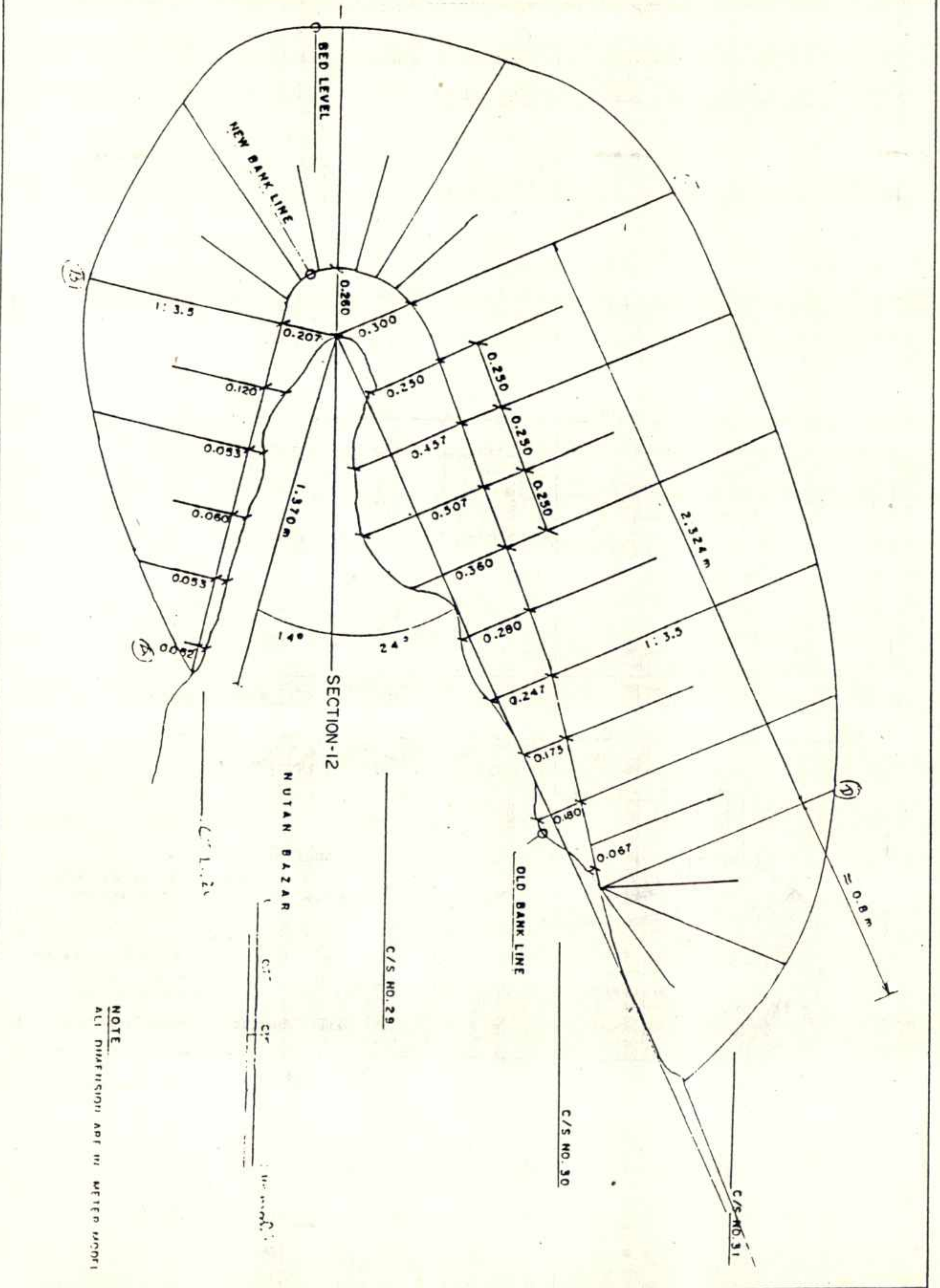
LAYOUT ALTERNATIVE WITH ADVANCED PROTECTION IN PURAN BAZAR

CHANDPUR

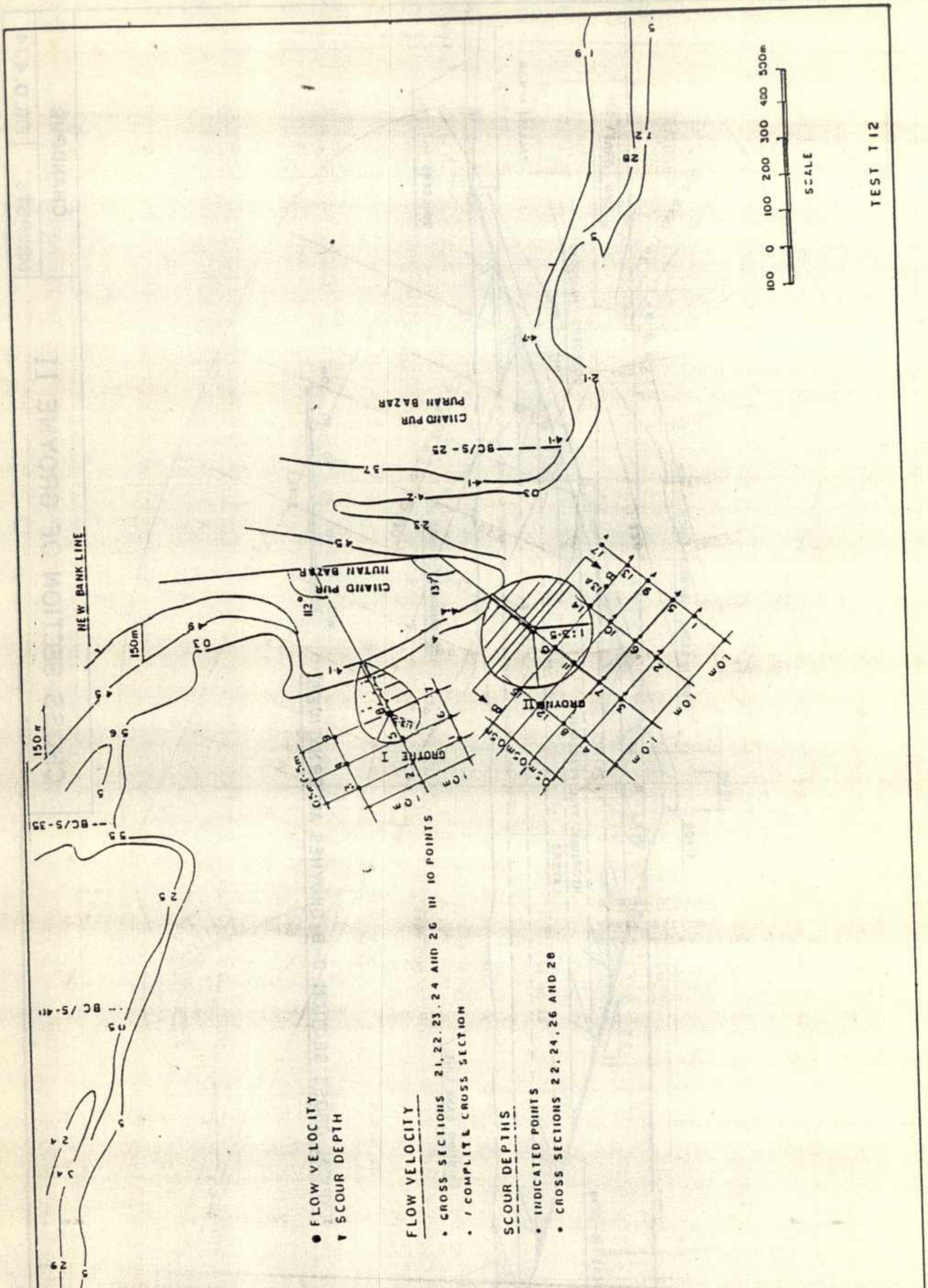
MRBPSTS

FIG. D.4.11

239



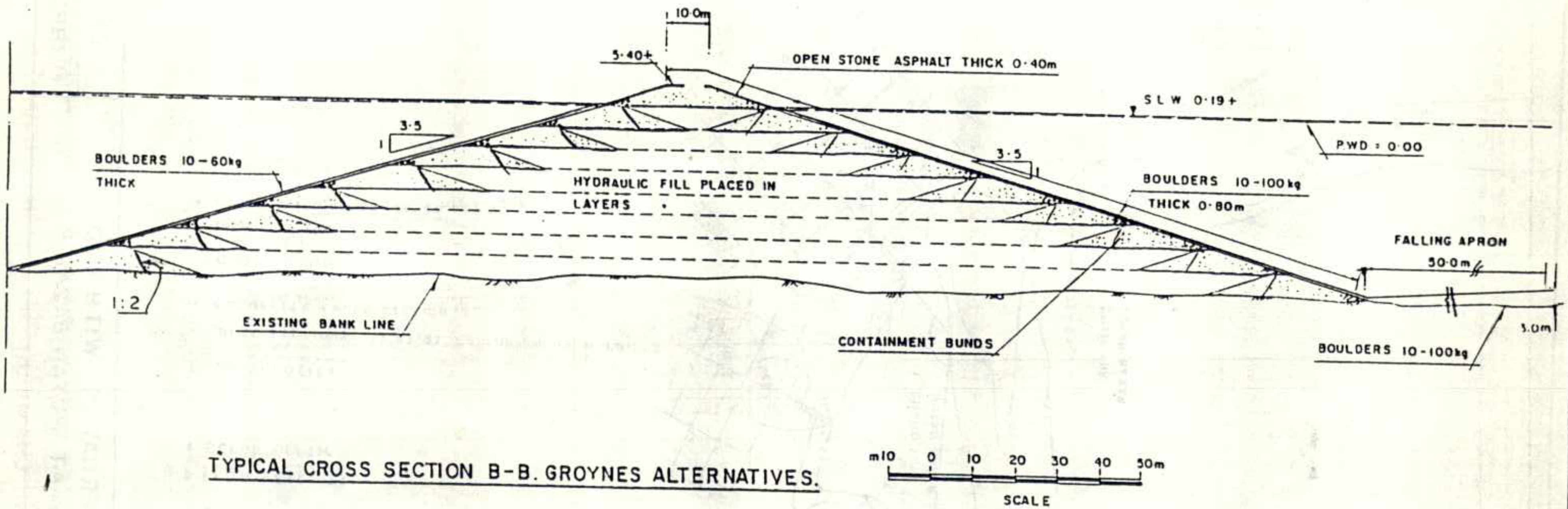
LAYOUT ALTERNATIVE WITH A HARD POINT AROUND NUTAN BAZAR	CHANDPUR	
	MRBPSTS	FIG. D.4.12



LAYOUT ALTERNATIVE WITH TWO GROYNES AT NUTAN BAZAR

CHANDPUR

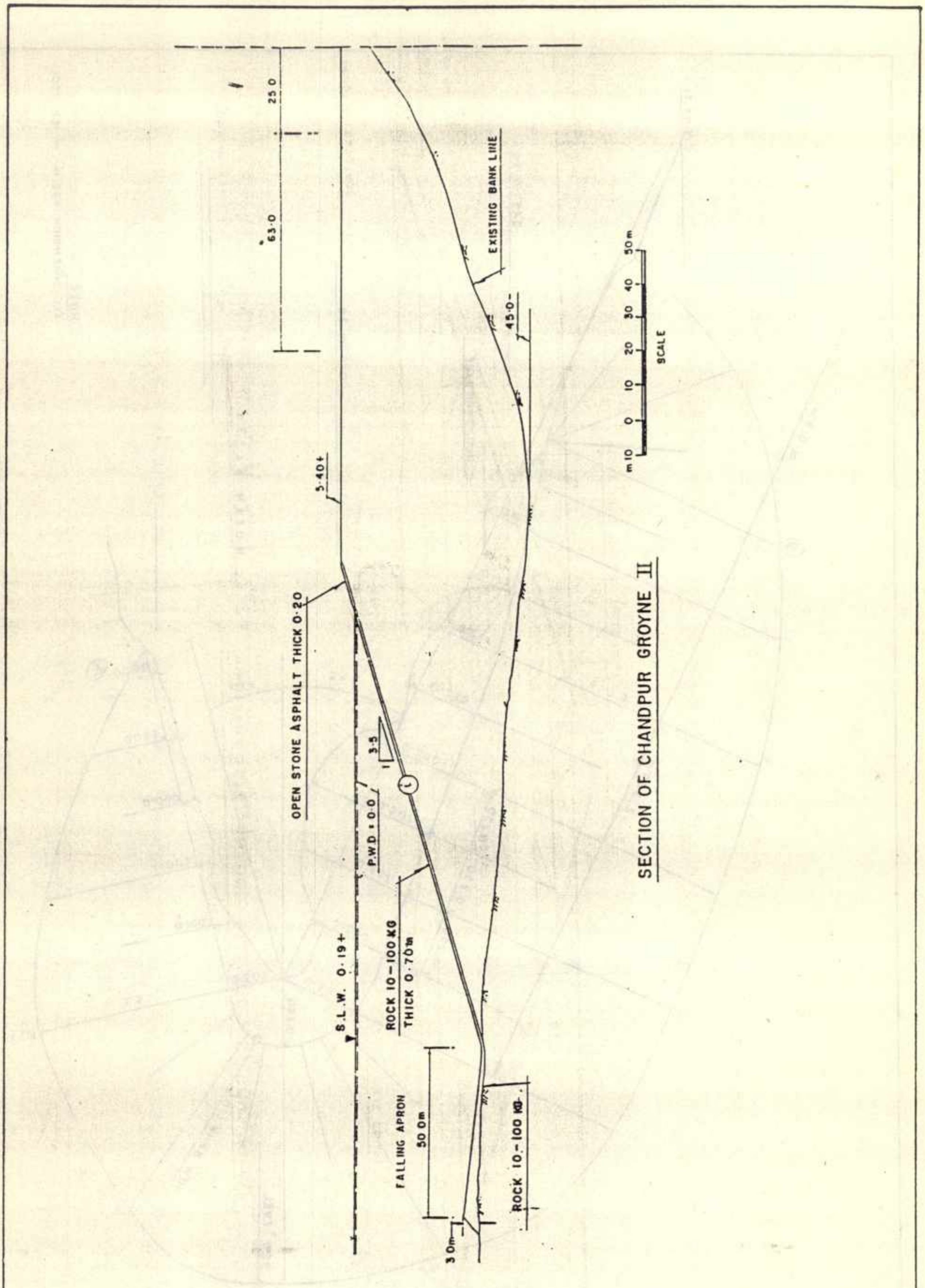
MRBPSTS | FIG. D. 4.13



TYPICAL CROSS SECTION B-B. GROYNES ALTERNATIVES.

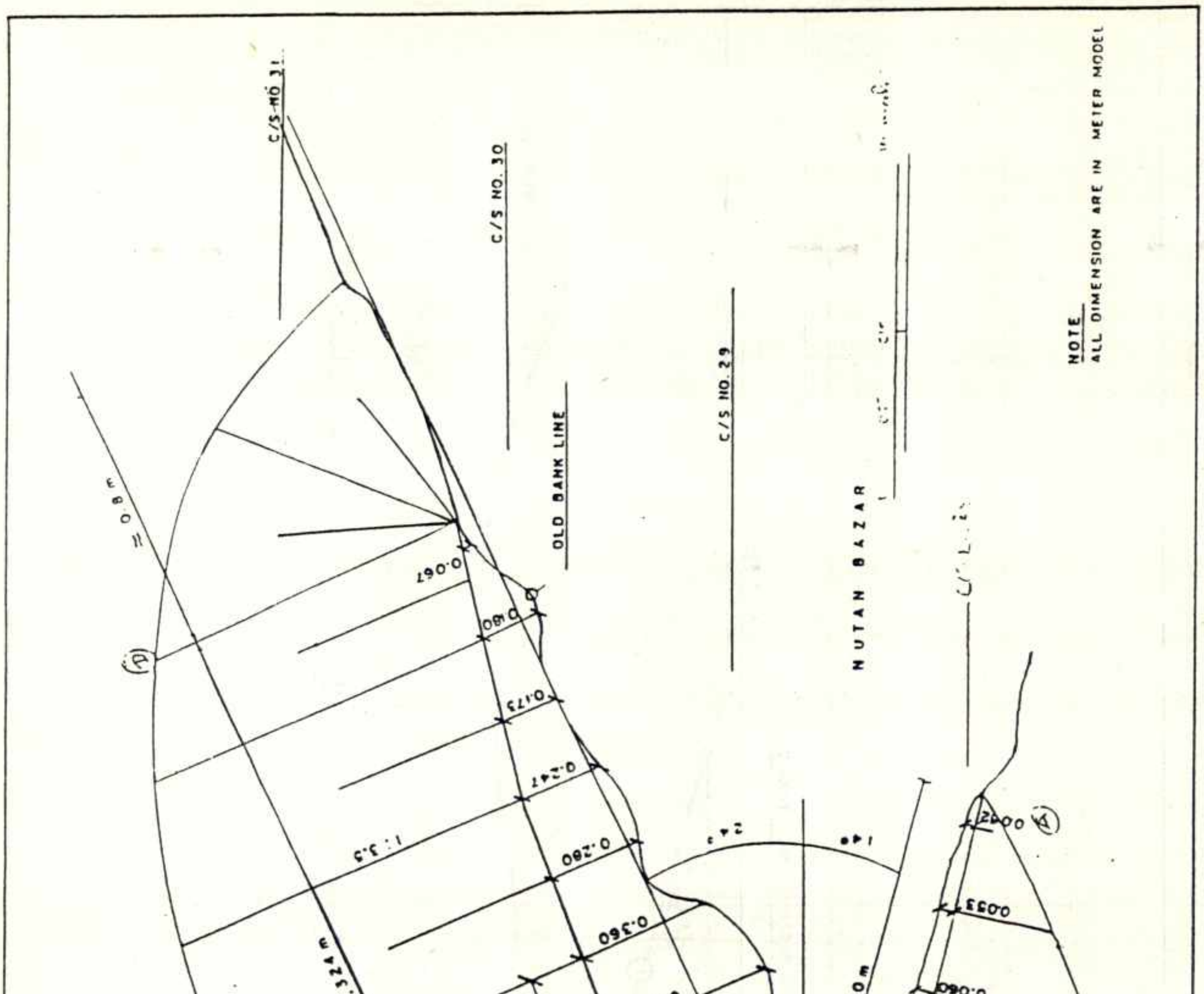
CROSS SECTION OF GROYNE II

CHANDPUR	
MRBPSTS	FIG. D.4.14



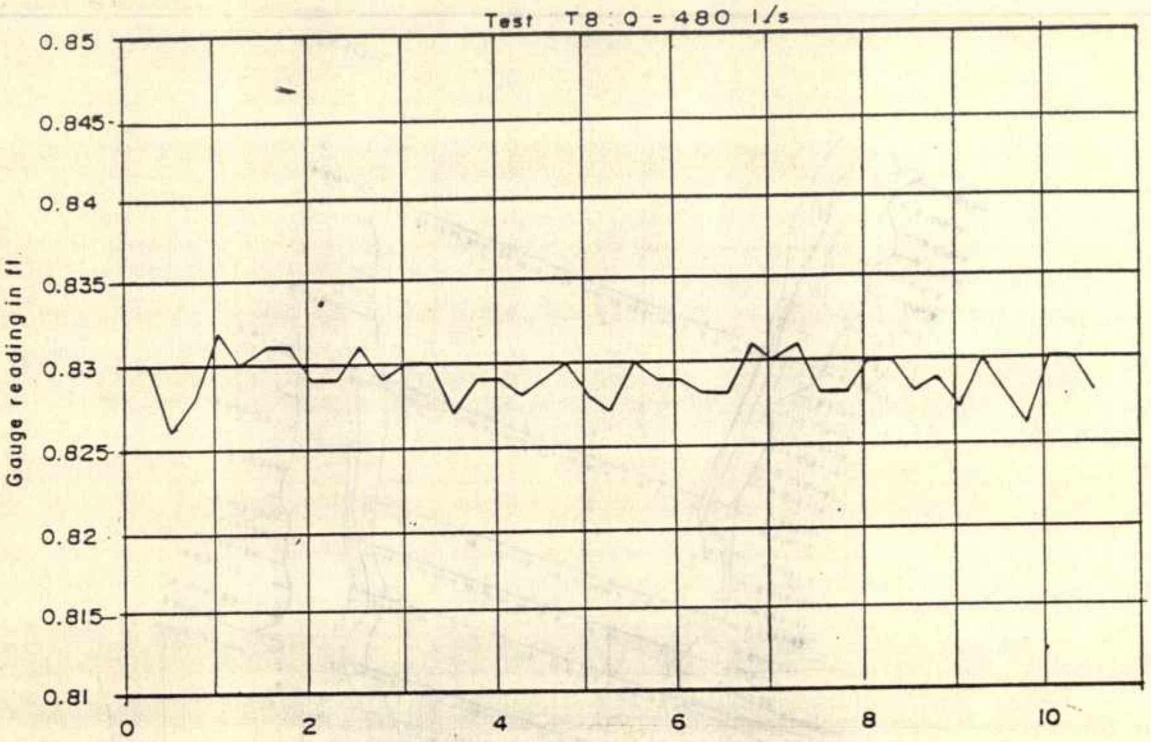
LONGITUDINAL SECTION OF GROYNE II
CHANDPUR

FIGURE. D. 4.15



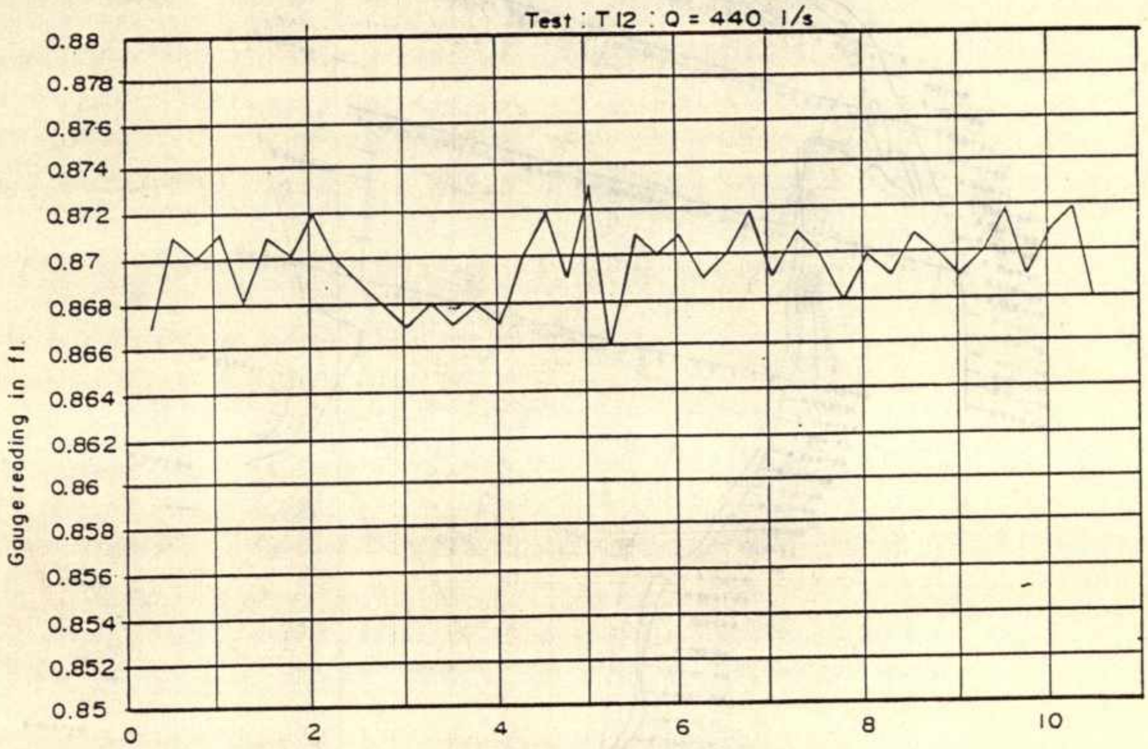
LAYOUT ALTERNATIVE WITH A HARD POINT (PROTECTION) AROUND NUTAN BAZAR	CHANDPUR	
	MRBPSTS	FIG. D.4.16

Test T8 : Q = 480 l/s



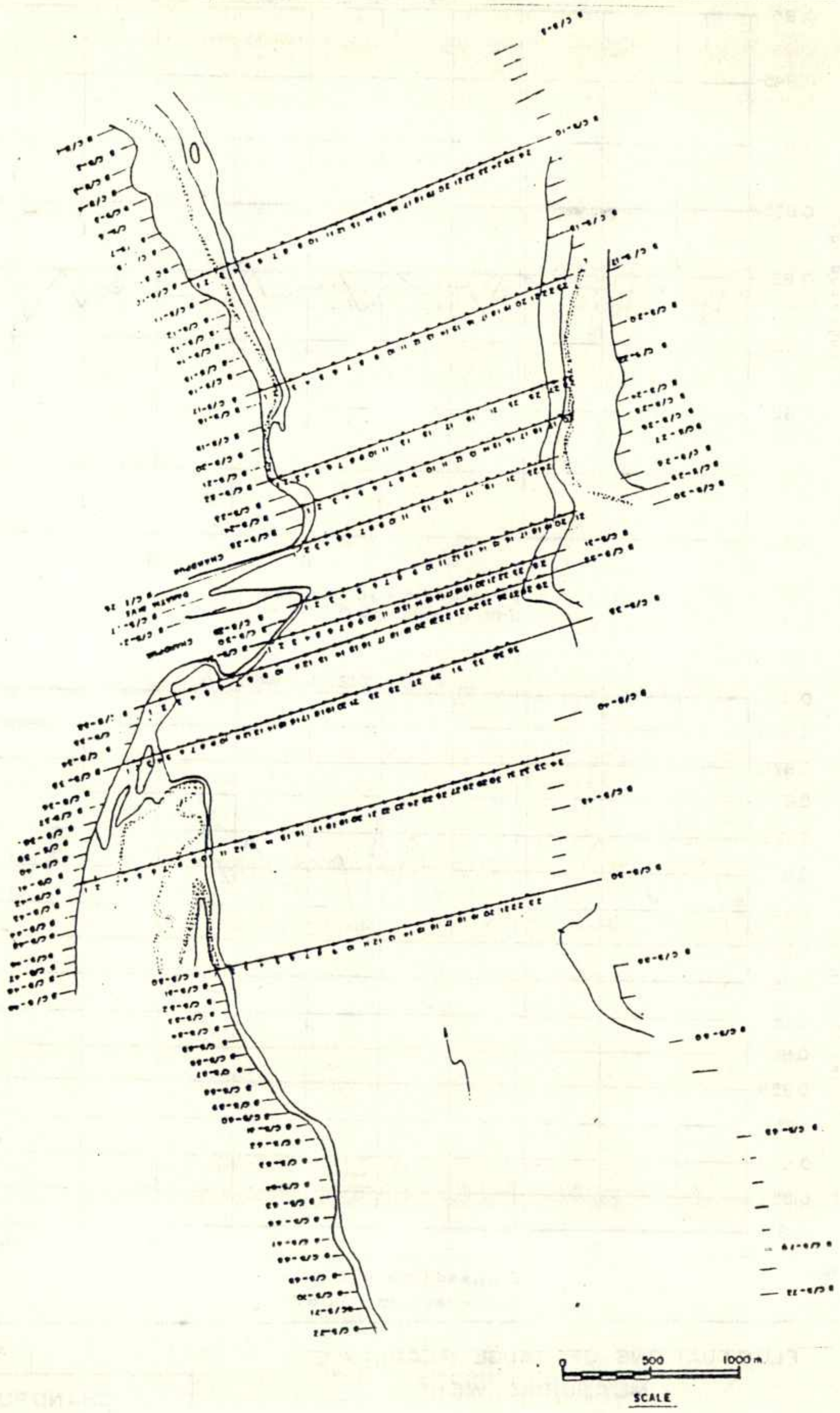
Elapsed time in minutes
Gauge reading = 0.826 ft.

Test : T12 : Q = 440 l/s



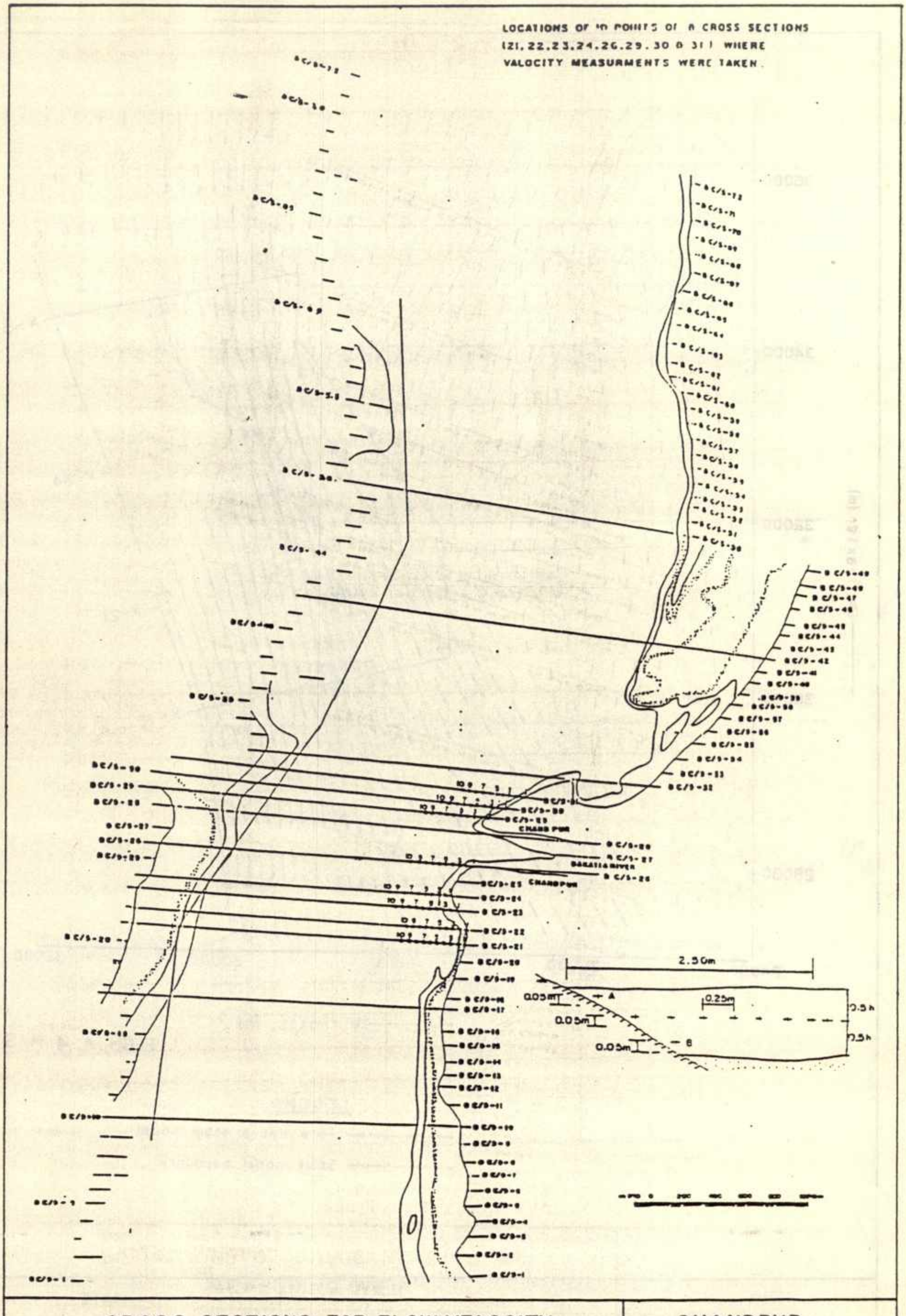
Elapsed time in minutes
Gauge reading = 0.867 ft.

FLUCTUATIONS OF GAUGE READING OF MEASURING WEIR		DATE
	CHANDPUR	
	MRBPSTS	FIG.D. 4.17



FLOW LINES IN TEST T	CHANDPUR	
	MRBPSTS	FIG. D.4.18

328



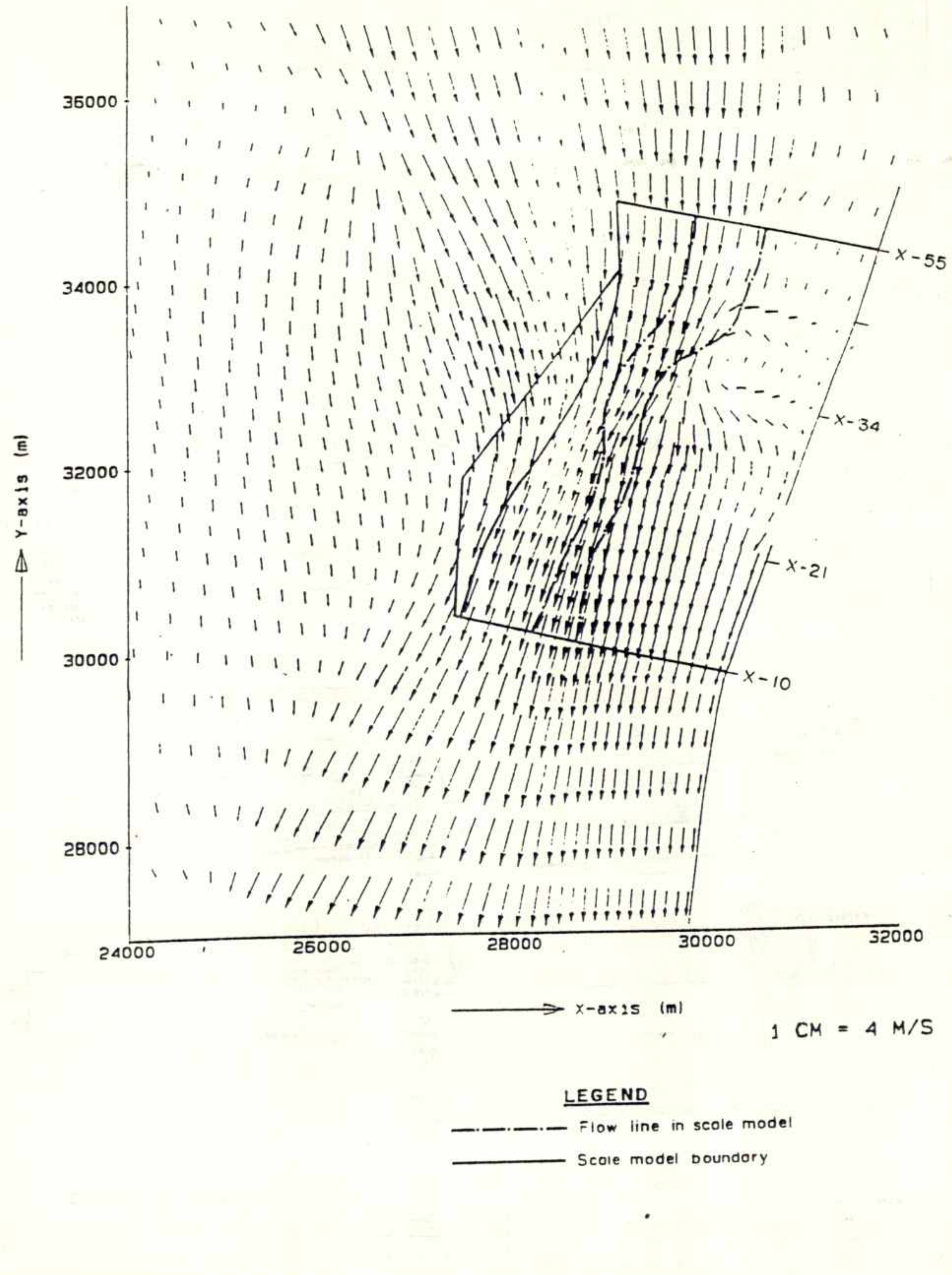
CROSS SECTIONS FOR FLOW VELOCITY MEASUREMENTS

CHANDPUR

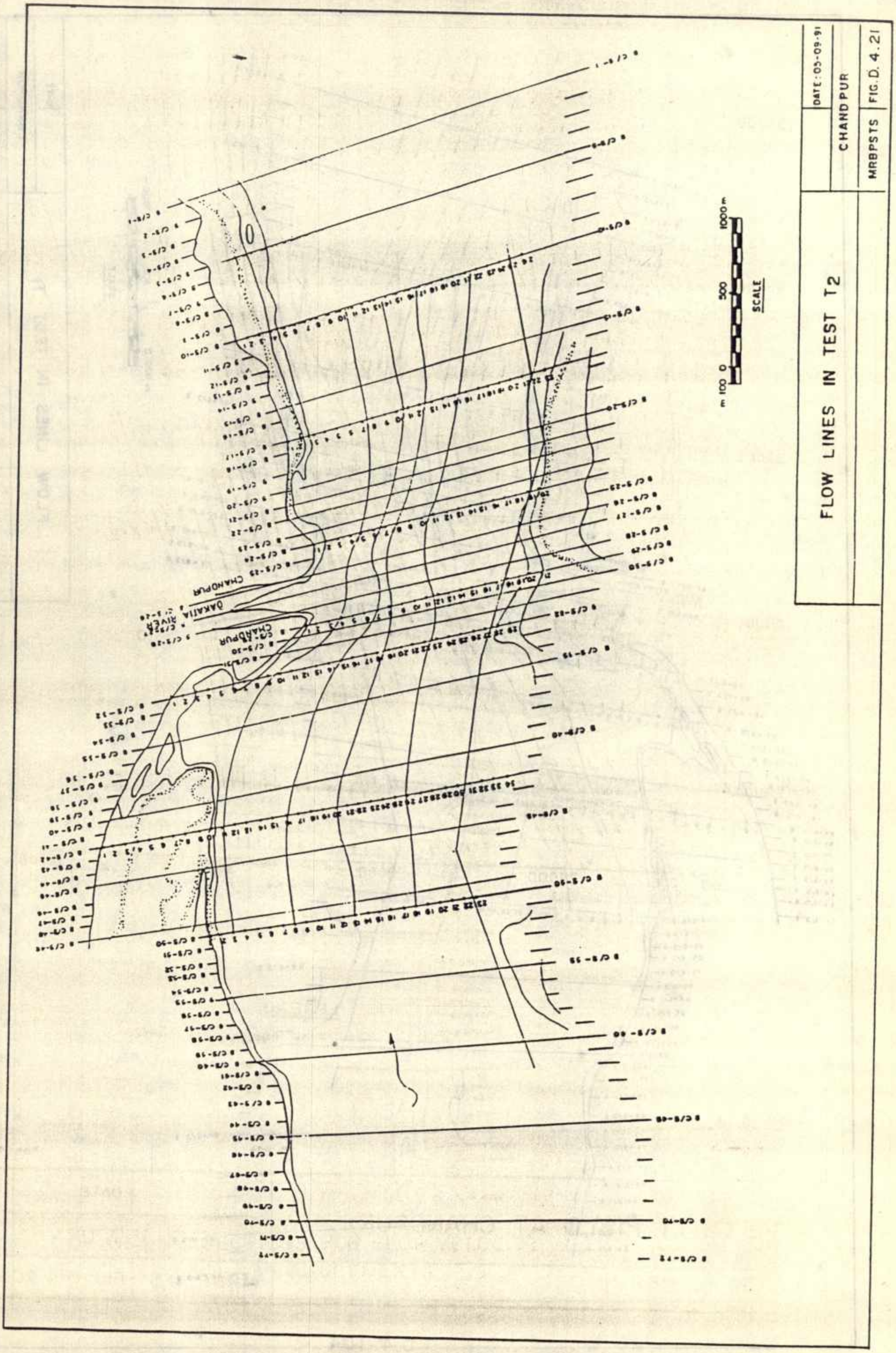
MRBPSTS

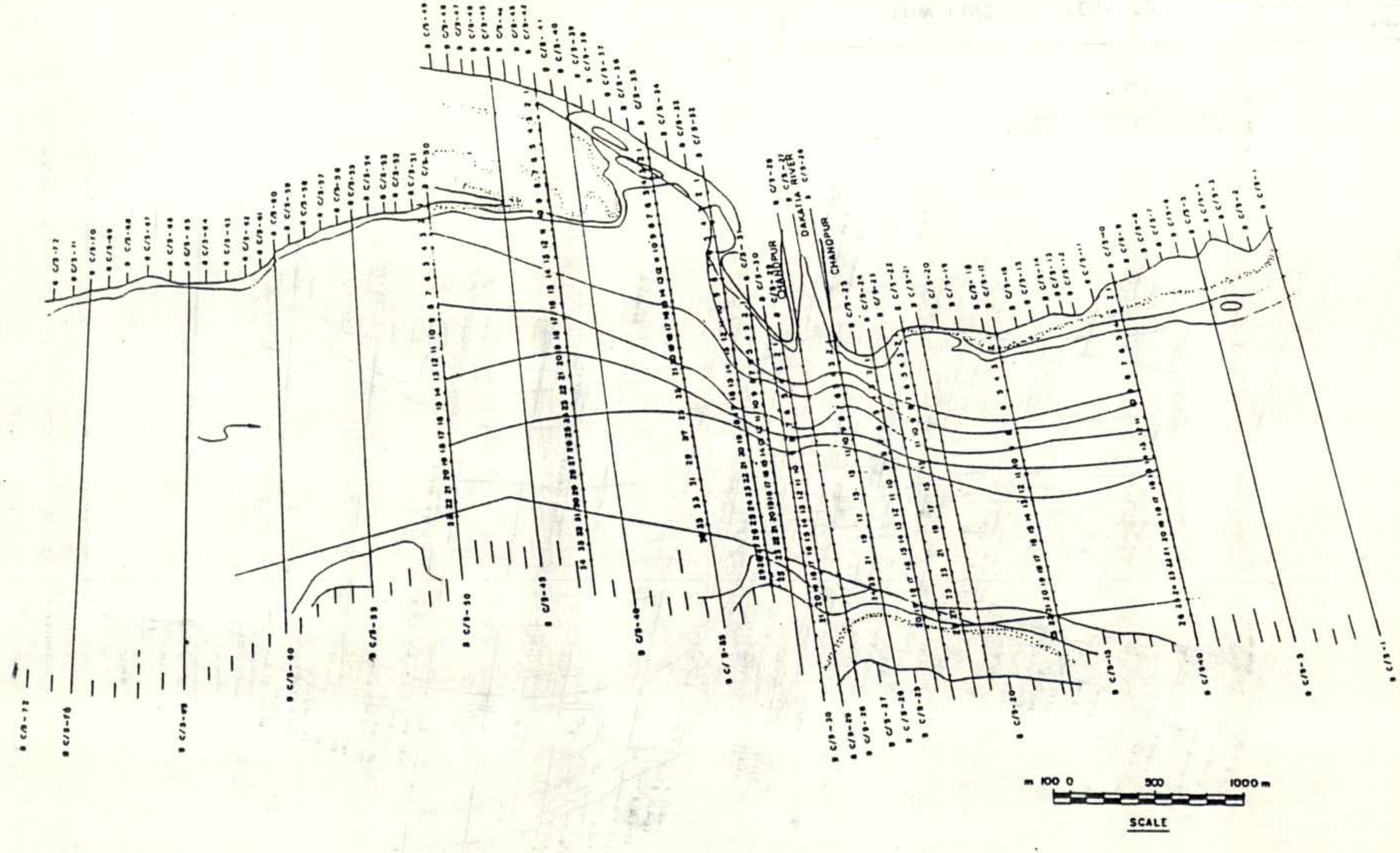
FIG. D.4.19

DZC

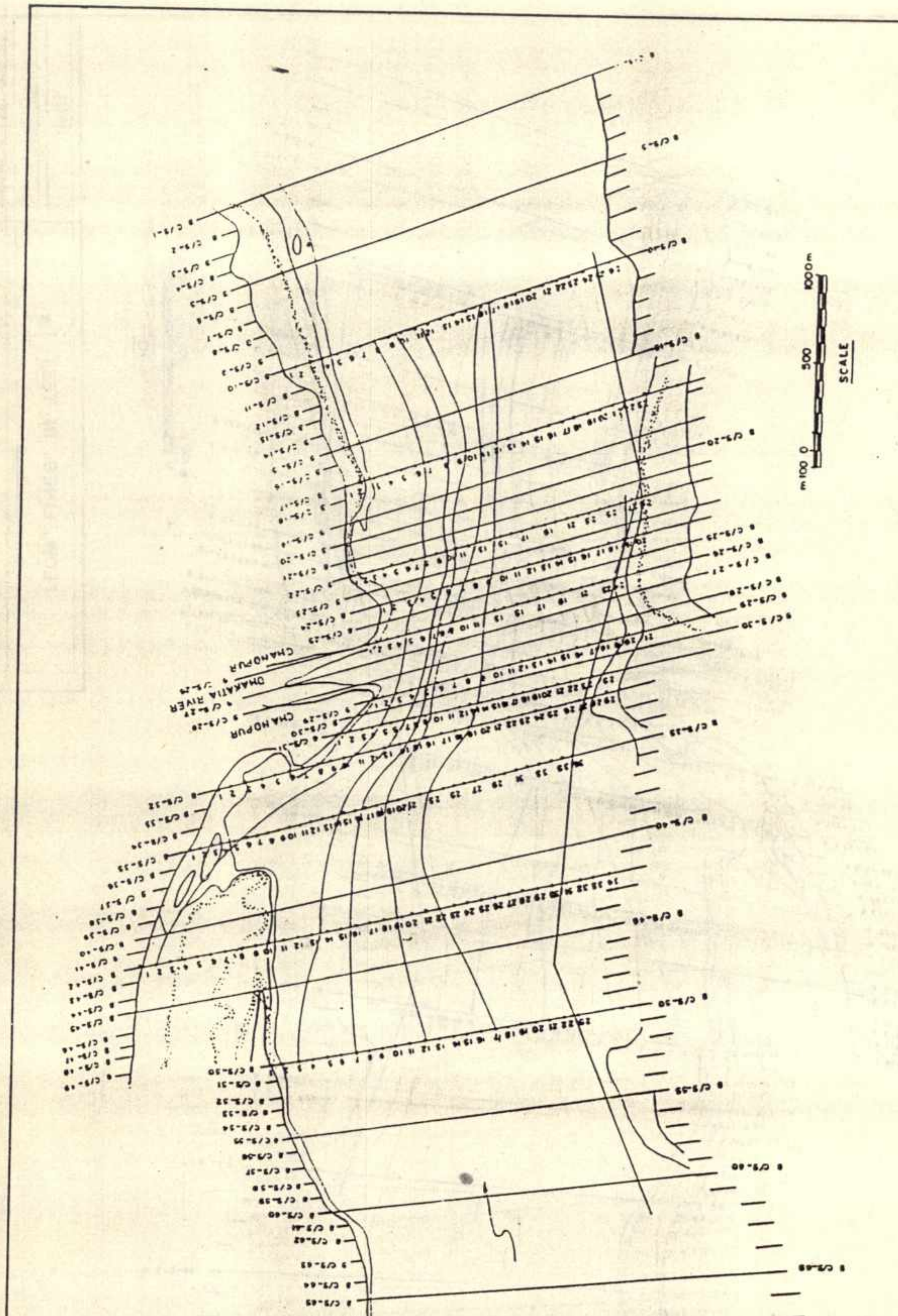


VELOCITY FIELD AT CHANDPUR		DATE
	CHANDPUR	
	MRBPSTS	FIG. D.4.20


FLOW LINES IN TEST T2
CHANDPUR
DATE: 05-09-91
MRBPSTS
FIG. D. 4.21
SCALE
m 100 0 500 1000 m
CHANDPUR
DAKATIA RIVER
CHANDPUR



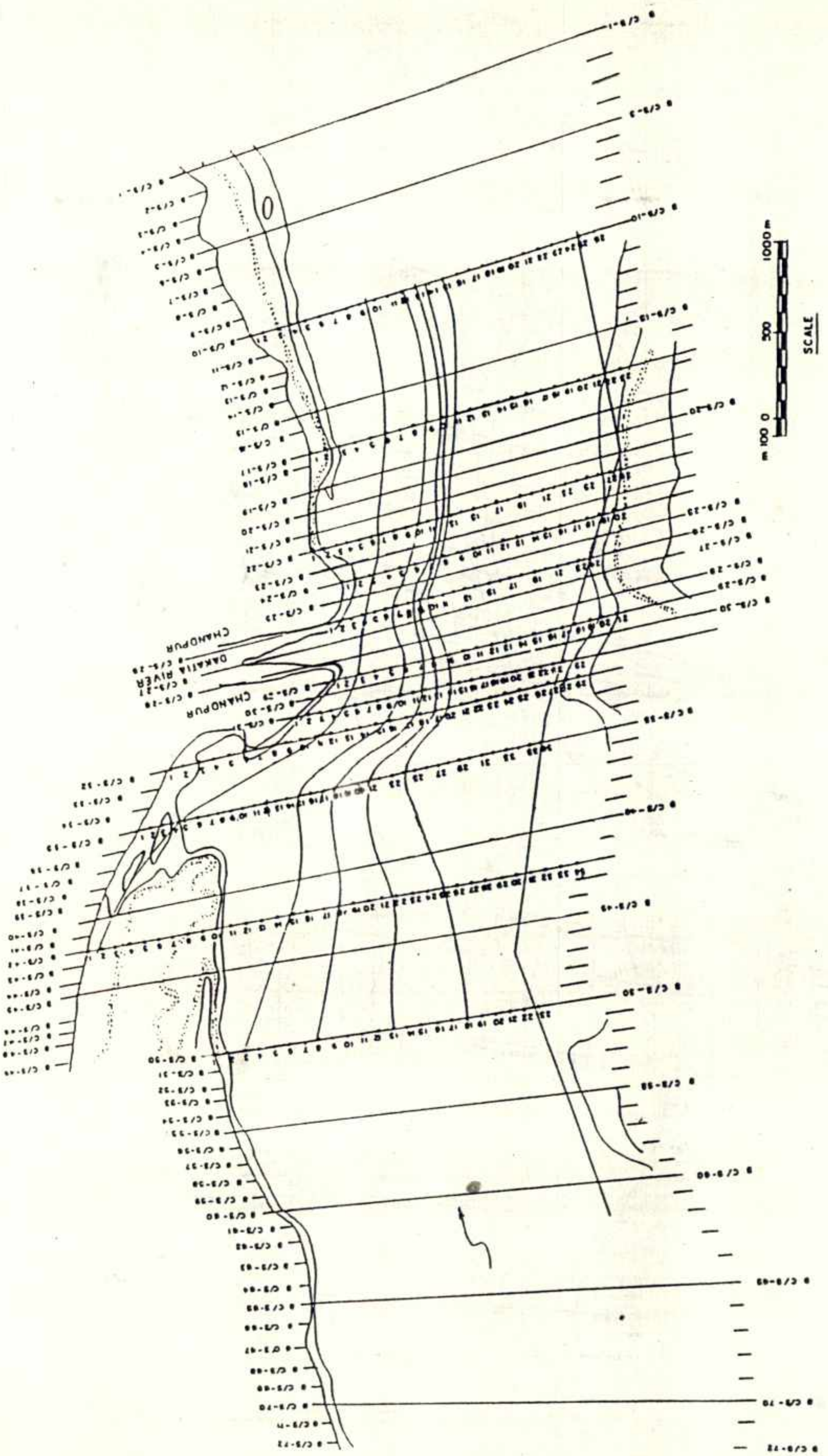
FLOW LINES IN TEST T_1	DATE
	CHANDPUR
	MRBPSTS | FIG. D.4.22



FLOW LINES IN TEST T_2

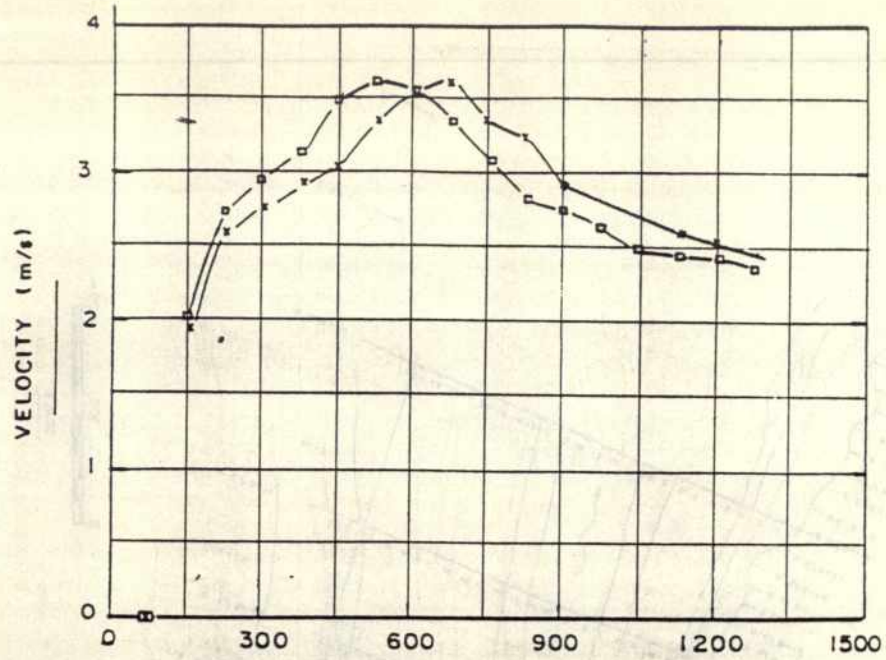
MRBPSTS | CHANDPUR | DATE 18-07-91 | FIG. D.4.23

Ch



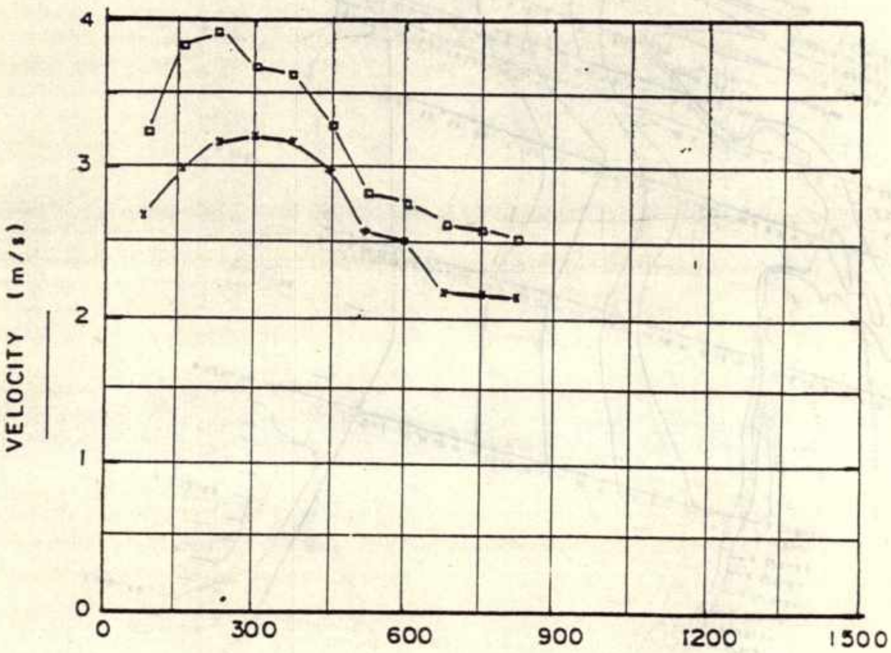
FLOW LINES IN TEST T_3

	DATE:
CHANDPUR	
MRBPSTS	FIG. D.4.24



□ Calibration (T_2)
x Groyne (T_9)

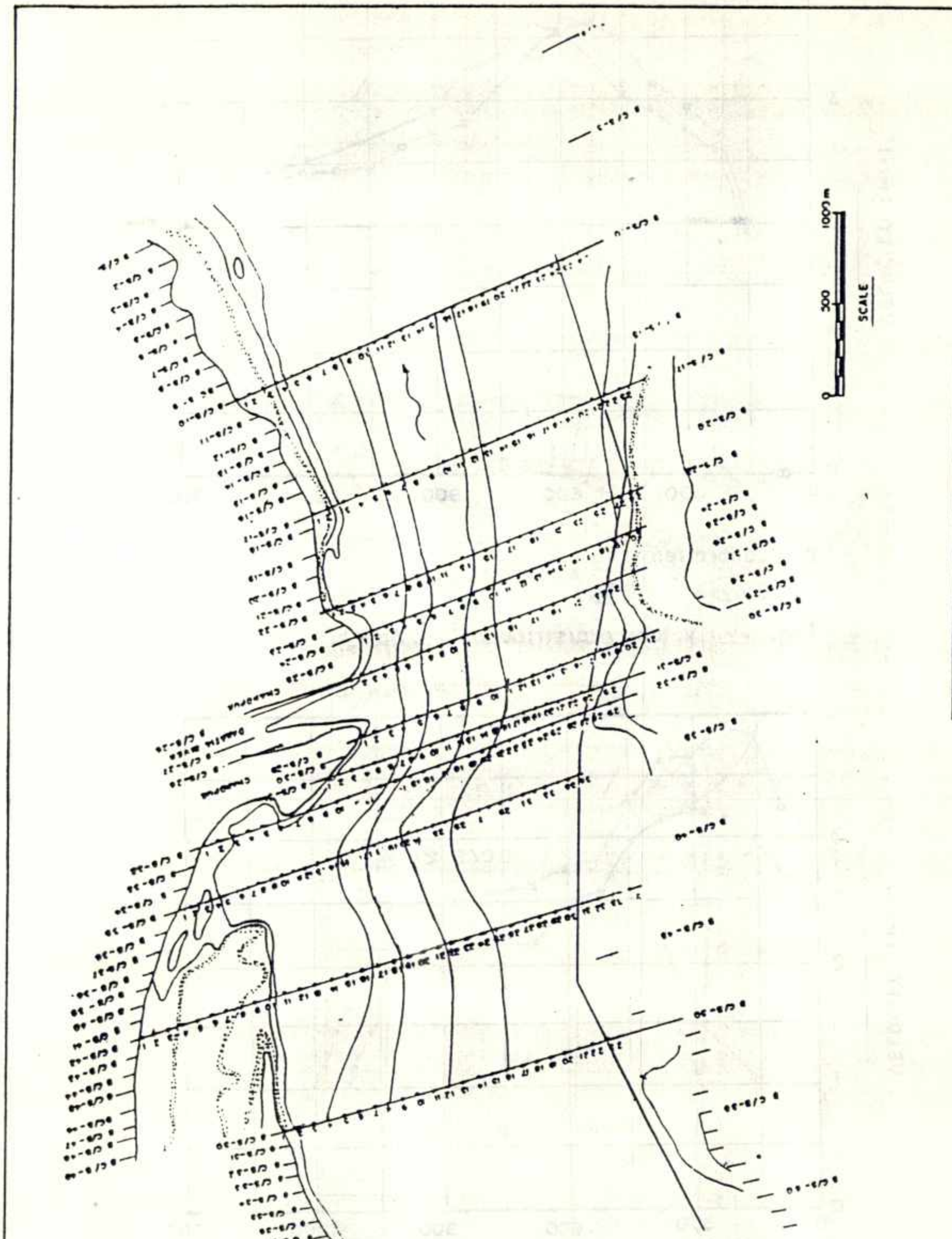
Fig. Velocity distribution C/S-17



□ Calibration (T_2)
x Groyne (T_9)

Velocity distribution C/S-29

FLOW VELOCITY DISTRIBUTION IN CROSS SECTIONS 17 AND 29 IN T_2 AND T_9	CHANDPUR	
	MRBPSTS	FIG. D.4.25

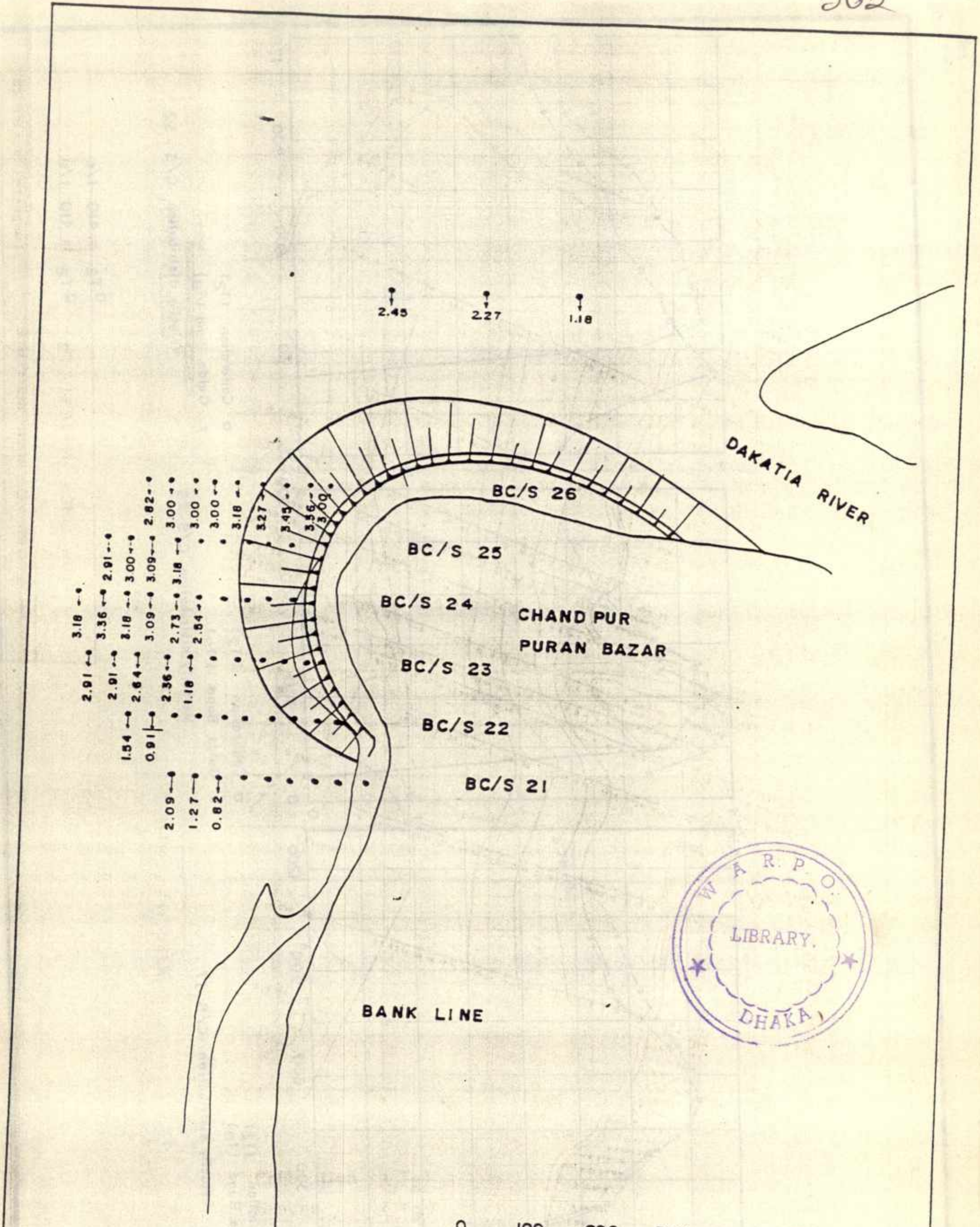


FLOW LINES IN TEST T9

RIVER RESEARCH INSTITUTE

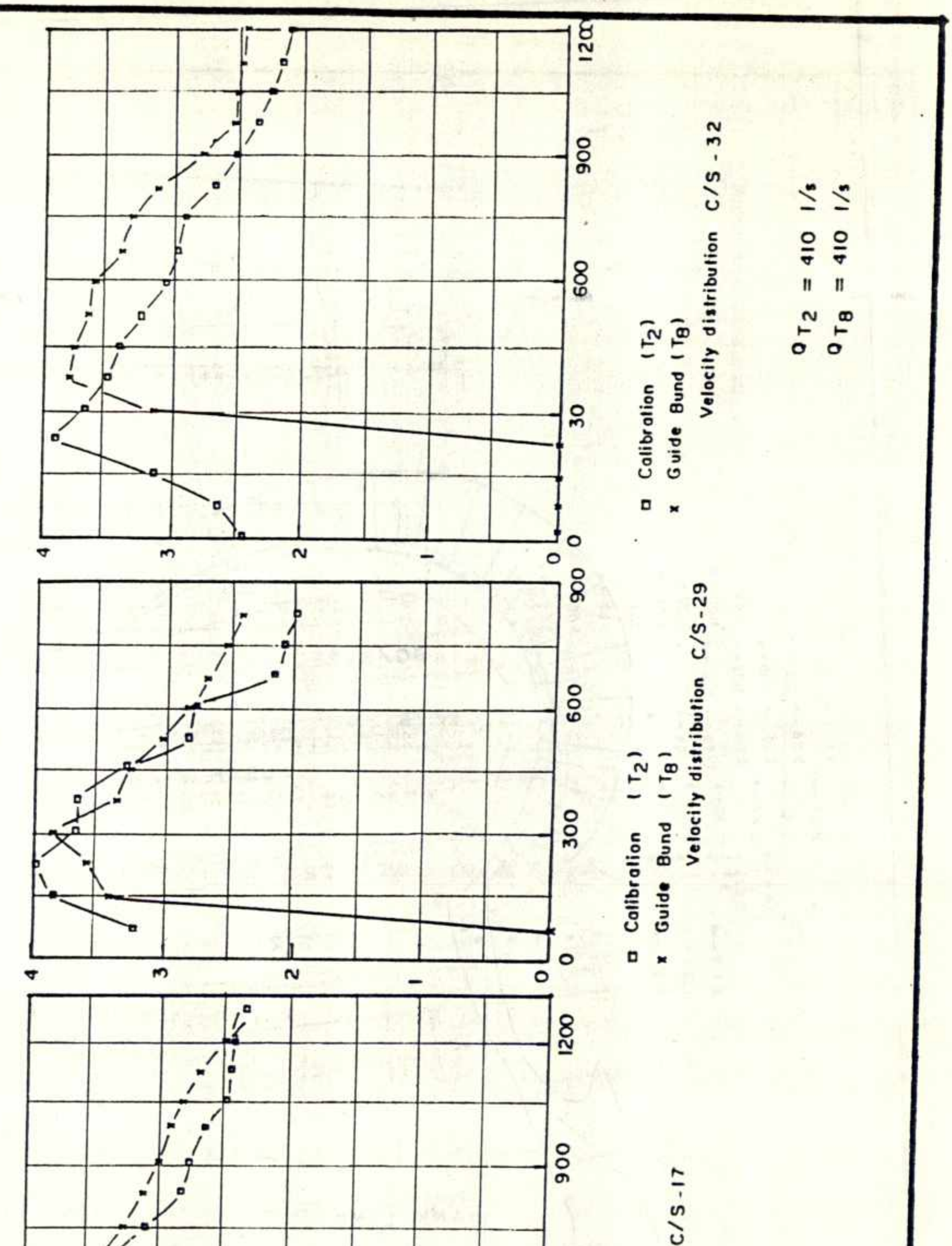
CHANDPUR

MRBPSTS FIG. D.4.26



FLOW VELOCITIES AROUND PURAN BAZAR TEST T11		DATE.
	CHANDPUR	
	MRBPSTS	FIG.D.4.27

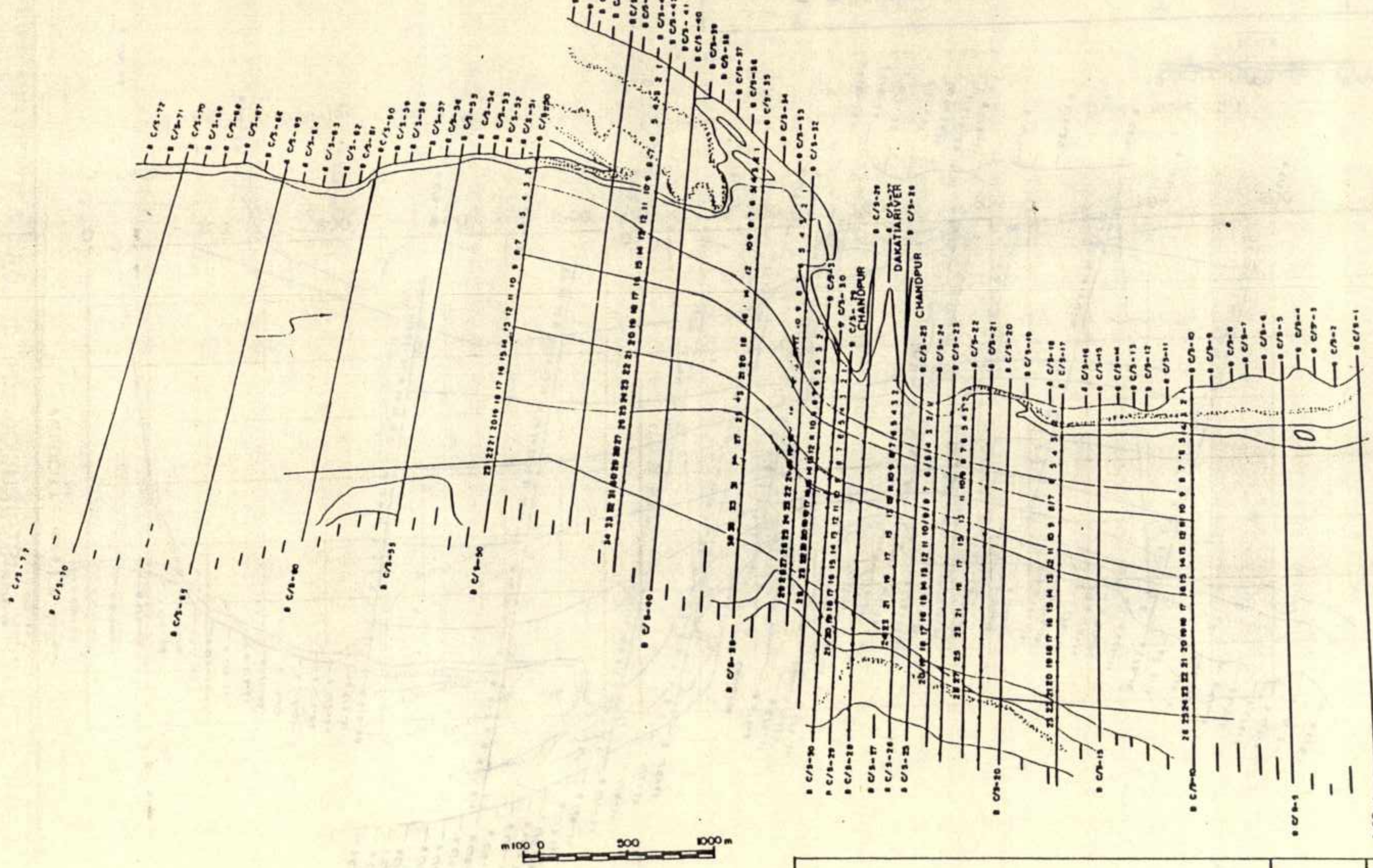
DG6



FLOW VELOCITY DISTRIBUTION IN CROSS SECTIONS 17, 29, 32 IN T_2 AND T_B

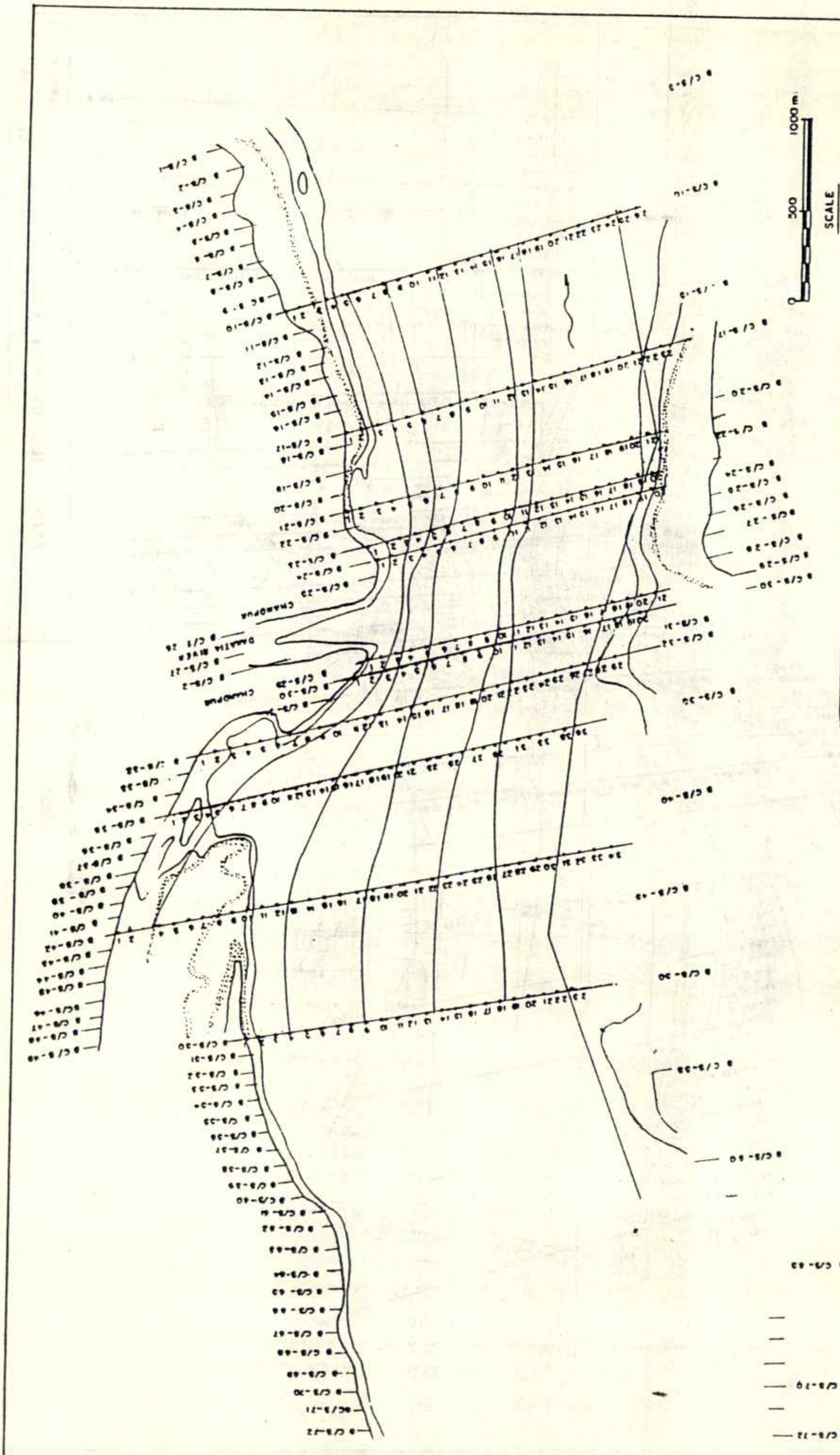
CHANDPUR

MRBPSTS | FIG. D.4.28



FLOW LINES IN TEST T_8

	DATE - 19-09-91
CHANDPUR	
MRBPSTS	FIG. D.4.29



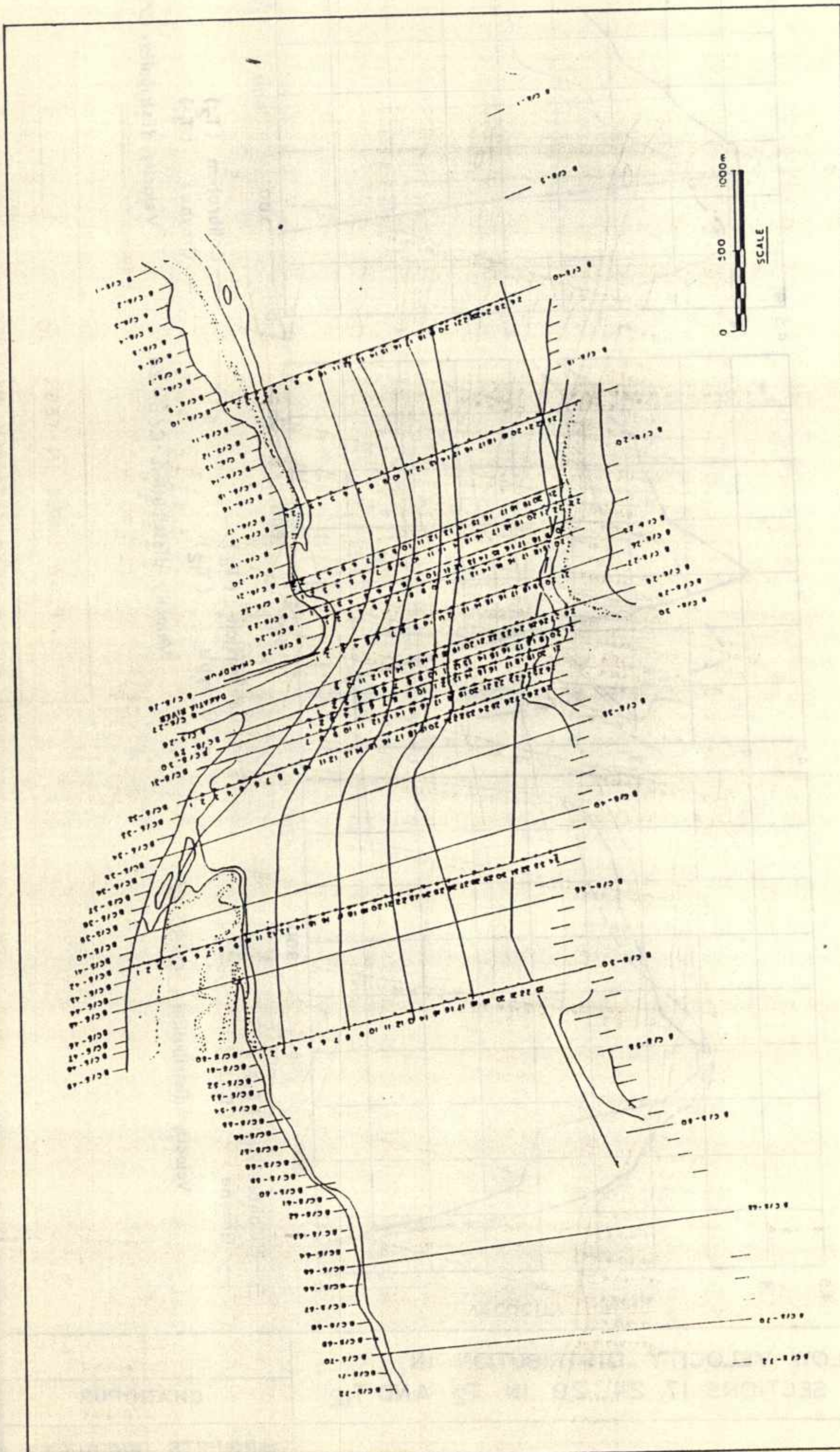
FLOW LINES IN TEST T_{10}

CHANDPUR

RIVER RESEARCH INSTITUTE

MRBPSTS

FIG. D.4.30



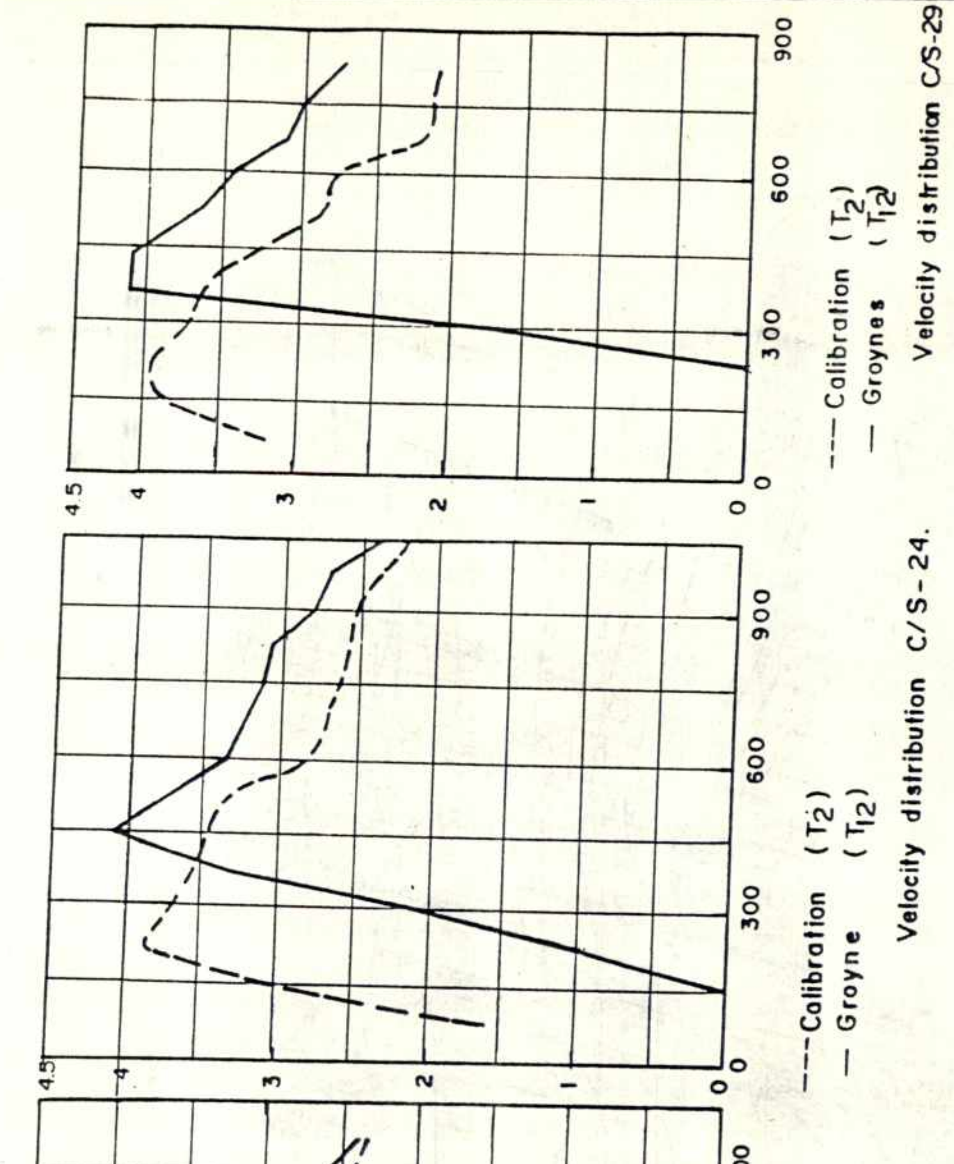
FLOW LINES IN TEST T_{II}

RIVER RESEARCH INSTITUTE

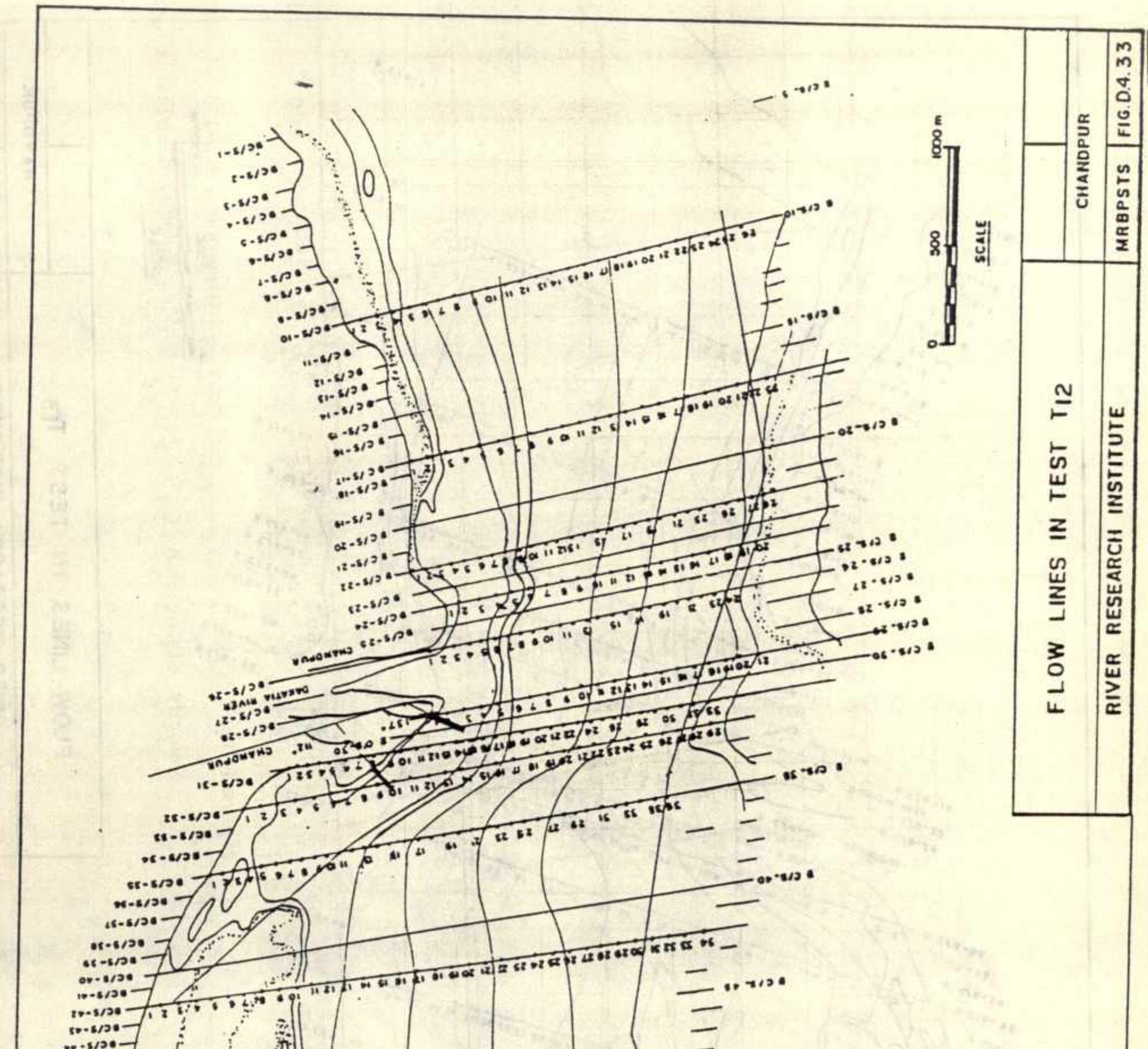
CHANDPUR

MRBPSTS | FIG. D.4.31

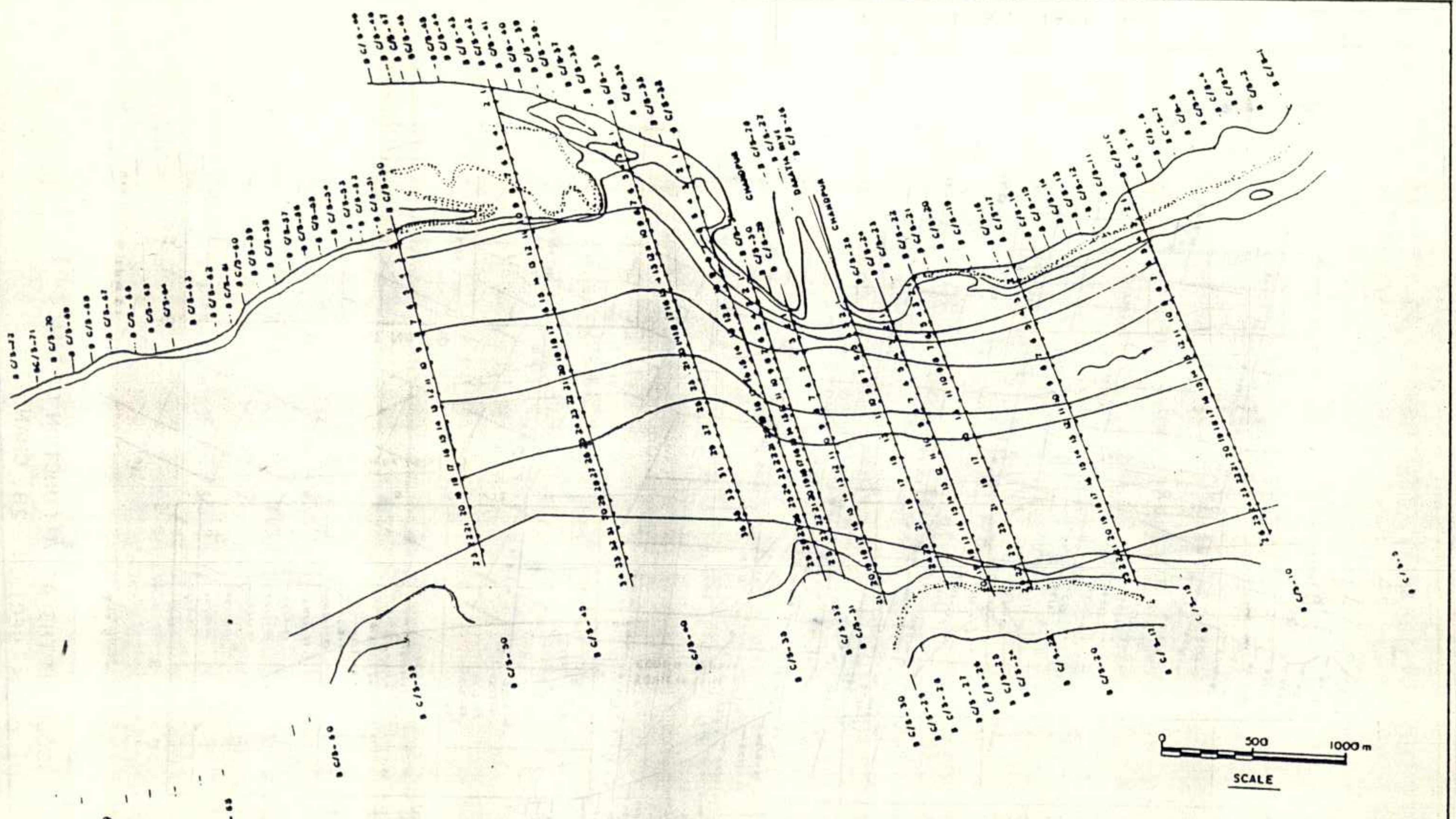
269



FLOW VELOCITY DISTRIBUTION IN CROSS SECTIONS 17, 24, 29 IN T_2 AND T_{12}	CHANDPUR	
	MRBPSTS	FIG. D.4.32

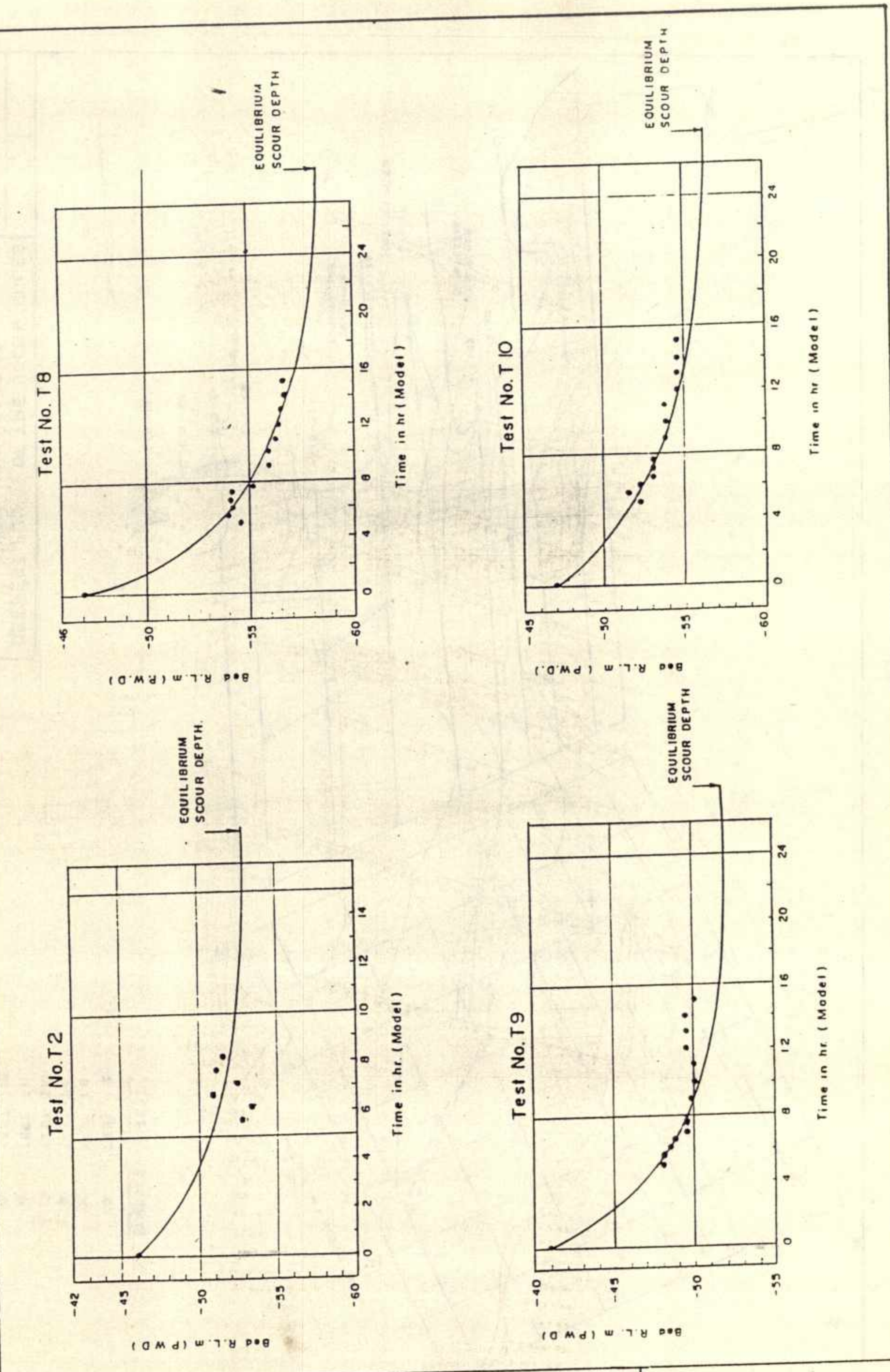


FLOW LINES IN TEST T12
RIVER RESEARCH INSTITUTE
CHANDPUR
MRBPSTS
FIG. D4.33
SCALE
0
500
1000 m
CHANDPUR
DAKATIA RIVER



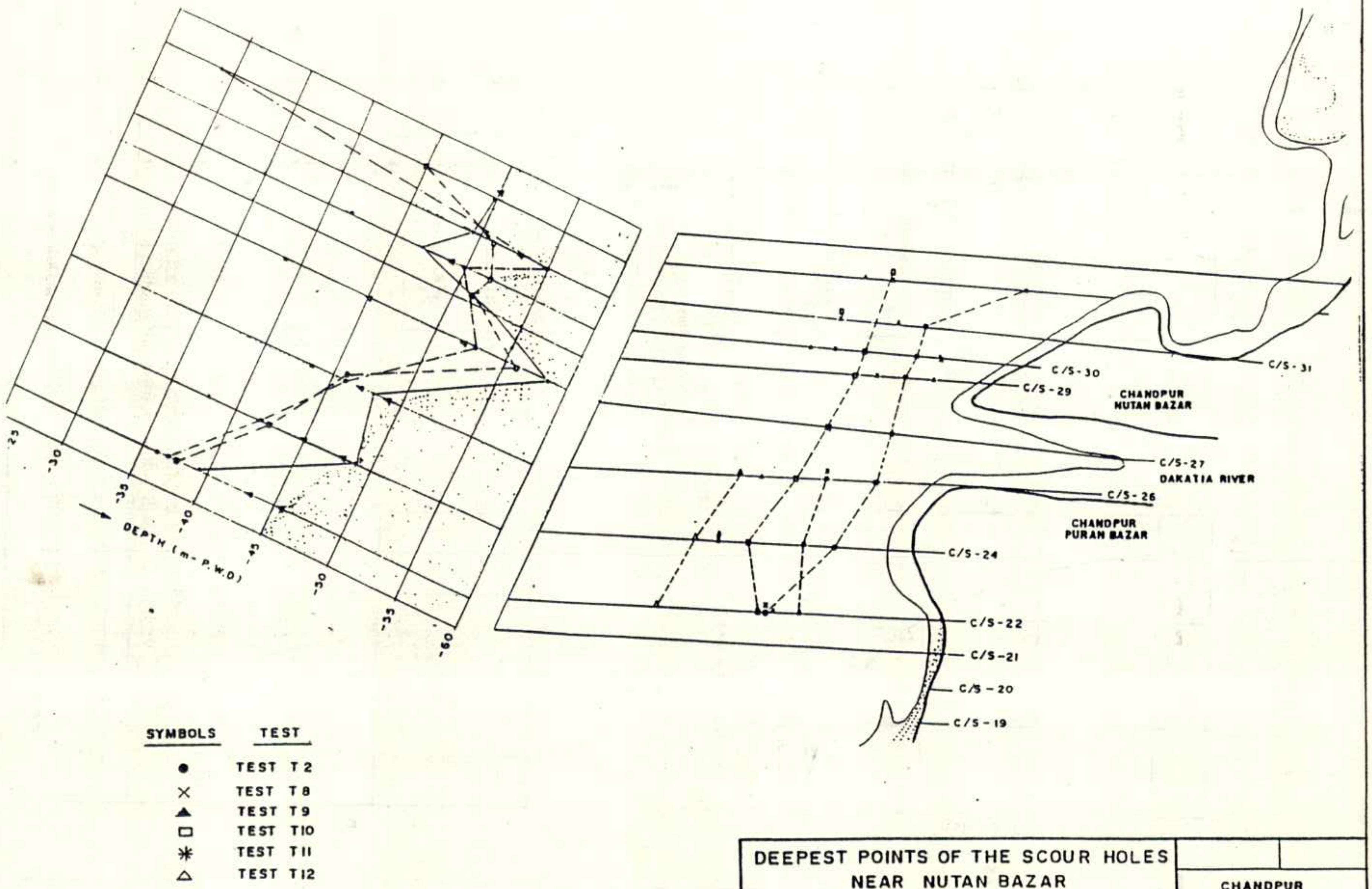
FLOW LINES IN TEST T_{13}	CHANDPUR	
RIVER RESEARCH INSTITUTE.	MRBPSTS	FIG. D.4.34

280



SCOUR DEPTHS AS A FUNCTION OF TIME IN CROSS SECTION - 2B, CHANDPUR

FIGURE D. 4. 35



DEEPEST POINTS OF THE SCOUR HOLES NEAR NUTAN BAZAR

CHANDPUR

MRBPSTS | FIG D.4.36

282

D.5 EKLASHPUR MODEL

Contents

TABLES

DV

D.5 EKLASHPUR MODEL

D.5.1 Introduction

Within the frame work of the Meghna River Bank Protection Short-Term Study a scale model was constructed of a part of the Lower Meghna River near Eklashpur, and some aspects of the proposed bank protection works were studied in that model. The upper boundary of the scale model is the confluence of the Padma River with the Upper Meghna River and the downstream boundary is a few kilometres downstream of Eklashpur. More specifically, the purpose of the model investigation is (i) to study the effect of a retreating bank on a bank protection along the Lower Meghna River and (ii) to test a solution with a groyne. In the inception phase of this project it was planned to study also the effect of a retreating bank upstream and downstream of a existing bank protection, more specifically of the bank protection, which has been designed by BWDB. During the project the priority of this aspect has been lowered, and it was not included in the test program. Instead the effect of a retreating bank including the existing bank protection was tested.

In this local scour model the development of local scour and the flow velocities near the bank protection works and the existing bank were studied in detail.

From this model investigation the following aspects are treated in this Annex D, Chapter D.5: the prototype data, which have been used in the scale model (Section D.5.2), the model set-up (Section D.5.3), a description of the test program (Section D.5.4), the measurements during the model tests (Section D.5.5), the interpretation of the test results including the flow velocities, the flow lines and the scour depths (Section D.5.6). Finally, a summary with the main conclusions can be found in Section D.5.7.

D.5.2 Prototype data

The data for the bed geometry of the model, were obtained from the bathymetric survey in February 1991. The water level and the total river discharge were determined as a function of the recurrence interval in the Hydrologic Study (Annex A) of this project. The data of the flood survey in August and the beginning of September 1991 were used for the calibration of the flow velocities. These data were used in a tentative manner, because a complete report with a description of the survey was not yet available.

The location of Eklashpur is just downstream of the confluence of the Upper Meghna River with the Padma River and downstream of this confluence the river is called Lower Meghna River. The discharge of the Lower Meghna River varies between 10,000 and 130,000 m3/s, whereby about 60,000 m3/s corresponds to bankfull discharge.

Some characteristic discharges and water levels at Eklashpur are presented in Table D.5.1, these data are obtained from the aforementioned Hydrologic Study.

TABLE D.5.1 CHARACTERISTIC WATER LEVELS AND DISCHARGES JUST DOWNSTREAM OF THE CONFLUENCE

Description	flood level (m + P.W.D.)	discharge (m^3/s)
bankfull discharge	~4.30	60,000
1 in 2 years	5.60	~ 90,000
1 in 25 years	5.80	120,000
1 in 100 years	6.20	130,000

The average slope of the river is around 2 cm per km. Some tidal effects are observed in Eklashpur which has a tidal range of about maximum 0.8 m during the flood season and about maximum 1.5 m during the season with low-discharges. These tidal ranges also have some influence on the local flow field near Eklashpur. The bed roughness of the Lower Meghna River was analyzed in the Mathematical Model Study (Annex E). It turned out that no detailed data are available about the bed roughness of this part of the river system.

The bed material in the Lower Meghna River is more or less uniform sand with some silt and a trace of mica. The characteristic diameters are determined from the average of about 7 samples, which have been taken during the survey in February: D_{50} = about 0.09 mm, D_{10} = 0.015 mm, D_{90} = 0.20 mm.

The area, which is to be represented in the model is about 5 km long and 1.5 km wide, this means a partial reproduction of the total width of the Lower Meghna River only, see Figure 5.1.

D.5.3 Model set-up

The planform of the Lower Meghna River is characterised by a fairly straight river alignment with a braided channel and char pattern, the height of the banks in Eklashpur varying from about 5 to 6 m + P.W.D. During the 1 in 100 year flood these banks are flooded. In the model the banks are raised to a higher level to operate the model more easily, therefore in the model the flooding of the bank is not represented, see Photograph D.5.1. This will not affect the results of this study too much.

For the verification of the model some results of the field survey in February 1991 were used. During this field survey the discharge was low, therefore also the measured flow velocities were relatively low, and only the flow directions have been used for the calibration of the scale model. For the calibration of the flow velocities the results of the Mathematical Model Study (Annex E) and the flood survey in August and September 1991 were used.

The scale model has four upstream inflow boundaries, which represent the different channels at the confluence, and one outflow boundary. In the first tests, in which the bed geometry of February 1991 was simulated, only three of these inflow boundaries were used. In the tests with a worst case all four inflow boundaries were in operation, see Figure 5.2.

In the steady flow model of Eklashpur a constant discharge was pumped around. The influence of the tidal fluctuations on the scour process has been neglected, as is usual in this type of models, because of the conflicting time scales of the tidal flow and the scour process. The available head difference between the inflow and the outflow of the model is too small for a flexible operation of the model. Since in the model the initiation of motion of the bed material is observed locally, no facilities for the circulation of the sediment are required.

The flow field in the model is guided largely by the banks and the model boundaries. Therefore a local deviation from the roughness condition for the reproduction of a flow field can be accepted. The discharge distribution along the upstream and the downstream flow boundaries of the scale model were derived from the Mathematical Model Study.

The grain size distribution of the sand in the model was determined from only three samples, see Figure 5.3. The bed material of the scale model has the following characteristic diameters: D_{50} = 0.16 mm, D_{10} = 0.08 mm and D_{90} = 0.30 mm. In the prototype situation five to 10% of the samples has a diameter less than 0.074 mm. For a local scour model it is rather particular that the grain size distribution of the model sand is coarser than the prototype sand (see Section D.5.2 prototype data). The scale laws require that the size of the model sand should be finer than the prototype sand. This scale effect is compensated by increasing the model discharge.

284

D.5.4 Test program

In the scale model different situations for two types of solutions for the bank erosion were tested: a revetment and a groyne. These solutions were tested in the situation of the channel pattern in the Lower Meghna River of February 1991 and in a worst case situation. The worst case situation was defined in the Geomorphologic Study (Annex B) of this project. In brief the worst case situation can be described by a shifting of the existing bank because of severe bank erosion and by a certain channel geometry, which has some similarity with the channel geometry in 1988. In the Hydrologic Study (Annex A) of this project the discharges of the Padma River and the Meghna River were analyzed. This analysis showed that different combinations of these discharges can probably be decisive in a worst case situation and therefore two extreme situations were tested in the model, see Table D.5.10 (placed at end of Chapter).

In the two calibration tests the model was run with two different discharges:

(i) a low discharge according to the Froude law to reproduce the flow field and to prevent considerable sediment transport in the model (this to keep the model bed smooth and the Chezy coefficient as high as possible).

(ii) a higher discharge as required for the reproduction of the local scour process.

The flow fields in both calibration tests have been compared in order to check if a higher model discharge does not induce unacceptable changes in the flow field.

Near the village of Eklashpur over a distance of about 600 m a revetment has been designed by BWDB. This revetment is under construction now and it has been built in the model in test T3, (see lay-out in Figure 5.4 and a representative cross section in Figure 5.5).

As an alternative solution an upstream groyne was tested in T6. This groyne has a length of 700 m, (see lay-out in Figure 5.6 and a typical cross section in Figure 5.7). In the subsequent tests T10 to T13 the alignment was changed a little and the length was reduced to 600 m, (see Figure 5.8).

The other alternative solution is a continuous revetment for which a typical cross section is shown in Figure 5.9.

A review of all the model tests is given in Table D.5.2.

TABLE D.5.2 TEST PROGRAM FOR BANK PROTECTION WORKS AT EKLASHPUR

Test	description	bathymetry	bed motion	discharge $Q_M/(Q_P+Q_M)$
T1	existing bank, calibration	1991	flat bed	0.29
T2	existing bank, calibration	1991	local scour	0.29
T3	existing bank, BWDB-protection	1991	local scour	0.29
T6	existing bank, groyne type 1'	1991	local scour	0.29
T6	existing bank, groyne type 1	1991		0.29
T6	existing bank, calibration	1991		0.29
T6	eroded bank with revetment	worst case	flat bed (Froude)	0.37
T7	eroded bank with revetment	worst case	local scour	0.37
T8	eroded bank with revetment	worst case	flat bed	0.2
T9	eroded bank with revetment	worst case	local scour	0.2
T10	eroded bank with groyne type 2	worst case	flat bed	0.37
T11	eroded bank with groyne type 2	worst case	local scour	0.37
T12	eroded bank with groyne type 2	worst case	flat bed	0.2
T13	eroded bank with groyne type 2	worst case	local scour	0.2

The following explanation of the test program shown in this Table is given:

eroded bank = The part of the embankments, which protect an irrigation project and which are eroded, need a new revetment. In addition also a new revetment around the removed pump station is required, but this is outside the model area and therefore not tested in the model.

worst case = A new bed topography based on the bed topography in 1988, but with the whole channel having shifted to the eroded bank.

285

flat bed = The flow velocities are scaled according to the Froude condition and no sediment movement will occur in the model.

local scour = The flow velocities in the model are sufficiently high to reproduce the local scour process in the model.

type 1 = length 700 m, the groyne starts in cross section 55 and the angle between the axis and the base line is 34 degrees.

type 2 = length 600 m, the groyne starts in cross section 60 and the angle between the axis and the base line is 54 degrees.

In the model the ratio of the discharges of the Padma River, the Upper Meghna River and the Lower Meghna River can vary between 0.2 and 0.4 during the higher discharges, according to the Hydrological Study (Annex A).

D.5.5 Measurements

D.5.5.1 General

In the course of a complete test the discharge, the water level, the flow velocities, the flow lines and the scour depths were measured. The water temperature was measured during each test. The photographs of the flow field as required in the Terms of Reference for the model investigations at RRI were made as an experiment, but the results can be improved in next studies. Some photographs of the dry scour hole were taken after a test was finished and the model was drained, see Photographs D.5.2 to D.5.4.

D.5.5.2 Discharge

The model discharge has been calculated from the water level upstream of a Rehbock-weir (rectangular weir with a standard shaped crest with a brass strip, however in the model corrugated weirs were applied with an irregular crest). The upstream water level was substituted in the standard formula for this type of weirs to determine the model discharge, provided the upstream flow velocity can be neglected:

$$Q = 1.86 \;.\; b \;.\; h_w^{1.5}$$

in which Q = discharge (m3/s), b = width of the weir (m), and h_w = vertical distance between the crest of the weir and the upstream water level (m).

However, in the four measuring weirs in the model the upstream flow velocity can not be neglected and, consequently, a more detailed formula has to be applied:

$$Q = (0.407 + 0.053 \;.\; \frac{h_w}{p}).\sqrt{2.g}.b.[(h_w + \frac{u_w}{2.g})^{1.5} - (\frac{u_w}{2.g})^{1.5}]$$

in which u_w = the average flow velocity in the cross section in which h_w has been measured (m/s), p = the height of the crest of the weir above the bottom (m), and g = acceleration by gravity (m/s^2).

No calibration formula has been determined for each weir separately as was requested in the Terms of Reference for the model investigations. The supply of free air under the falling jet should be improved. One of the sources of inaccuracy in the determination of the discharge is the corroded crest of the weir, which could induce a inaccuracy of 5 to 7 % if the discharge is low. The final accuracy of the determined discharge is therefore probably not as high as required.

D.5.5.3 Water level

In the model the water level can be regulated by the tailgates. After the calibration test the position of the gates should be kept approximately constant during all tests. The position of each tailgate relative to the other tailgates is important mainly for the distribution of the discharge in a cross section just upstream of the weirs. The average water level is checked with the four point gauges and is well controlled.

D.5.5.4 Flow velocity

The time averaged flow velocities have been measured by an Ott flow propeller having a diameter of about 0.03 m. The measurement time for counting the revolutions of the propeller was reduced from 1 minute to 0.5 minute, and this has resulted in an accuracy of + or - 0.025 m/s in the prototype values.

The flow velocities have been measured in the complete cross sections 20, 25, 30, 35, 40, 45, 50, 55 and 62, see Figure 5.10. In these cross sections the measuring points were selected at an interval of 150 m and at half the water depth. For more detailed information about the flow velocities near the left bank the flow velocities were measured in cross sections 32, 37, 42, 47, 52, 57, 62 and 66 from 0 to 750 m from the left bank and perpendicular to that bank, and at an interval of 75 m, (see Figures 5.11 and 5.12). When a groyne was tested, the flow velocities were also measured around the head of the groyne.

In the Terms of Reference for this model investigation a calibrated flow measuring instrument, which also could measure the turbulent fluctuations, was requested. The Ott flow propeller was composed of parts of different Ott propellers and the calibration was not repeated.

D.5.5.5 Flow direction

The flow direction has been measured with flow lines, which have been determined by floats. These floats were made from small bottles, which float with a depth of approximately 0.04 m in the water. The position of the float was determined at the moment the float passes the line of a cross section. The accuracy of this method to locate the float was about 0.05 m.

D.5.5.6 Scour depths

In the model the scour depths were measured by a rod. The depth was read from the position of the rod relative to a reference line. This line was also used to fix the location relative to the bank. With this simple method a reasonable accuracy could be maintained. The scour depths were measured in short cross sections 32, 37, 42, 47, 52, 57, 62 and 66, (see Figure 5.11), and at an interval of 0.5 hour between 4 and 8 hours, sometimes extended to 16 hours after the start of the test. When a groyne was tested also the scour depths were measured around the head of the groyne.

From these measurements and the initial bed level the equilibrium maximum scour depth was estimated by graphical extrapolation. This procedure was applied to the deepest point in the cross section of the initial scour hole and to the point in that cross section, where the maximum increase in scour depth was determined. This procedure was followed because the location of the deepest point in a cross section can shift during a test.

Some experimental photographs from the dry scour hole with contour lines with constant depth were taken from an about 5 m high tower but this height made it impossible to take these photographs from this wide model without oblique.

As an additional measurement the temperature of the water was measured in each test. The average temperature was about 30 degrees Celsius.

D.5.6 Interpretation of the results

D.5.6.1 Introduction

The interpretation of the results of the tests in the model is focused on: i) the maximum flow velocities along the existing banks and along the continuous revetment and the groyne for the design of the toplayer of the protection works of these structures, and ii) the maximum scour depths especially near the revetment and the groyne in view of their importance for the design level of toe of the protection and/or length of the falling apron.

It should be pointed out that in this model a falling apron was modelled from small gravel in some tests. The hydraulic gradient (Section D.5.6.2), the flow velocities and the flow lines (section 5.6.3) and the scour depths (Section D.5.6.4) are the main items dealt with in the interpretation of the test results.

D.5.6.2 Hydraulic gradient

The hydraulic gradient has been determined from the water levels which are measured in the 4 point gauges in the model, and the distances between these point gauges. These distances should be measured along a flow line. The determined waterlevel gradients in tests T1 to T6 are presented in Table D.5.11 and from this Table it is concluded that the variation in the water level slope is more than expected. Since for this variation no physical explanation can be found, it is assumed that some of these gauges were partially blocked during most of the tests. Probably point gauge number 3 was rather accurate, but point gauge number 4 (the most downstream one) has been blocked most probably up to test T11. About point gauges 1 and 2 no specific information is available. Therefore these results were not used for a more detailed analysis, and the calculation of the Chezy coefficient, which is presented in Table D.5.12, can be considered as tentative, and as an illustration of the method to calculate this coefficient.

D.5.6.3 Flow velocities

D.5.6.3.1 General

In this section the flow velocities are treated together with the flow lines. For the flow velocity near the bank a representative flow velocity has been defined and subsequently the flow velocities in the different alternative solutions, a groyne and a continuous revetment, are described.

It is emphasized that in almost each test the scale factor of the flow velocity is different, see Table D.5.10, because of different prototype discharges and different model discharges.

D.5.6.3.2 A representative flow velocity and calibration

For the design of a bank protection the flow velocity near the bank is a decisive parameter. In the model the flow velocities near the bank are relatively too low due the bed roughness and probably also due to some viscosity effects. Therefore, the flow velocity had to be extrapolated from the flow velocities, which have been measured more in the middle of the model. In Figure 5.13 it can be seen that at 225 m from the bank the flow velocity is not influenced by these viscosity effects. In that figure the tendency in the flow velocities at 170, 270 and 370 m from the bank has been extrapolated and a representative flow velocity for the hydraulic load by the current on the bank, can be defined as 75 % of the flow velocity at 225 m from the bank.

The comparison from the representative flow velocities along the bank in test T2 and test T3 with the flow velocities, which were measured in the flood survey is as follows (Table D.5.3):

TABLE D.5.3 FLOW VELOCITIES ALONG THE BANK IN TESTS T2 AND T3, AND IN THE FLOOD SURVEY

cross section	representative velocity T2	T3	flood survey
-	m/s	m/s	m/s
17			1.1
20	0.87	0.90	
22			1.4
30	0.90	1.10	
32 / 33	1.07	1.09	1.1
35	1.07	1.09	
37	0.93	0.96	
40	0.62	0,70	
42 / 43	0.86	0.86	0.68
45	0.77	0.57	
47	0.89	0.67	
50	0.26	<0.2	
52 / 53	0.69	0.54	0.6
55	0.99	0.79	
57 / 58	1.21	1.37	0.78
63			1.0

In tests T2 and T3 the differences between the flow fields are small, because the influence of the BWDB protection on the flow field can be neglected. In the Table it can be seen that the representative flow velocities are almost the same downstream of cross section 45. Upstream of this cross section the differences are not negligible, but still a fair reproduction has been obtained.

It should be pointed out that in the short cross sections perpendicular to the bank (32, 37, 42, etc.) the flow velocities have been measured at 0.8 of the water depth and all other flow velocities at 0.5 of the water depth. By comparing the data the following relation has been adopted: $u_{0.8\,h} = 1.12\ u_{0.5\,h}$. By applying this relationship all the flow velocities are presented at 0.5 h in Table D.5.3.

When comparing the flow velocities measured in August-September 1991 along the bank of Eklashpur with those of tests T2 and T3 (Table D.5.3) one should bear in mind a noticeable tidal influence in the prototype on the direction and the size of the flow velocities. Therefore, for this calibration only the flow velocities in the main direction of the river flow are used. It can be concluded that:

- the general tendency in the flow velocities along the bank are well reproduced in the model,
- the small differences in the flow velocities are acceptable, and
- the representative flow velocity seems to be a useful parameter.

From the flow lines in test T2, Figure 5.15, it can be seen that the flow attaches the bank between cross sections 45 and 40. The maximum representative flow velocities have been measured between cross sections 40 and 30.

The influence of increase of the model discharge on the flow lines can be seen by comparing the flow lines in test T1, (see Figure 5.14), with the flow lines in test T2, (see Figure 5.15). A careful comparison shows that in T2 the radius of the curvatures of the flow lines is smaller than in test T1. This was caused by the increase in the bed roughness which has been generated by the higher discharge. The flow lines in test T1 are more accurate, and the difference with the flow lines in T2 is considered as an acceptable scale effect. However it is a reason to carry out different tests for reproducing the flow velocities and for reproducing the local scour processes in the "worse case" tests of the test program.

D.5.6.3.3 BWDB protection

The flow velocity distribution in cross section 35 in the calibration test T2 and in the test with the BWDB protection, T3, shows a good similarity (as had been expected, see Figure 5.16). It should be mentioned that the sharp dip in the flow velocity distribution in that graph has been caused by model bridge piers. In the first tests these piers had an angular cross section, in later tests these piers have been replaced by piers having a circular cross section. These circular piers cause less disturbance of the local flow field.

The BWDB protection has been designed between cross sections 44 and 38, at the upstream side of the measured maximum representative flow velocities. It should, however, be realized that the situation of February 1991 is probably not decisive for the bank erosion of recent years.

D.5.6.3.4 Submerged groyne

In the tests with a groyne, T10 and T12, its combination with a maximum discharge from the Upper Meghna River, results for T10 in very high flow velocities around the head of the groyne and this concentration of the discharge is still recognisable in cross section 35 as a high peak in the flow velocity distribution at 600 m from the bank, see Figure 5.16. In Figure 5.17 with the flow lines of test T10 this concentration of the discharge results in a small distance between the flow lines from cross section 40 to cross section 30. This is confirmed by comparing the flow lines in test T10 with the flow lines in test T12, (see Figure 5.18).

Along the left bank the following representative flow velocities have been determined as the maximum from test T10 Q_M/Q_P = 0.37 and test T12 Q_M/Q_P = 0.2, see Table D.5.13 for all the representative velocities in tests T9 to T13.

TABLE D.5.4 THE REPRESENTATIVE FLOW VELOCITIES ALONG THE LEFT BANK, IN WORST CASE SITUATION

cross section -	representative velocity m/s
20	1.3
30	1.9
35	1.7
40	0.8
45	0.5
50	0.5
55	1.2
60	1.2

If $Q_M/(Q_P+Q_M)$ = 0.2 than the flow velocities along the bank are about the same as in the test with $Q_M/(Q_P+Q_M)$ = 0.37 in case of the alternative with the groyne.

Along the upstream side of the body of the submerged groyne and near the head of the groyne the following flow velocities are calculated, see Figure 5.19:

	$Q_M/(Q_P+Q_M)$ = 0.2	$Q_M/(Q_P+Q_M)$ = 0.37
upstream side	u = 1.5	2.1 m/s
head of the groyne	u = 2.5	2.8 m/s

The flow velocity along the downstream side of the body of the groyne is small. In the tests carried out with a higher discharge to reproduce the scour processes at scale (T11 and T13) the same or lower flow velocities have been measured than in T10 and T12.

D.5.6.3.5 Continuous revetment

For the design of the revetment from cross section 20 to cross section 65 the maximum representative flow velocity has been determined from tests T6 and T8. Although in test T6 the discharge is higher than the discharge in T8 (75,000 versus 60,000 m3/s), it is observed that the flow velocities between cross sections 52 and 40 in T8 are decisive. This effect was expected when during the preparation of the test program different combinations of discharges were selected.

TABLE D.5.5 THE REPRESENTATIVE FLOW VELOCITIES ALONG THE LEFT BANK, IN WORST CASE SITUATION, REVETMENT

cross section	representative velocity m/s
20	... *
30	1.7
35	1.6
40	1.2
45	1.0
50	1.2
55	1.8
60	1.9
62	1.3

* unreliable result

The maximum representative flow velocity along the revetment is 1.9 m/s in the sharp bend of the revetment (Table D.5.5).

A maximum discharge from the Upper Meghna River, in test T6, pushes the flow lines about 300 m more from the bank into the Lower Meghna River than in test T8 with the maximum discharge of the Padma River, see Figures 5.20 and 5.21.

A comparison of the revetment tests with the tests with a groyne shows that a groyne reduces the flow velocities along the bank about 0.5 m/s up to cross section 30; that is 2.5 km downstream of the groyne or about 4 times the length of the groyne.

D.5.6.4 **Scour**

D.5.6.4.1 General

The scour depths along the bank near Eklashpur are important for the stability of the existing banks and for the design of the toe of a continuous revetment in one of the alternatives. In addition the local scour around the head of the groyne has been measured for the design of the toe level of the protection of the groyne head.

One of the conditions for a reliable reproduction of the scour process in the model is a sufficiently high flow velocity in the model. This flow velocity should exceed twice the critical flow velocity. If this condition is fulfilled, then the size of the scour hole depends largely on the geometry, it therefore has been checked for all tests.

Firstly, the critical flow velocity in the model has been calculated. The critical flow velocity is estimated using the formula for the shear stress parameter for a well developed, open channel flow:

$$\Psi_{cr} = \frac{u_{cr}^2}{C^2.\Delta.D_{50}}$$

in which:

Ψ	=	shear stress parameter	(-)
C	=	Chezy coefficient	($m^{0.5}s^{-1}$)
Δ	=	relative density	(-)
D_{50}	=	diameter of the sand (bed material)	(m)

For the prototype situation the following values have been estimated: Ψ = 0.03, C = 140 m0.5s-1, Δ = 1.65, D_{50} = 0.9 10-4 m and after substitution of these values in the formula the critical flow velocity is calculated:

u_{cr} = 0.29 m/s.

In the model situation the following values have been estimated: psi = 0.03, C = 40 to 50 m0.5s-1, delta = 1.65, D_{50} = 1.6 10-4 m and after substitution of these values in this formula the critical flow velocity is calculated:

u_{cr} = 0.11 to 0.14 m/s.

This means that if the model flow velocity just upstream of the scour hole is more than 0.3 m/s than the scour depths are measured at the depth scale. If less than 0.3 m/s an additional correction should be made. In tests T2, T3, T6, T11 and T13 the average flow velocity is higher than, or equal to 0.3 m/s, therefore these tests have been used to study the local scour process in detail.

An impression of the accuracy of the reproduction of the initial bed level in different tests can be obtained from the bed levels at 225 m from the bank in tests T10 to T13, see for example Tables D.5.17 and D.5.18. Differences between 0.2 and 2 m are measured, with an average of 0.5 m. This is probably due to the rain and the model filling process which can easily damage a good modelled bed in an open air model.

D.5.6.4.2 Calibration test

In the calibration test in the whole model some sedimentation has been measured, because in the inflow section far too much sand had been eroded (maximum erosion depths of 10 to 20 m). After this test calibration the inflow section has been improved by a submerged hydraulic jump in order to guarantee a good spreading of the flow velocities and to prevent this excessive scour. After this improvement had been implemented, no excessive scour was observed.

D.5.6.4.3 BWDB protection

In test T3 with the BWDB protection the same deficiency as reported upon above can be observed (see Table D.5.14 with the bed levels and the scour depths). Therefore these results are not described here in detail.

D.5.6.4.4 Groyne

In the tests T11 and T13 with a groyne type 2 in cross section 60 the scour depths have been measured along the bank and around the head of the groyne, see Photographs D.5.2 to D.5.4. In test T11 a falling apron has been simulated by a layer of small gravel around the groyne and in test T13 this layer has been omitted, because the settlement of the falling apron is not well reproduced in the model and this can result in a slightly under-estimation of the scour depths. The alignment of the groyne has been selected such that the head of the groyne is just upstream of the confluence scour hole. The local scour

hole near the groyne head may coincide with this confluence scour hole.
The increase in the measured scour depths between 5 and 16 hours after the start of the test have been extrapolated to the equilibrium scour depths (see two examples in Figure 5.22). From the extrapolated scour depths around the head of the groyne the following maximum equilibrium depths of the scour hole can be determined:

TABLE D.5.6 DEPTH IN THE SCOUR HOLE AROUND THE HEAD OF THE GROYNE

cross section -	depth level in scour hole m-P.W.D.
A groyne axis	-28
B 150 m d/s	-27
C 250 m d/s	-36
D 350 m d/s	-29

As a design scour depth for the protection of the head of the groyne the deepest point of the scour hole can be selected, e.g. 36 m-P.W.D. The shape of the scour hole becomes probably wider and less deep in the case of presence of a falling apron: compare Photographs D.5.2 and D.5.3 with the Photographs D.5.4 and D.5.5. The different discharge distributions in these tests contribute probably to a lesser extent to the differences in the shape of the scour hole than the falling apron.

Along the bank downstream of the groyne the scour depths are determined at 225 m from the bank, because near to the bank the flow in the model is relatively too low (ref. definition of the representative flow velocity) and this reduces the scour depths near the bank.

TABLE D.5.7 THE BED LEVEL AT 225 M FROM THE BANK, WORST CASE, GROYNE

cross section -	depth level in scour hole m-P.W.D.
32	-20
37	-23
42	-24
47	-16
52	-16.5
55	-12.5

Downstream of the groyne up to cross section 42 the scour depths are small because of the influence of the groyne, but downstream of this cross section the scour depths increase to 10 m.
It should be mentioned that the scour depths near the bank are probably a little bit less than the scour depths at 225 m from the bank.

Downstream of a not-submerged groyne it is expected that some accretion will take place. The model tests give no information on the size of this accretion. If the groyne is of the submerged type it depends on the ratio of the time that the groyne is submerged whether or not some siltation of shallow areas will take place.

D.5.6.4.5 Continuous revetment

For the alternative solution of a continuous revetment from cross section 30, downstream of Eklashpur to cross section 60, just downstream of Mohanpur the scour has been measured in 7 cross sections perpendicular to the bank. Each cross section has a length of 600 m. The results of tests T7 and T9 are used to analyze this scour.

If the local flow velocity becomes less than 0.24 m/s in the model, then sedimentation is dominating the scour process. Since the sedimentation process is not on scale in the model this sedimentation is not reliable. Because the model is a local scour model without supply of upstream sediment, the local scour in the upstream part of the model is most reliable and these data are used for the analysis; (see also Figure 5.23).

TABLE D.5.8 THE EQUILIBRIUM SCOUR DEPTHS AND BED LEVELS AT 225 M DISTANCE FROM THE BANK

test	cross section	initial bed level	bed level after scour	scour depth
-	-	m-P.W.D.	m-P.W.D.	m
T7	55	- 9.75	-13.0	-3.25
T7	57	- 5	-12.5	-7.5
T9	55	- 9.3	-16.0	-6.7
T9	57	- 6.3	-10.7	-4.5

The extrapolation from the measured scour depths to the equilibrium scour depth is not so accurate if the scour depths are relatively small, therefore the decisive maximum equilibrium scour depth in cross section 57 is rounded off to the safe side to 10 m. Also more close to the bank the same scour depth is adopted.

The equilibrium scour depths in the other cross sections are estimated by relating the scour depth z_s in cross section 57 to the local flow velocity, u, (in that cross section: z_s is proportional to u^x in which $x > 1$). Given the local flow velocities at 225 m from the bank in other cross sections the local scour depths are estimated by their proportionality. The results are presented in the Table below:

TABLE D.5.9 ESTIMATED EQUILIBRIUM SCOUR DEPTHS AND BED LEVELS, WORST CASE, CONTINUOUS REVETMENT

cross section	initial bed level	bed level after scour	scour depth
-	m-P.W.D.	m-P.W.D.	m
32	-11.3	-19.3	-8
37	-16.1	-23.1	-7
42	-21.8	-26.8	-5
47	-13.4	-18.4	-5
52	-14.0	-19.0	-6
57	- 5.7	-15.7	-10
62	- 9.5	-17.5	-8
66	- 6.9	-11.9	-5

In the bend the maximum scour depths are estimated, but the bed levels are not very low if compared with the bed levels downstream of the bend. These results seem to be realistic.

In front of a continuous revetment from cross section 30 to cross section 60 some scour occurs over the full length of the revetment in a worst case situation. Because of this length this scour cannot be characterised as local scour, but rather as continuous scour along a bank. In general, Consultants prefer to study this type of scour in a river model with a moveable bed, instead of in a local scour model. Therefore, a rather complicated and tentative analysis has been applied on the results of the local scour model.

In the bathymetric map a remarkable confluence scour hole has been intersected by cross section 55. In test T7 the bed geometry does not include any confluence scour hole, because this bed geometry is more or less artificially constructed. With the results of T7 it can be checked if the confluence scour hole will be reproduced in the model. When comparing the scour depths in cross section 55 in T7 with the initial bed levels from the bathymetric survey, (see Figure 5.24), it can be concluded that the model has a tendency to reproduce the confluence scour hole. It is expected that in a complete moveable bed model the full depth of this confluence scour hole can be reproduced.

D.5.7 **Summary and Conclusions**

D.5.7.1 **General**

Within the frame work of the Meghna River Bank Protection Short-Term Study a scale model was constructed of a part of the Lower Meghna River near Eklashpur, and some aspects of the proposed bank protection works were studied in that model. In this scale model two types of solutions were tested: a groyne upstream of Eklashpur and a continuous revetment from Mohanpur to downstream of Eklashpur.

A part of the Lower Meghna River from the confluence of the Padma River with the Upper Meghna River to a few kilometres downstream of Eklashpur having a width of about 1.5 km was modelled at a length scale of 1 in 150. The non-distorted model has three or four inflow boundaries and one outflow boundary with different tailgates. The discharge distribution along the inflow boundaries was obtained from the mathematical model study.

In the steady flow model a constant discharge was pumped around during a test. The influence of the tidal fluctuations on the scour process was neglected in this model, as is usual in this type of models, because of the conflicting time scales of the tidal flow and the scour process. Since in the model the initiation of motion of the bed material is observed locally, no facilities for the circulation of the sediment are required.

In the first tests T1 to T6 the model bed represented the bed geometry which was measured during the field survey in February 1991. The flow velocities were calibrated with flow velocities measured during the field survey in August-September 1991.

In the actual situation the BWDB protection and a groyne upstream of Eklashpur were tested.
For the design conditions a so-called worst case situation was defined in the Geomorphological Study (Annex B). For this worst case situation two extreme discharge distributions were determined in the Hydrologic Study (Annex A). With the Mathematical Model Study results (Annex E) the detailed distributions over the inflow weirs were calculated. The continuous revetment and the upstream groyne were tested in these worst case conditions, see the Table D.5.2.

The BWDB protection has a length of about 600 m and is located upstream of Eklashpur. The continuous revetment starts just upstream of the bend and continues to a point downstream of Eklashpur. The upstream groyne has a length of 600 to 700 m.

D.5.7.2 Flow velocities

For the flow velocities along the bank a representative flow velocity was defined and this flow velocity was calibrated with the flow velocities measured during the flood survey in August September 1991.

(a) *Upstream groyne*

As design velocity along the upstream side of the groyne a maximum flow velocity of 2.1 m/s has been measured, and around the head of the groyne a maximum flow velocity of 2.8 m/s has been measured. The groyne reduces the representative flow velocities along the existing bank by 0.5 m/s to 2.5 km downstream of the groyne, or about 4 times the length of the groyne.

(b) *Revetment*

In the bend near the confluence the representative flow velocity is 1.9 m/s and this is the maximum flow velocity. Downstream of the bend this flow velocity reduces to 1.0 m/s and downstream of cross section 45 this flow velocity increases up to 1.7 m/s.

248

D.5.7.3 **Scour depths**

(a) *Upstream groyne*

Near the head of the groyne the local scour hole coincides with the confluence scour hole and the deepest point has a level of 36 m-P.W.D. Along the bank downstream of the groyne the scour depths are reduced by about 2 m up to cross section 40 (if compared with the continuous revetment).

(b) *Revetment*

Along the continuous revetment the deepest point of the scour hole has a level of 27 m-P.W.D. near Eklashpur. The scour depths are estimated at 5 m in that location. In the bend of the bank near the confluence the equilibrium bed levels after scour are higher than 27 m-P.W.D., but scour depths of about 10 m are maximum at that location.

TABLE D.5.10 PROTOTYPE AND MODEL DISCHARGES AND THE FLOW VELOCITY SCALE FACTOR

Test	$Q_M/(Q_M+Q_P)$	Q_M	Q_P	$Q_{p,m}$	Q_m	n_u
-	-	m3/s	m3/s	m3/s	m3/s	-
T1	0.29	38400	92000	43000	0.150	12.7
T2	0.29	38400	92000	43000	0.377	5.1
T3	0.29	38400	92000	43000	0.398	4.8
T6 21-9	0.29	38400	92000	43000	0.448	4.3
T6 25-9	0.29	38400	92000	43000	0.487	3.9
T6 8-10	0.29	38400	92000	43000	0.470	4.1
T6	0.37	48100	81900	75000	0.297	11.2
T7	0.37	48100	81900	75000	0.467	7.1
T8	0.20	26000	104000	60000	0.298	8.95
T9	0.20	26000	104000	60000	0.427	6.2
T10	0.37	48100	81900	75000	0.328	10.2
T11	0.37	48100	81900	75000	0.438	7.6
T12	0.20	26000	104000	60000	0.320	8.3
T13	0.20	26000	104000	60000	0.420	6.3

TABLE D.5.11 WATERLEVEL GRADIENT

Test number	slope waterlevel	slope waterlevel	slope waterlevel	slope water level
-	10-3 -	10-3 -	10-3 -	10-3 -
T1	0.0615	0.054	0.060	0.058
T2	0.946	0.063	0.051	0.149
T3	0.984	0.045	0.017	0.129
T6 21-9-91	2.19	0.909	4.310	2.1
T6 25-9-91	0.46	1.11	0.913	0.86
T6 8-10-91	0.038	0.58	1.42	0.91

TABLE D.5.12 DATA FOR THE CALCULATION OF THE CHEZY-COEFFICIENT IN THE MODEL TESTS

parameter	unit	T1	T2	T3	T6	T6
Q	m3/s	0.156	0.369	0.369	0.448	0.487
h	m +PWD	0.043	0.042	0.042	0.042	0.046
i	-	.000044	.000055	.000343	.0010	.00052
A	m2	1.239	1.229	1.229	1.229	1.269
u	m/s	0.126	0.30	0.30	0.364	0.384
R	m	0.120	0.119	0.119	0.119	0.123
C	m0.5/s	54.8	117	47.0	33.4	47.9

TABLE D.5.13 FLOW VELOCITIES ALONG THE LEFT BANK

cross sect.	T10	T11	T12	T13
	u (m/s)	u (m/s)	u (m/s)	u (m/s)
20	1.30	1.47	1.31	1.06
30	1.91	1.90	1.69	1.42
35	1.68	1.47	1.39	1.36
40	0.82	0.50	<0.3	0.65
45	<0.3	<0.3	<0.3	0.63
50	<0.3	<0.3	<0.3	1.16
55	1.16	1.56	1.25	1.44
62	1.16	1.01	0.90	0.76

730

TABLE D.5.14 BED LEVELS AND MAXIMUM SCOUR DEPTHS IN TEST T3

cross section	location, type	initial bed level	equilibrium bed level	scour depth
-	-	m	m	m
32	1 ,max	+3.0	-3.5	-6.5
32	4 ,deepest	-11.5	-9.25	+2.25
37	1 ,max	+3.4	-2.4	-5.8
37	9 ,deepest	-12.8	-3.5	+9.3
42	9 ,max deepest	-14.5	-21.3	-6.8
47	13 ,max	+3.0	-5.0	-8.0
47	21 ,deepest	-20.0	-20.0	0.0
52	15 ,max	+2.4	-3.1	-5.5
52	21 ,deepest	-26.0	-24.5	+1.5
55	2 ,max	-5.5	-7.7	-2.2
55	5 ,deepest	-35.0	-28.4	+ 6.6
57	1 ,max	+3.8	-5.0	-8.8
57	8 ,deepest	-24.0	-16.5	+7.5
62	14 ,max	+2.5	-4.8	-7.3
62	18,deep	-14.9	-17.5	-2.6
66	7,max	-9.4	-51.0	-41.6
66	4,deep	-11.0	-8.0	+3.0

* all levels below P.W.D.

TABLE D.5.15 BED LEVELS AND MAXIMUM SCOUR DEPTHS IN TEST T6, 25-9-91, AROUND THE HEAD OF THE GROYNE

cross section	location, type	initial bed level	equilibrium bed level	scour depth
-	-	m	m	m
A	4 ,max	-15.5	-23.0	-7.5
A	2 ,deepest	-18.5	-20.5	-2.0
B	3 ,max	-25.2	-29.0	-3.8
B	2 ,deepest	-27.0	-20.0	+7.0
C	2 ,max deepest	-27.9	-34.8	-6.9
D		sedimentation		

* all levels below P.W.D.

732

TABLE D.5.16 BED LEVELS AND MAXIMUM SCOUR DEPTHS IN TEST T6 8-10-91

cross section	location, type	initial bed level	equilibrium bed level	scour depth
-	-	m	m	m
32	9 ,max deepest	-10.7	-17.3	-6.5
37	7 ,max	-8.3	-14.0	-5.7
37	9 ,deepest	-9.4	-13.0	-3.6
42	20 ,max	-11.6	-13.7	-2.1
42	21 ,deepest	-12.2	-12.5	-0.3
47	21 ,max	-18.4	-22.0	-3.6
47	20 ,deepest	-18.9	-22.0	-3.1
52	22 ,max	-21.1	-28.0	-6.9
52	20 ,deepest	-23.5	-16.5	+7.0
55	9 ,max	-26.5	-30.0	-3.5
55	6 ,deepest	-37.0	-18.0	+19.0
57	1 ,max	+1.9	-3.5	-5.4
57	8 ,deepest	-21.3	-14.5	+6.7
62	14 ,max	+0.5	- 9.0	-9.5
62	17 ,deepest	-10.7	-14.5	-3.8

* all levels below P.W.D.

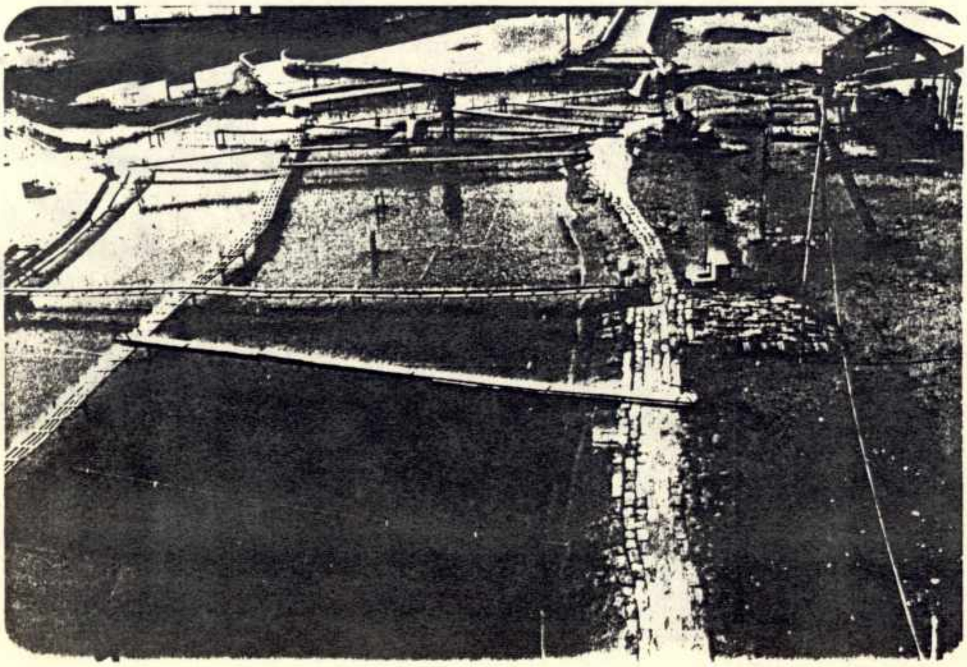
TABLE D.5.17 THE SCOUR DEPTHS AND BED LEVELS IN TEST T11 AT 225 M FROM THE BANK

cross section	initial level	equilibrium level	scour depth
-	m - P.W.D.	m - P.W.D.	m
32	-10.0	-20.0	-10.0
37	-14.5	-23.0	-8.5
42	-22.1	-24.0	-1.9
47	-14.4	-16.0	-1.6
52	-13.9	-12.0	+1.9
55	-9.2	-12.5	-3.3

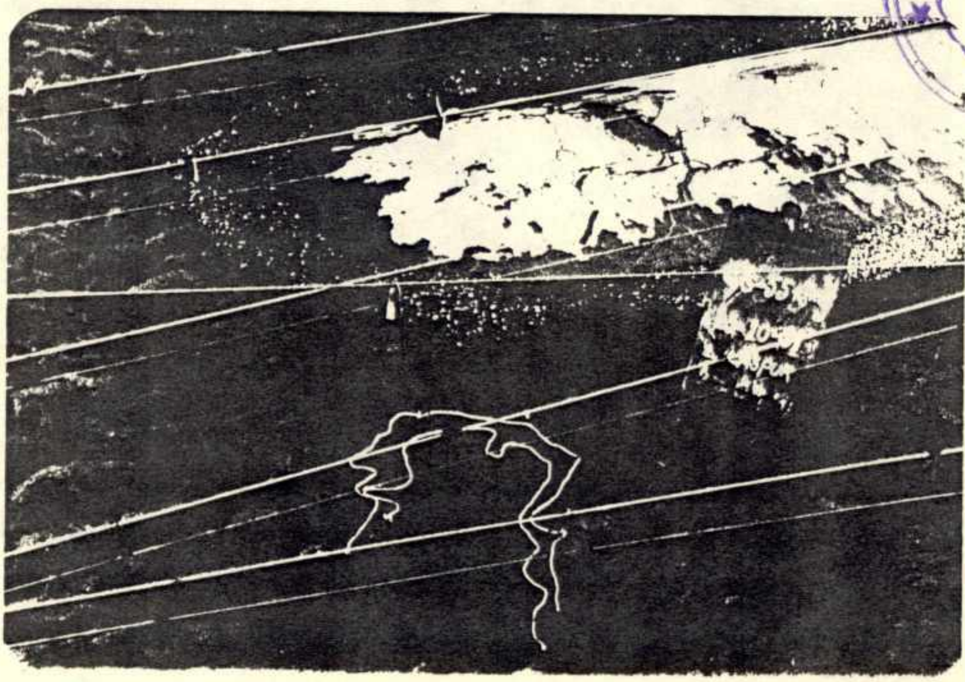
236

TABLE D.5.18 THE SCOUR DEPTHS AND BED LEVELS IN TEST T13 AT 225 M FROM THE BANK

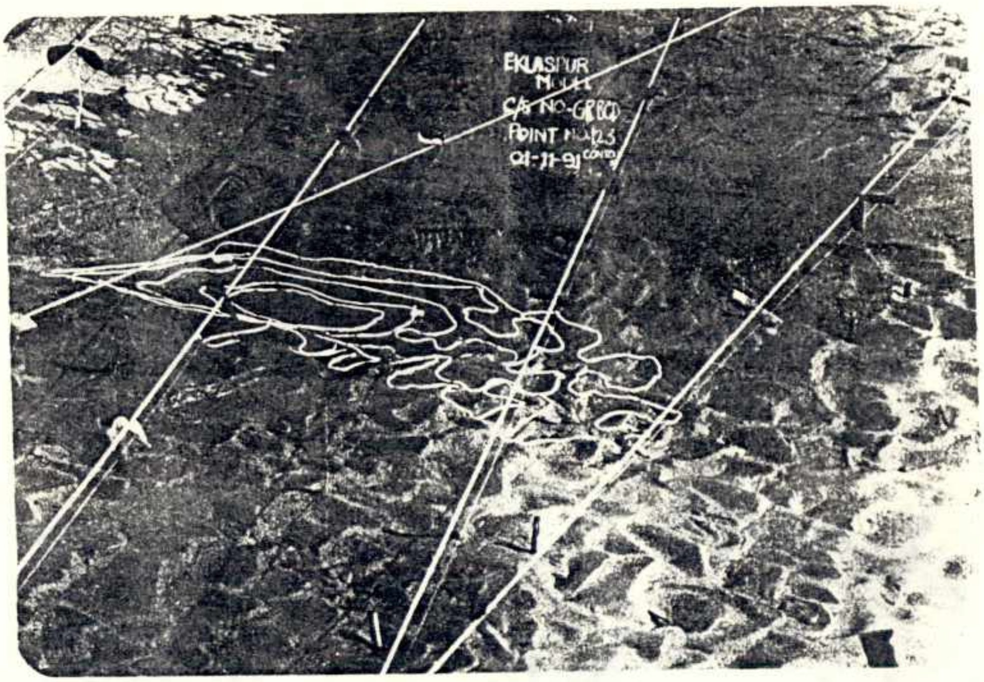
cross section	initial level	equilibrium level	scour depth
-	m - P.W.D.	m - P.W.D.	m
32	-9.7	-15.0	-5.3
37	-14.7	-18.0	-3.3
42	-21.0	-20.0	+1.0
47	-12.75	-14.0	-1.25
52	-14.4	-16.5	-2.1
55	-9.4	-11.8	-2.4



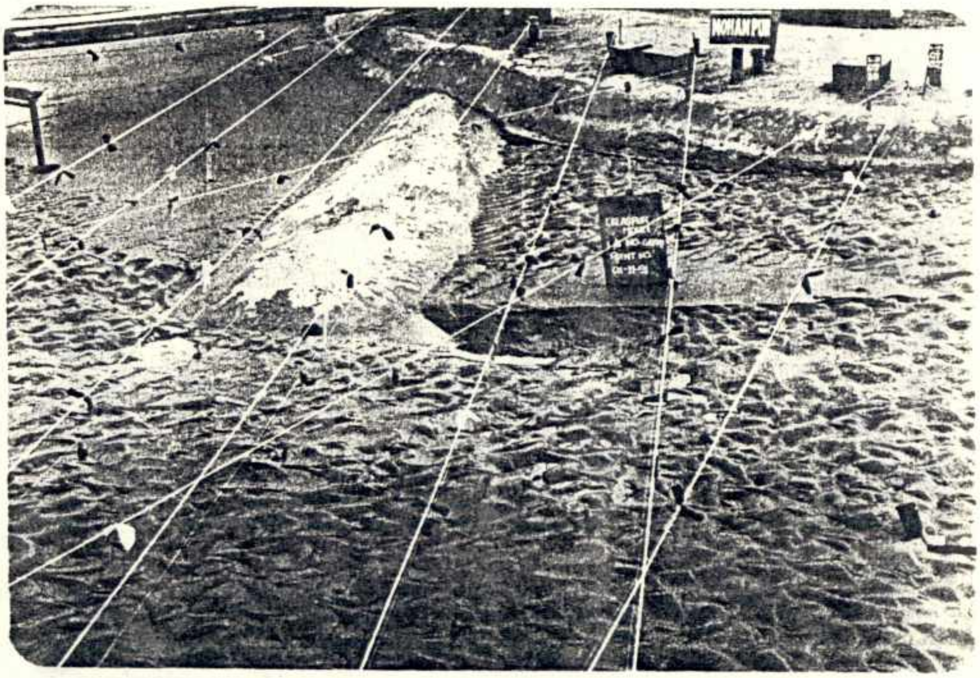
5.1 Overview of Eklashpur model



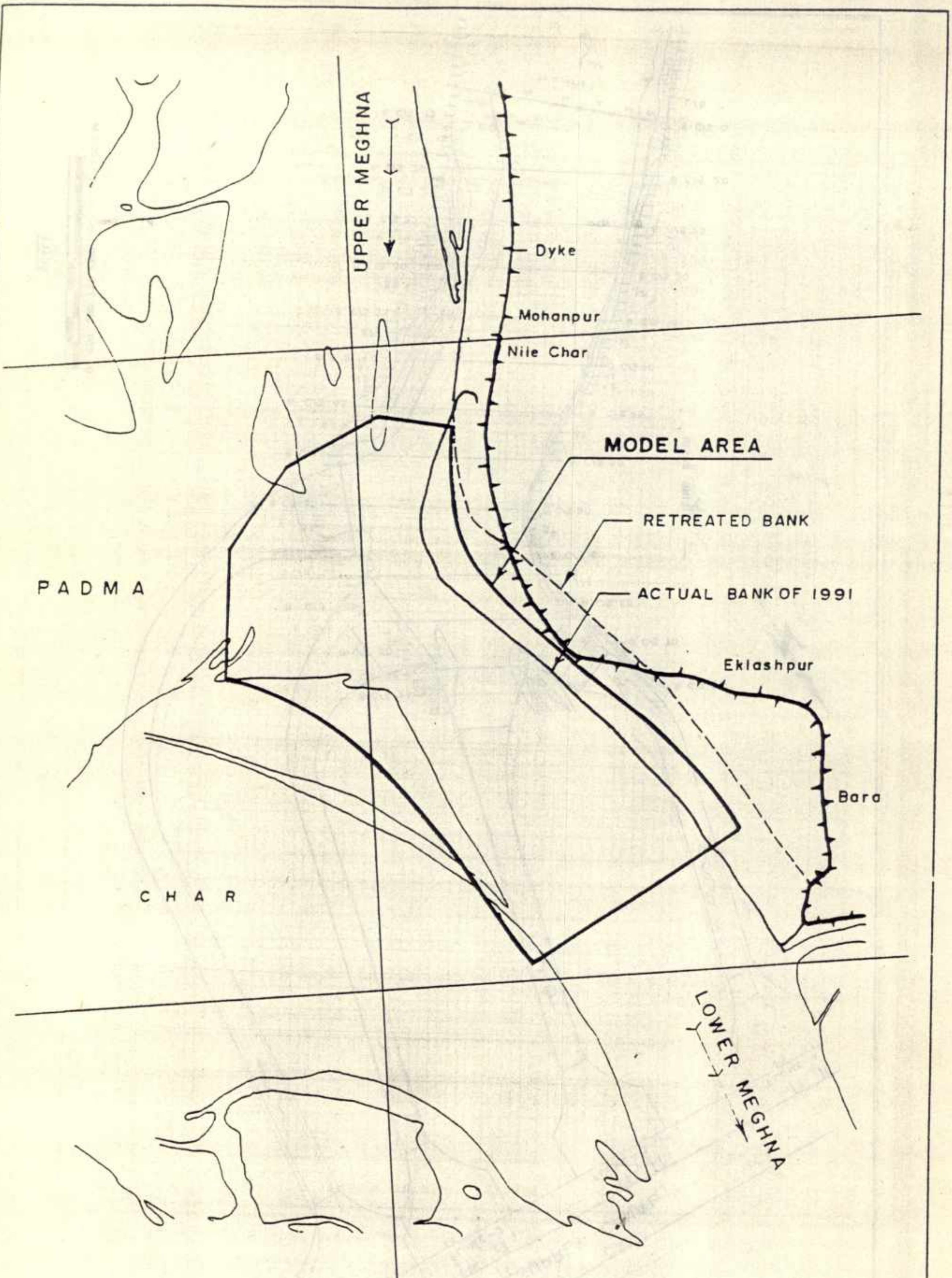
5.2 Detail of scour hole near groyne after test T11



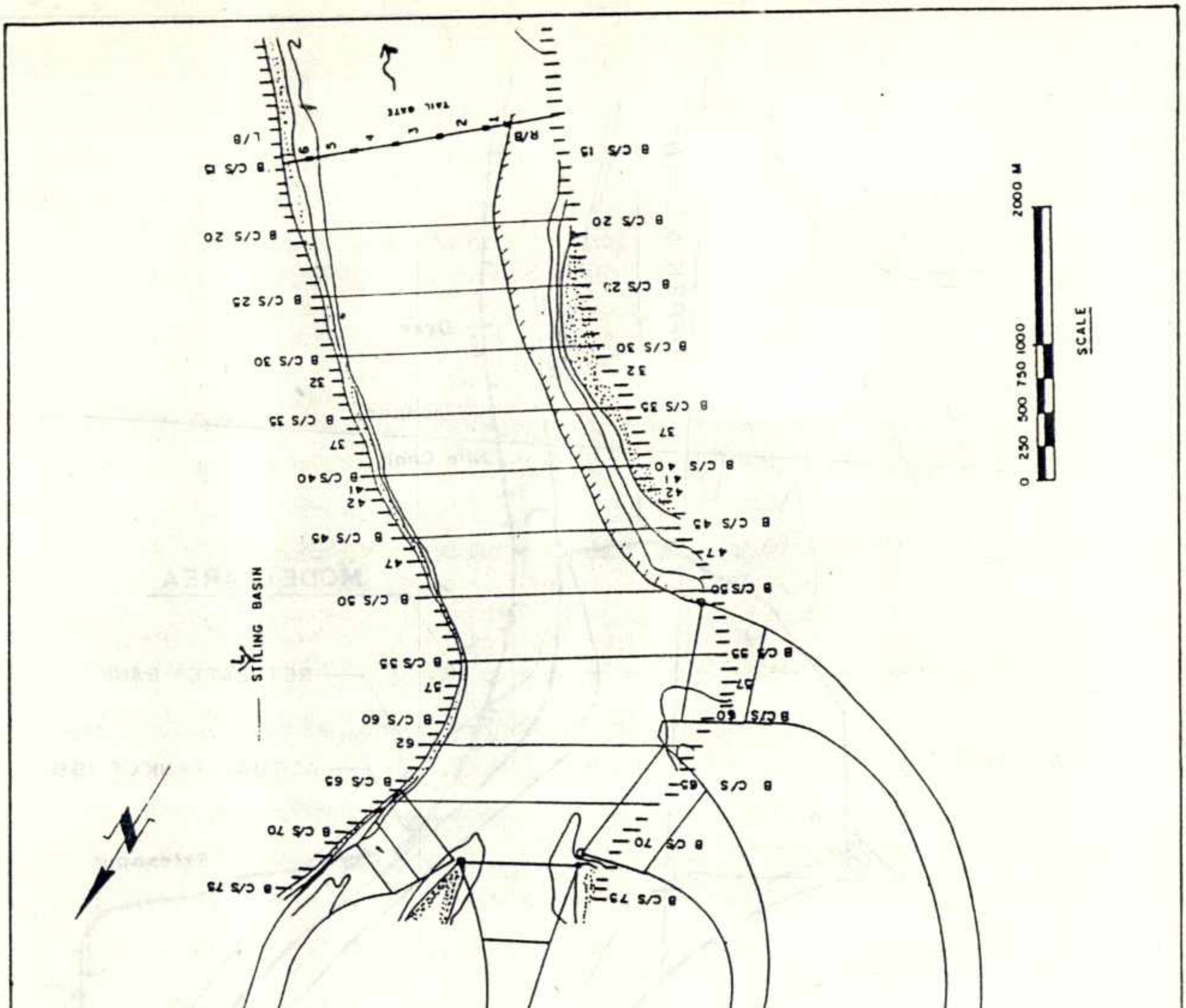
5.3 Groyne in dry bed after test T13



5.4 Detail of scour hole near groyne after test T13

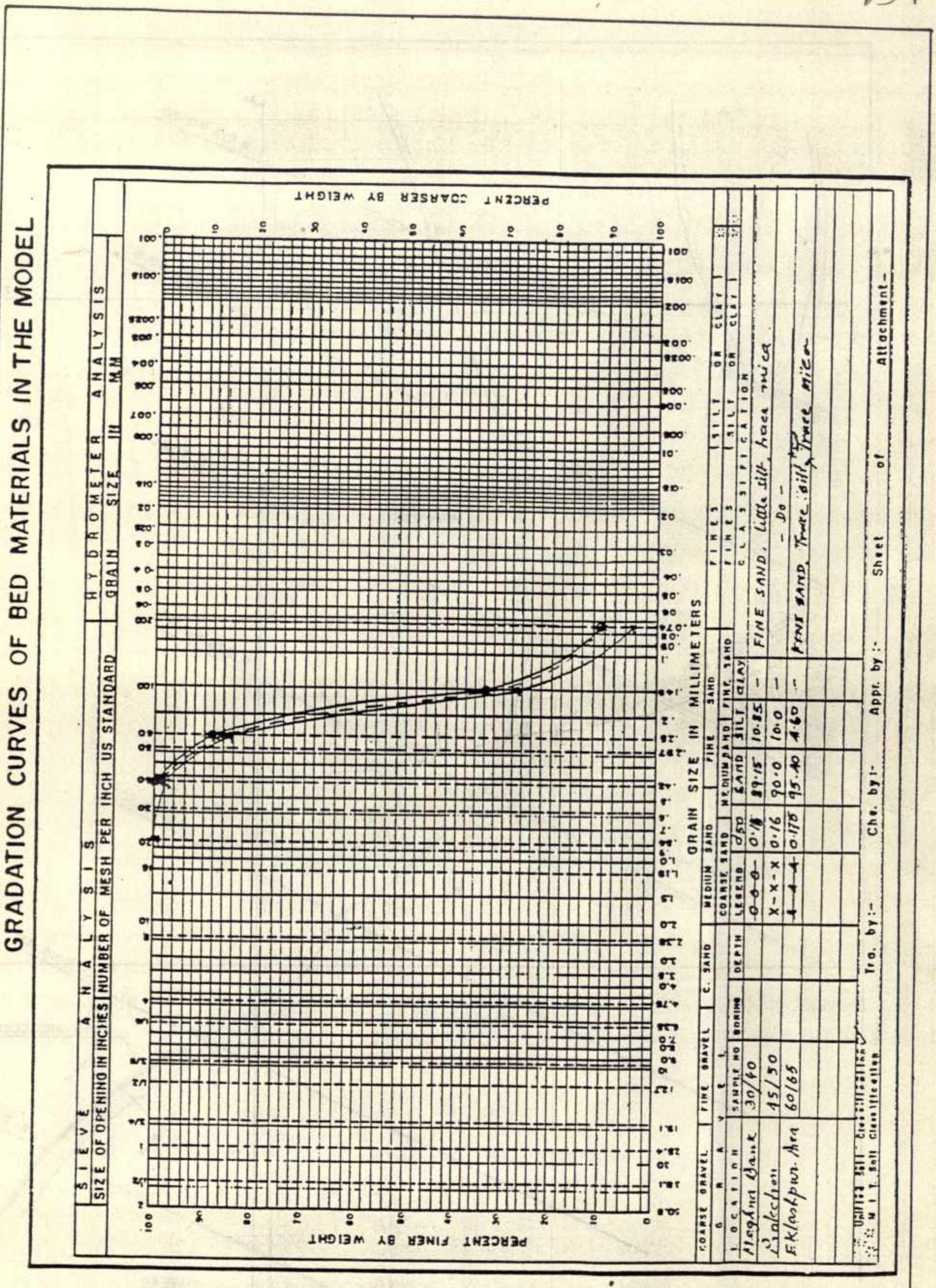


MODEL AREA	EKLASHPUR	
	MRBPSTS	FIG. D. 5.1

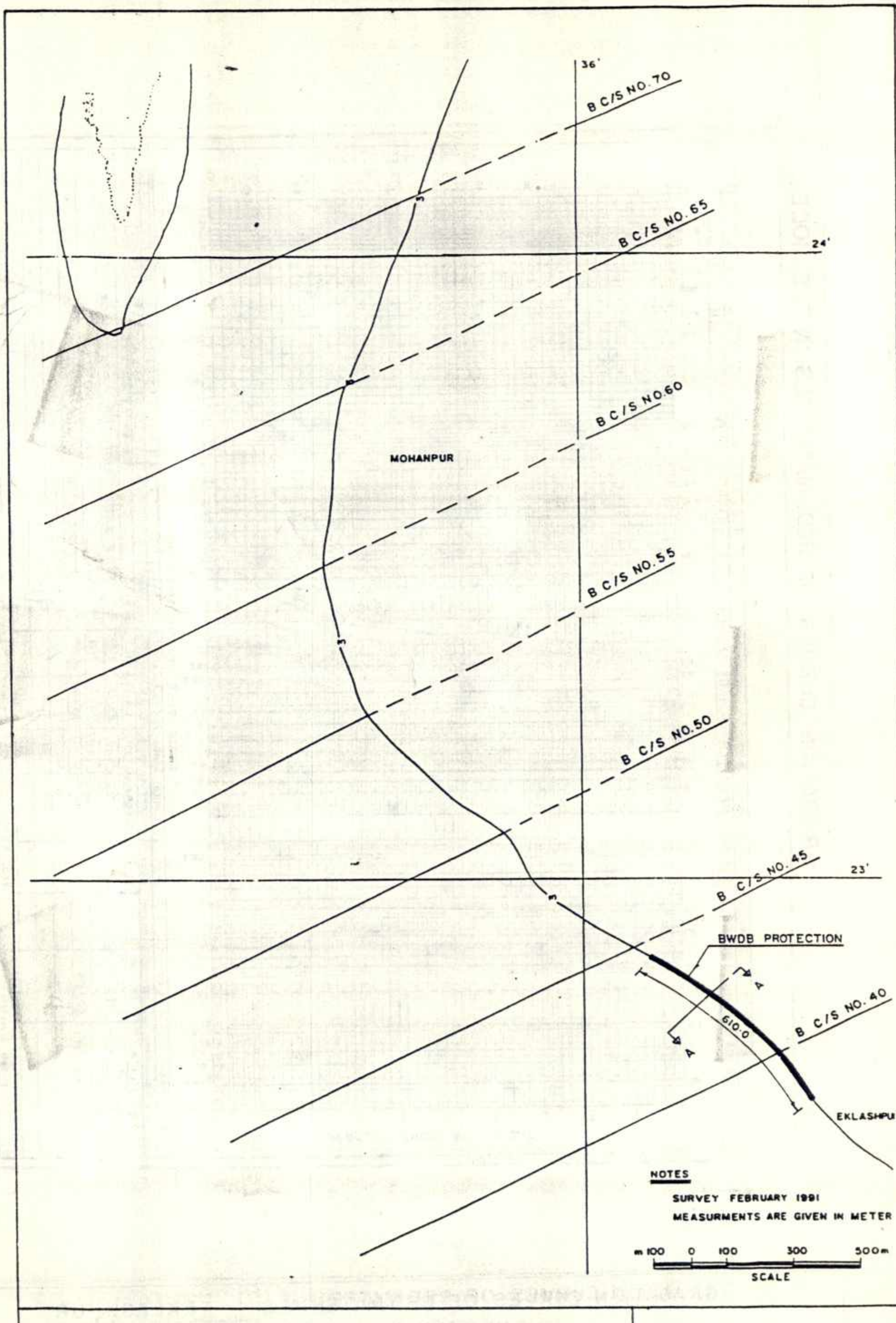


LAYOUT OF MODEL EKLASHPUR

FIGURE. D. 5.2



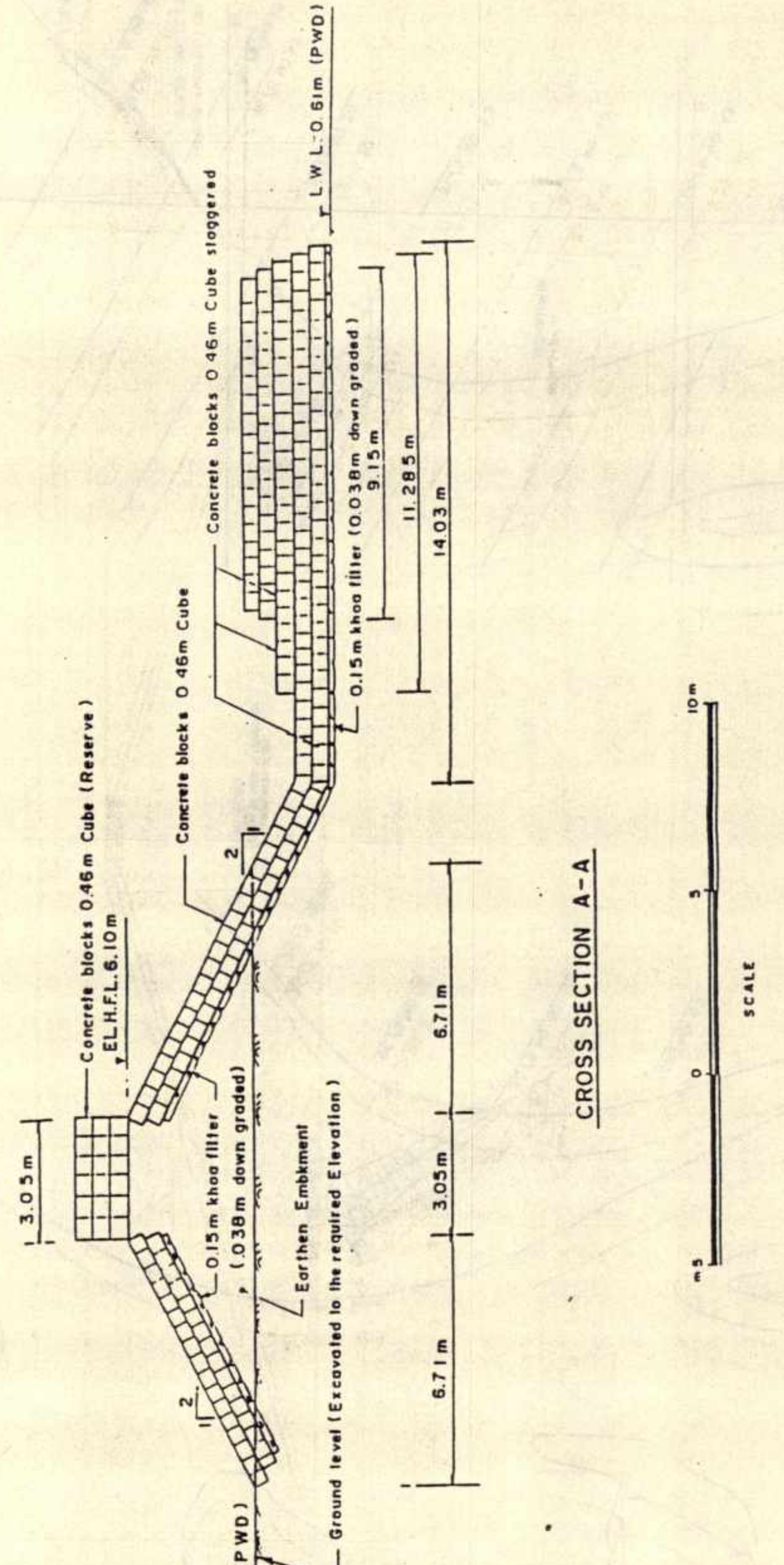
GRADATION CURVES OF BED MATERIALS IN THE MODEL	EKLASHPUR
	MRBPSTS — FIG. D.5.3



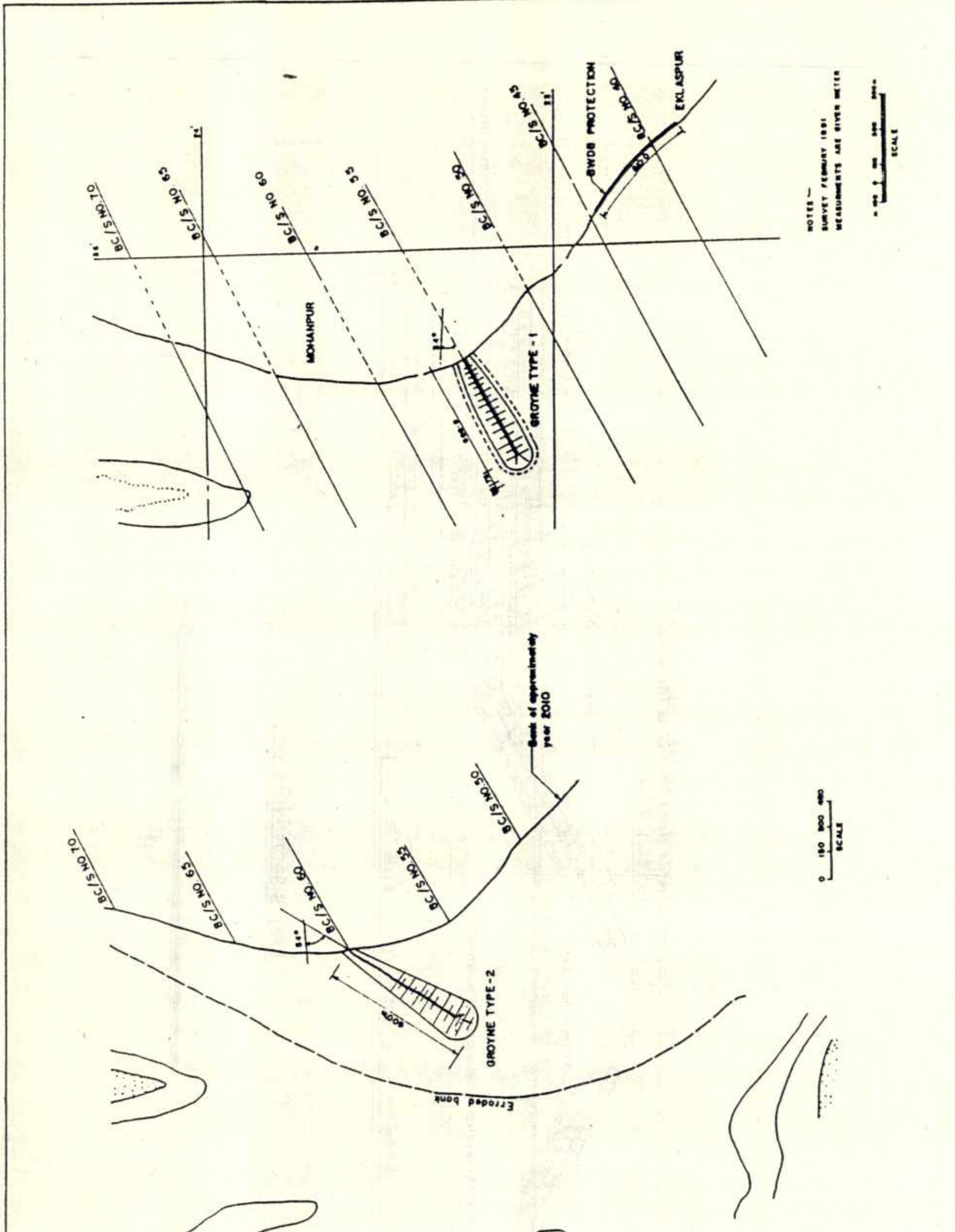
PROTECTION EKLASHPUR BWDB

FIGURE D.5.4

290

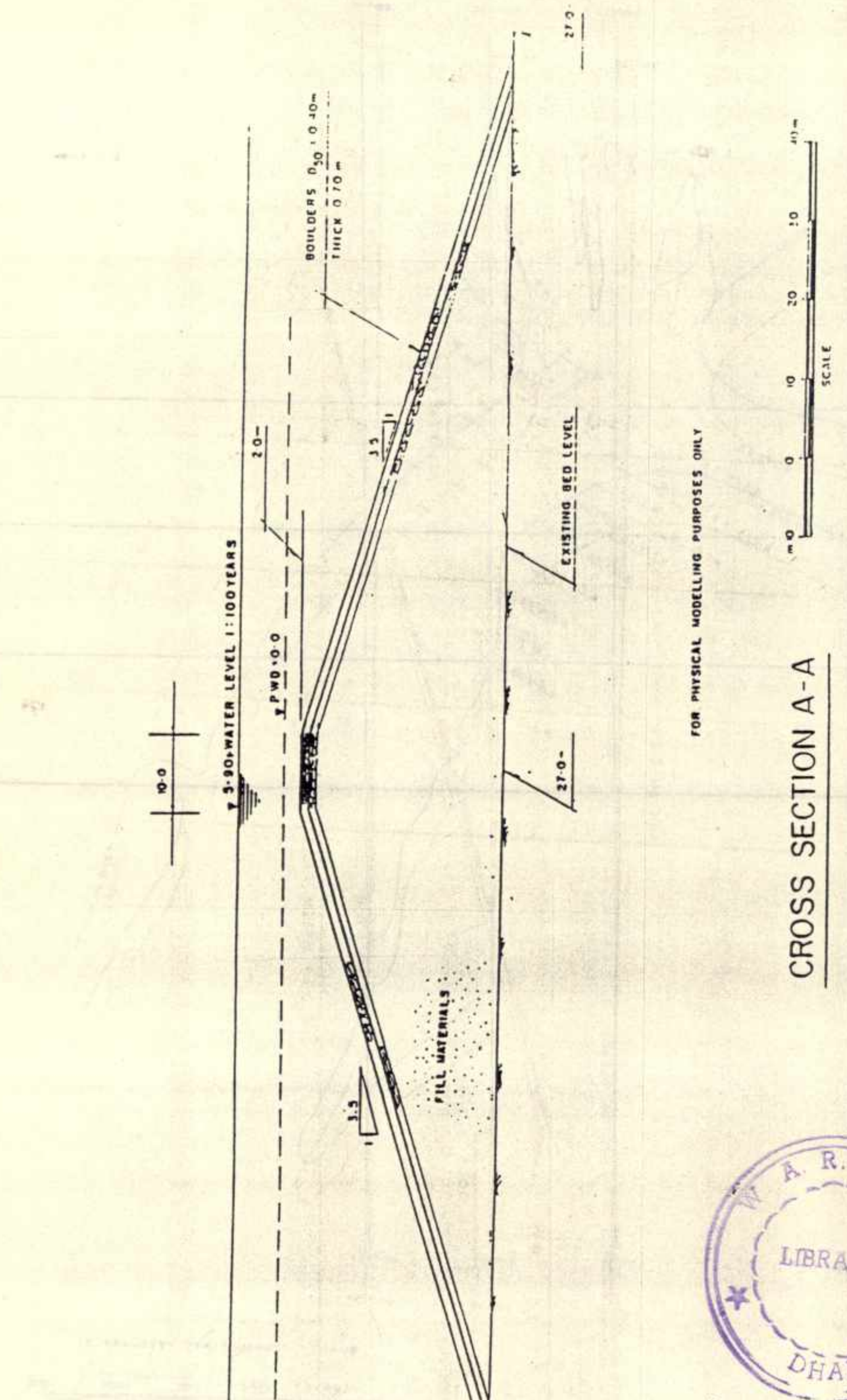


CROSS SECTION EKLASHPUR BWDB DESIGN	FIGURE D. 5.5

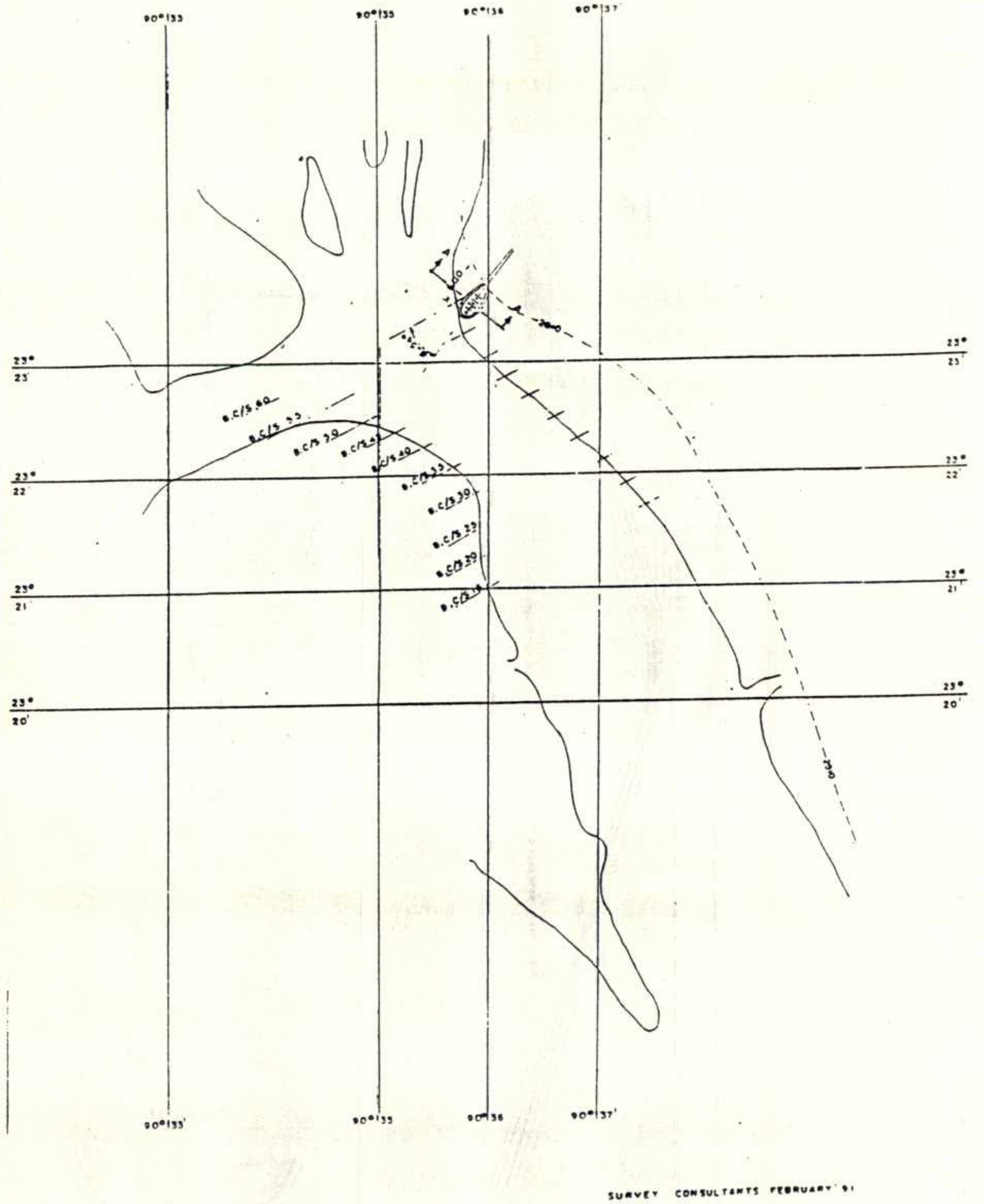


LAYOUT OF GROYNES, TYPE - I AND TYPE - 2
EKLASHPUR

FIGURE. D. 5.6



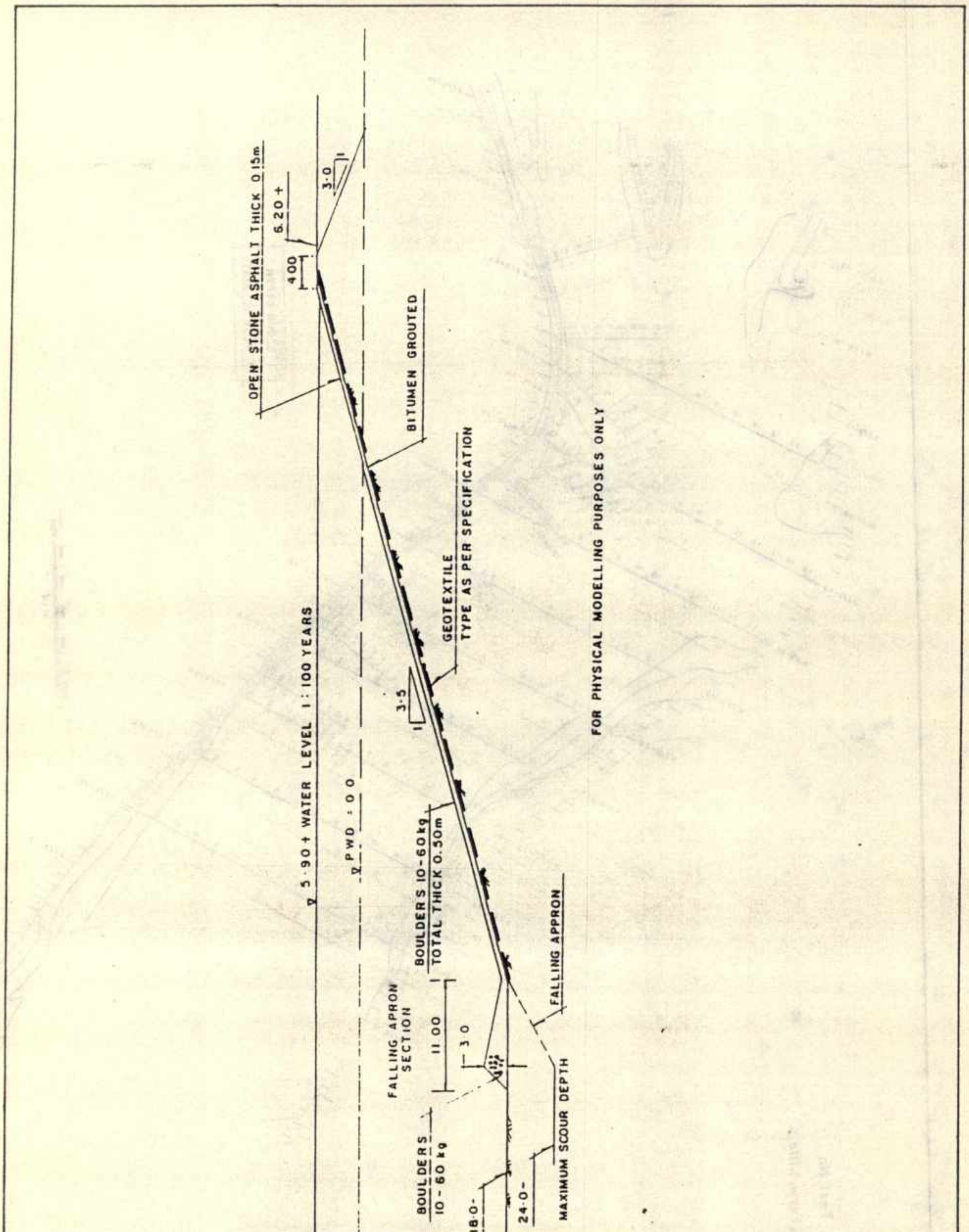
CROSS SECTION GROYNE EKLASPUR (LOCATION VARIABLE)	FIGURE D. 5.7



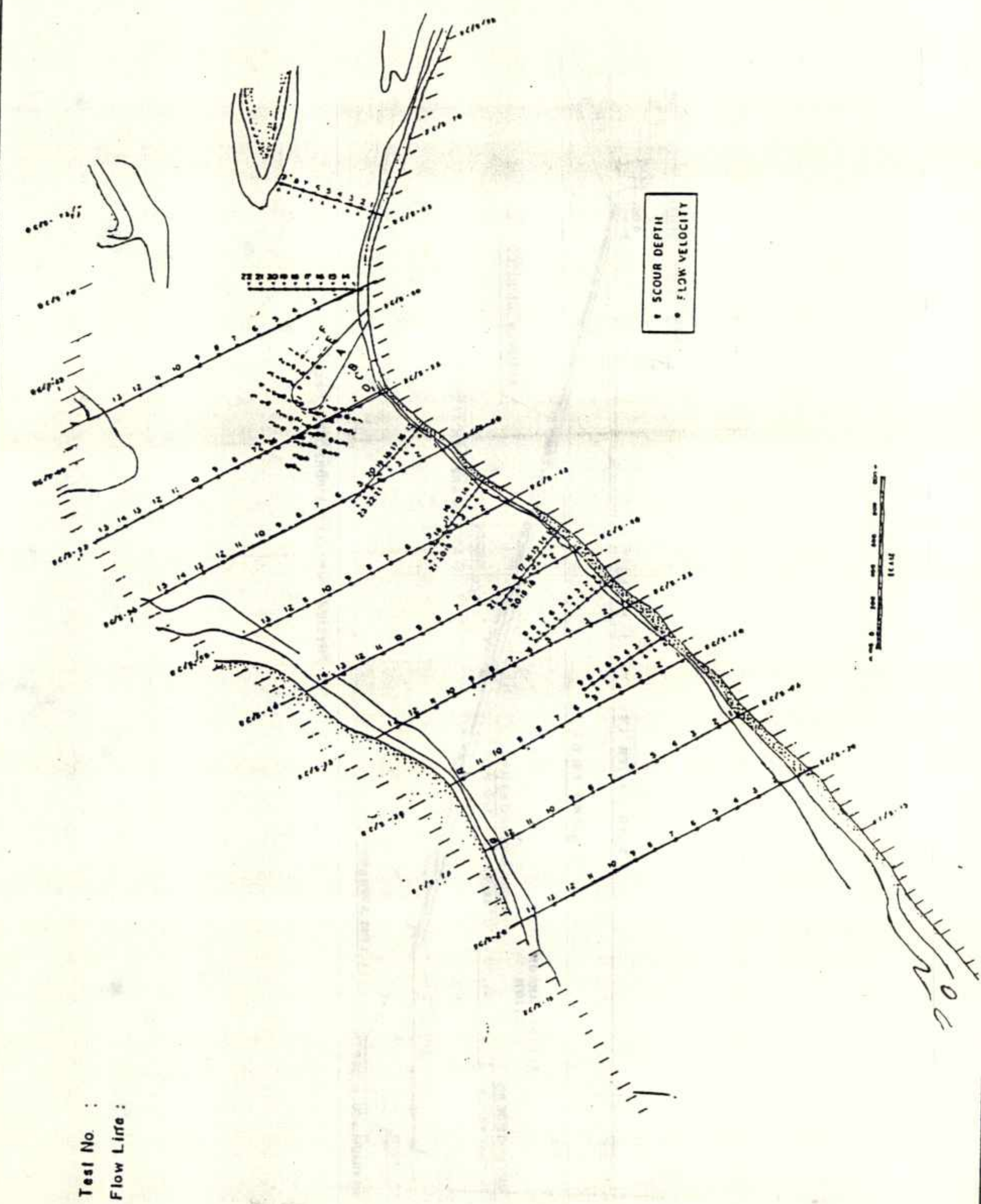
GROYNE ON FUTURE BANK LINE EKLASHPUR

FIGURE D. 5. 8

298

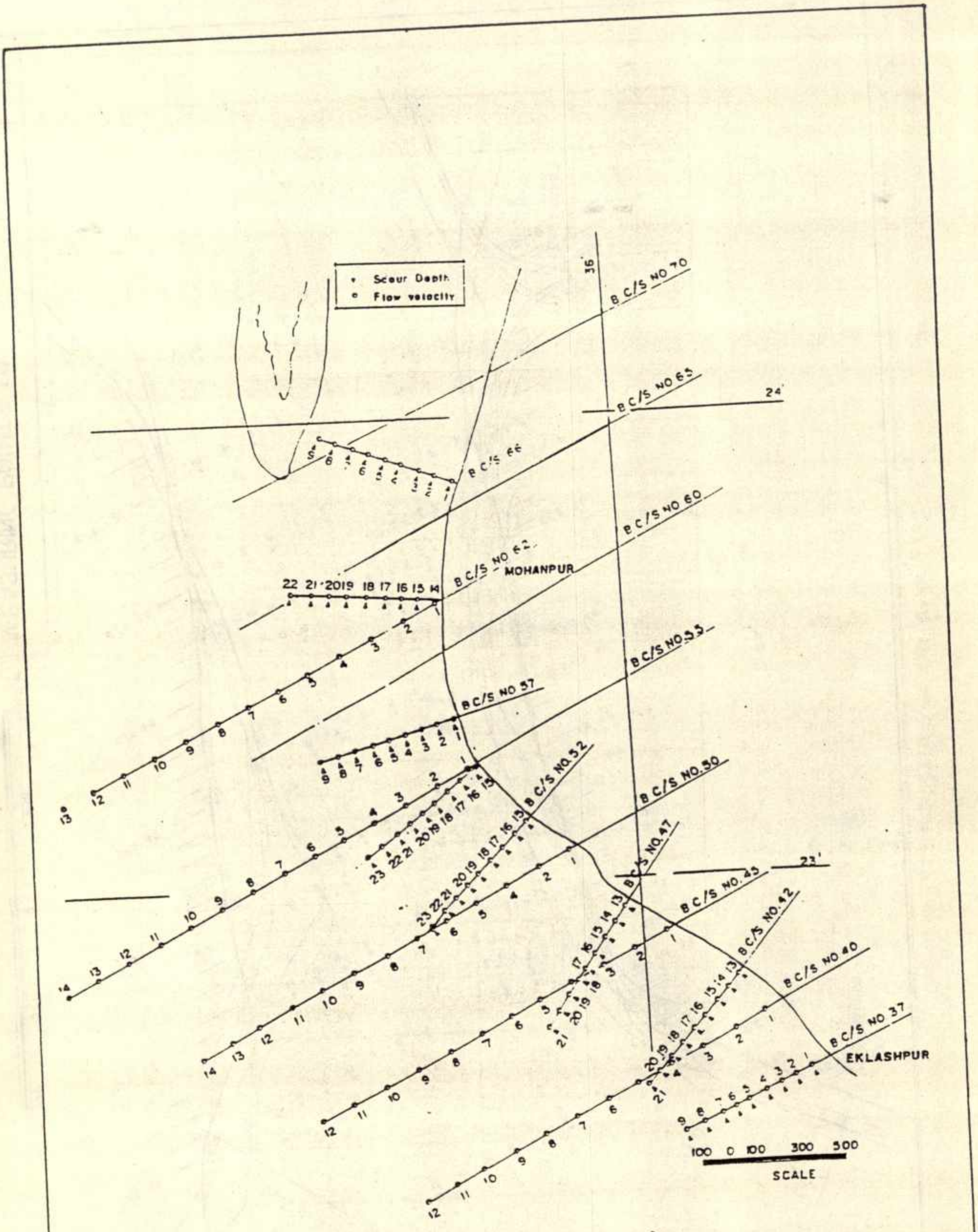


TYPICAL CROSS SECTION GUIDE BUND EKLASHPUR | FIGURE. D. 5. 9



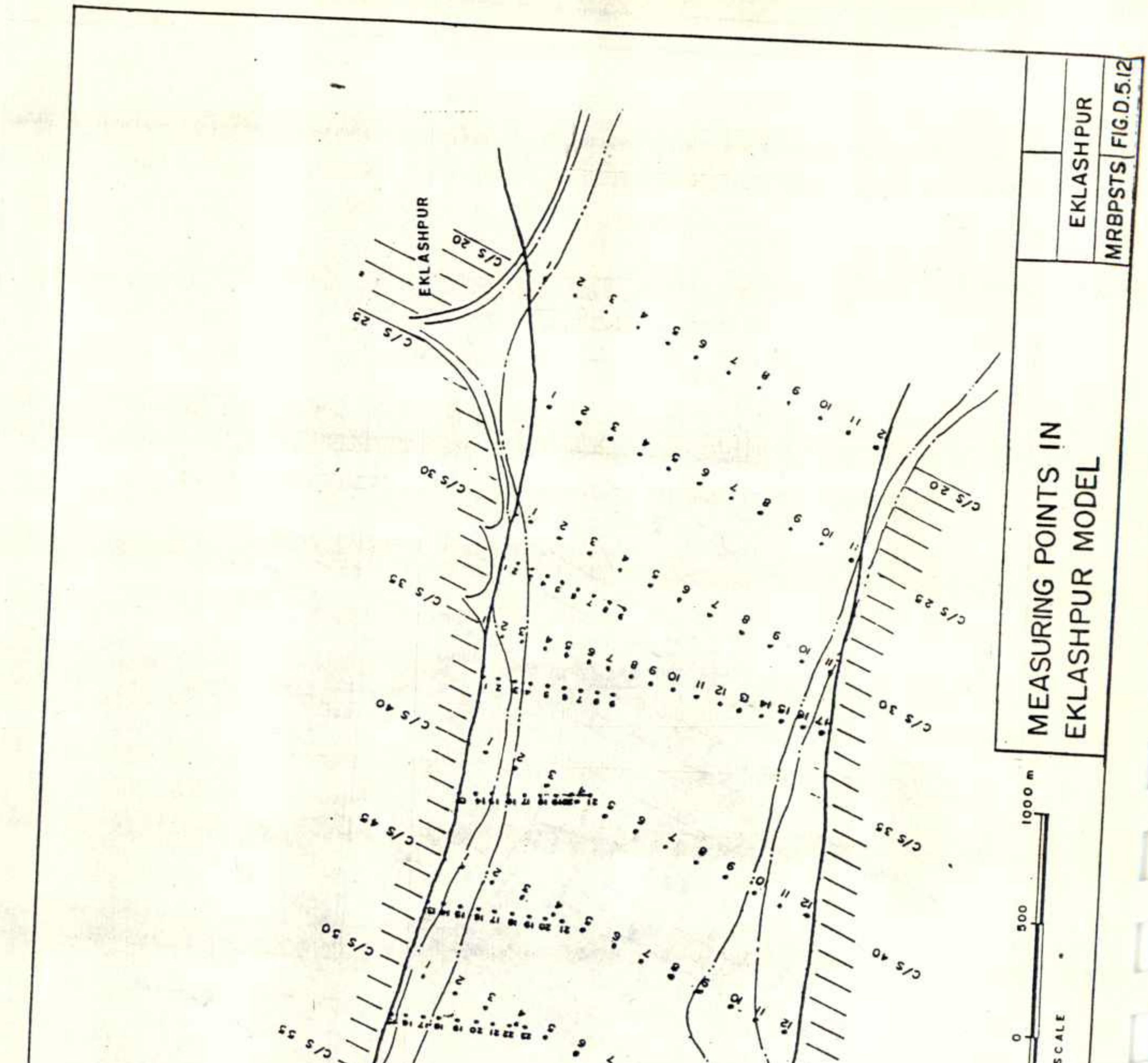
MEASURING POINTS IN TEST T_{10}
EKLASHPUR MODEL

EKLASHPUR	
MRBPSTS	FIG.D.5.10

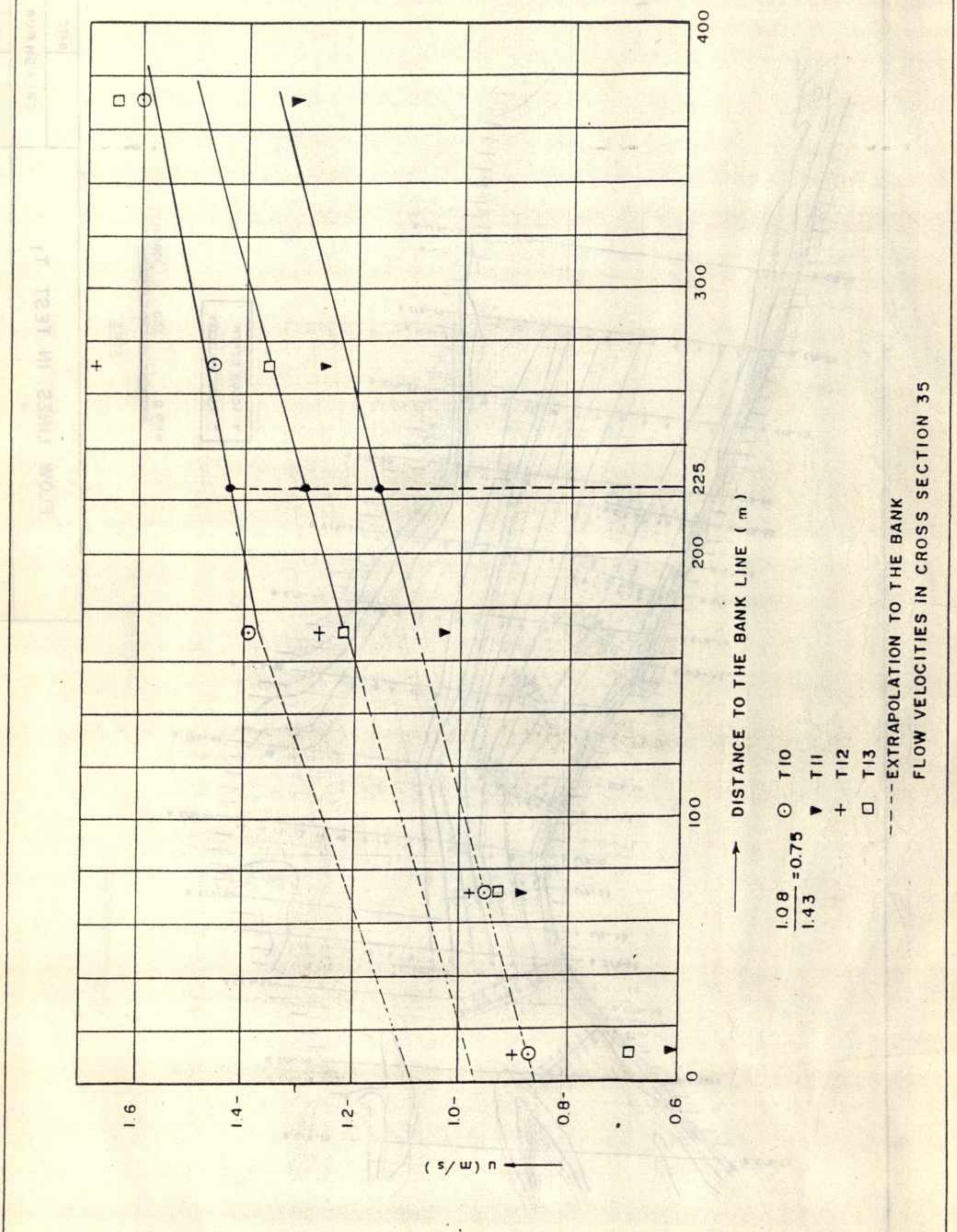


MEASURING POINTS IN EKLASHPUR MODEL, EKLASHPUR	FIGURE . D. 5. II

288

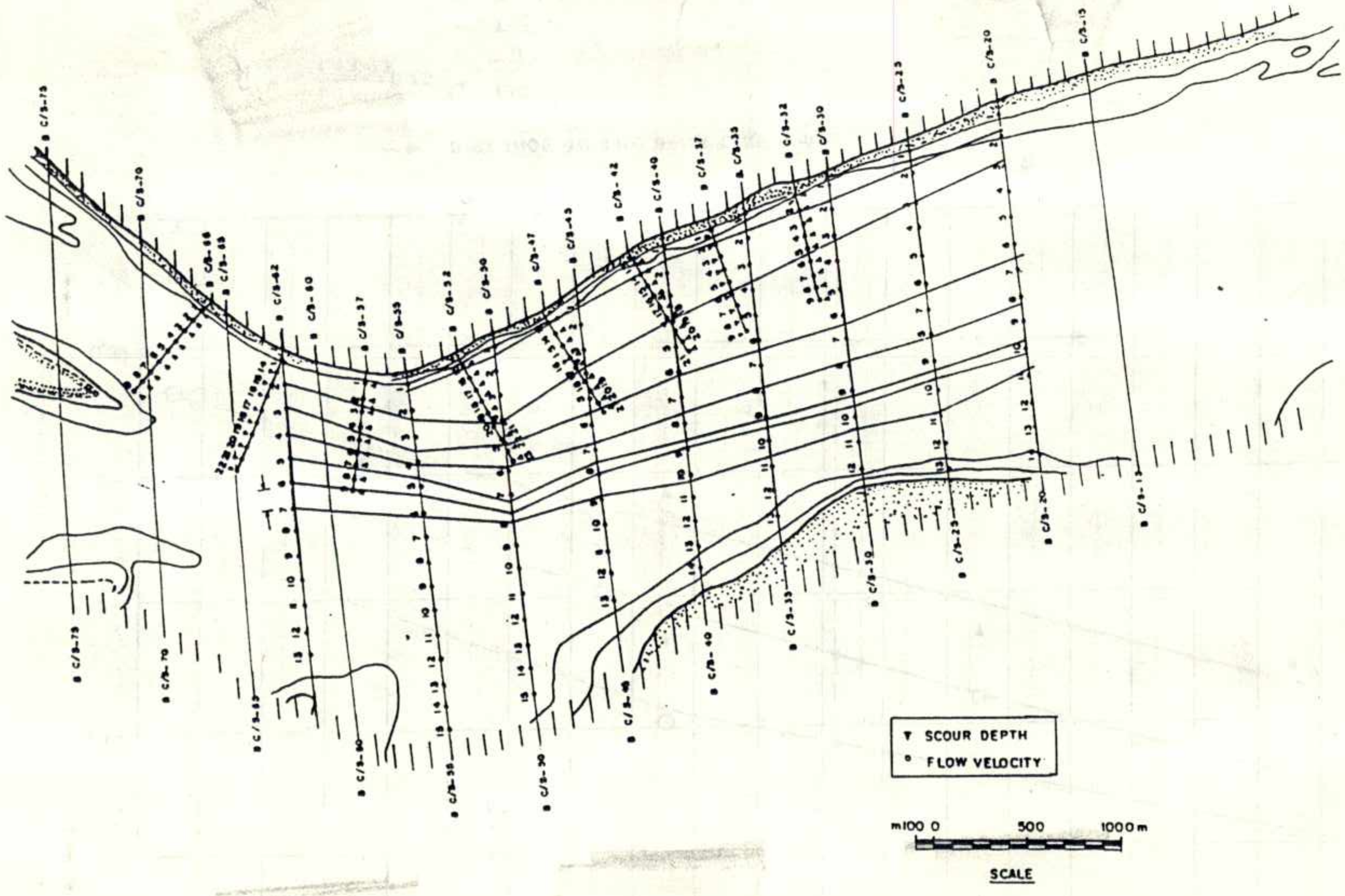


MEASURING POINTS IN
EKLASHPUR MODEL
EKLASHPUR
MRBPSTS FIG.D.5.12
EKLASHPUR
MOHANPUR
C/S 20
C/S 25
C/S 30
C/S 35
C/S 40
C/S 45
C/S 50
C/S 55
C/S 60
C/S 62
C/S 65
C/S 66
1000 m
500
0
500
m 1000
SCALE



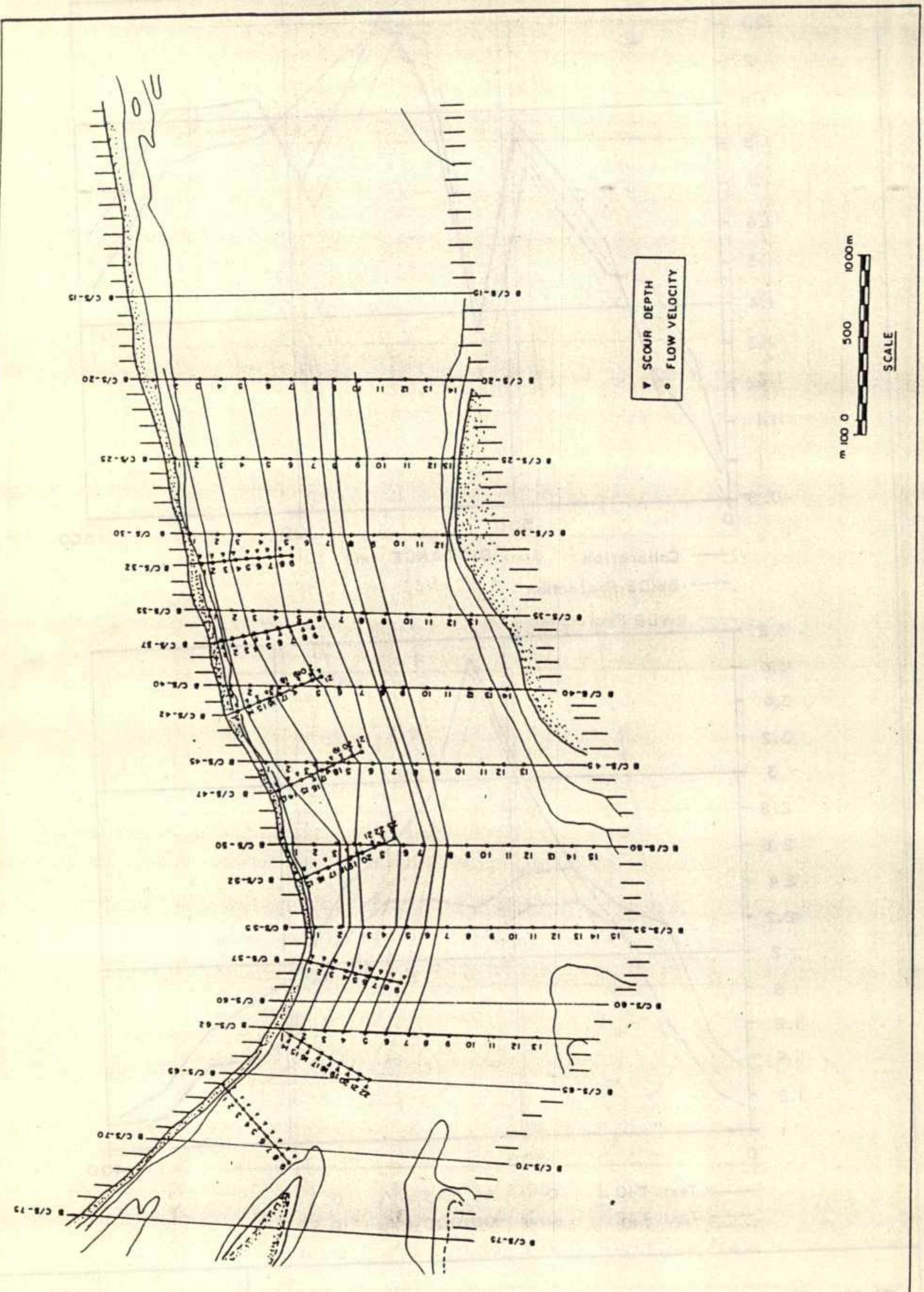
EXTRAPOLATION OF THE FLOW VELOCITIES OF THE BANK		DATE:
	EKLASHPUR	
	MRBPSTS	FIG. D. 5.13

282



FLOW LINES IN TEST T_1		DATE:
	EKLASHPUR	
	MRBPSTS	FIG. D.5.14

180

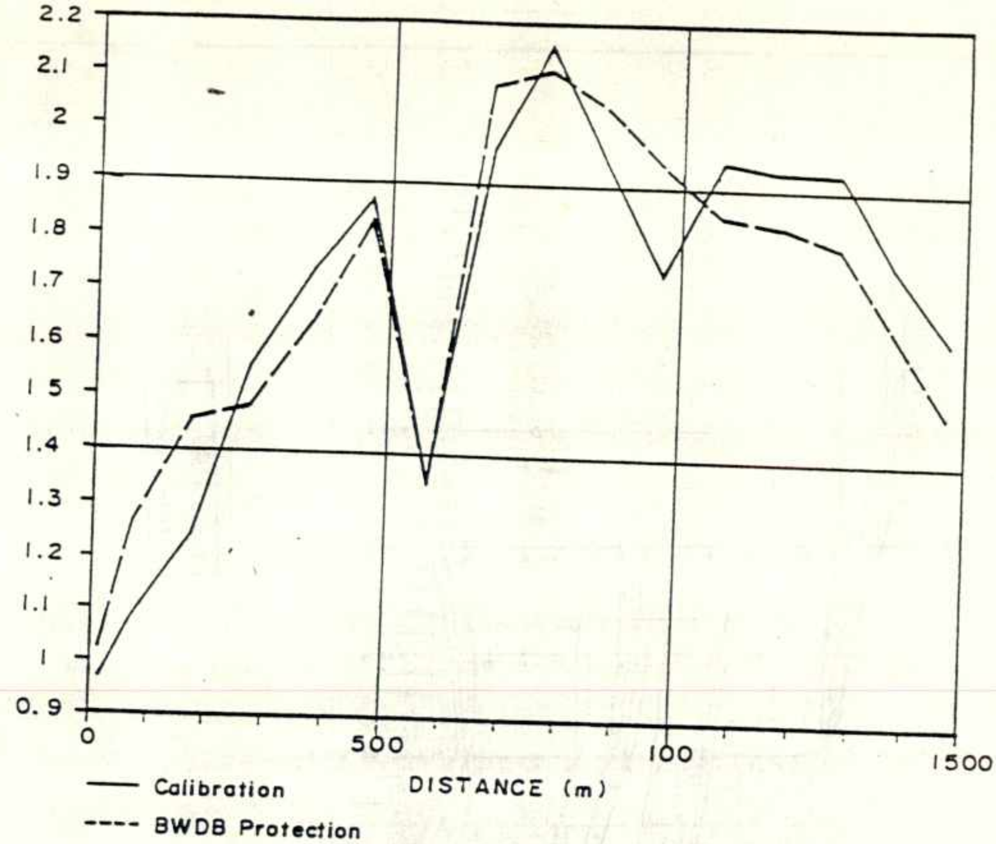


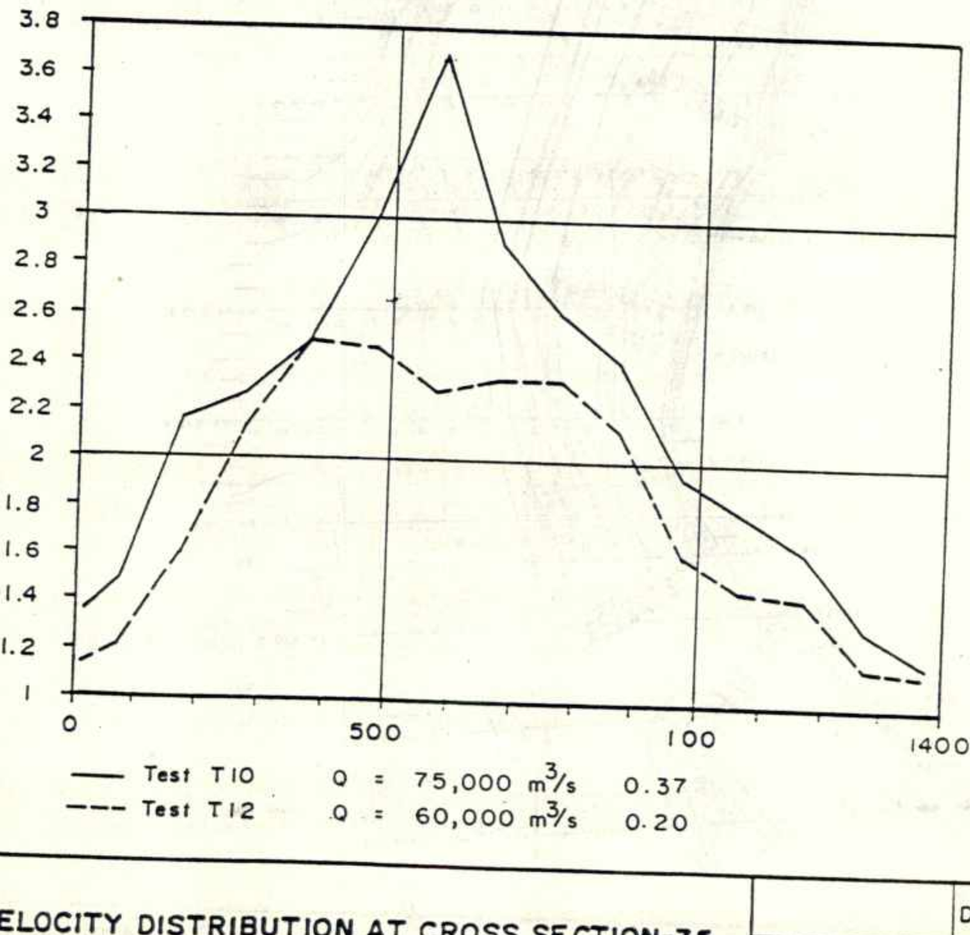
FLOW LINES IN TEST T_2

EKLASHPUR

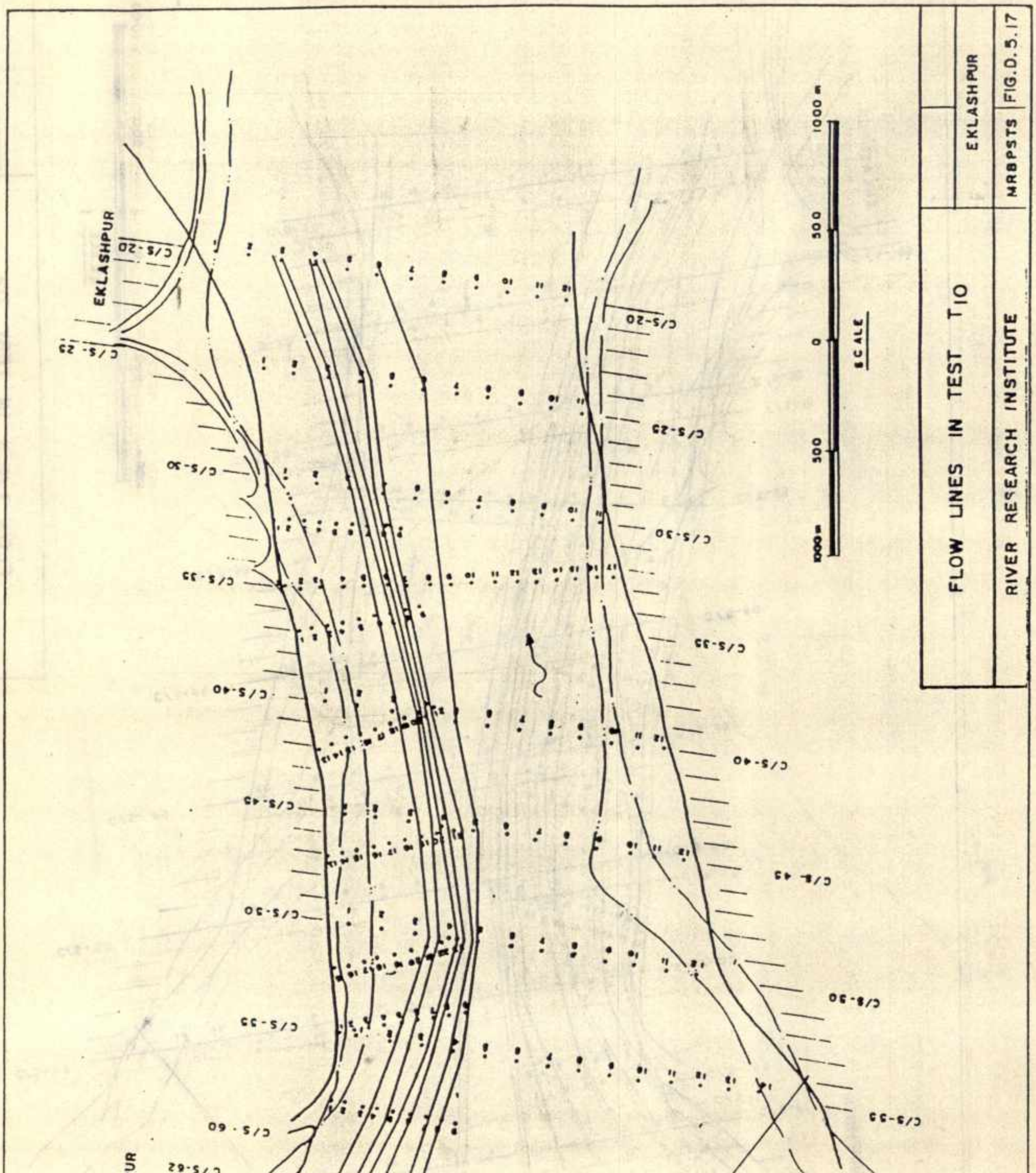
MRBPSTS | FIG. D.5.15

76





FLOW VELOCITY DISTRIBUTION AT CROSS SECTION-35		DATE.
	EKLASHPUR	
	MRBPSTS	FIG. D.5.16

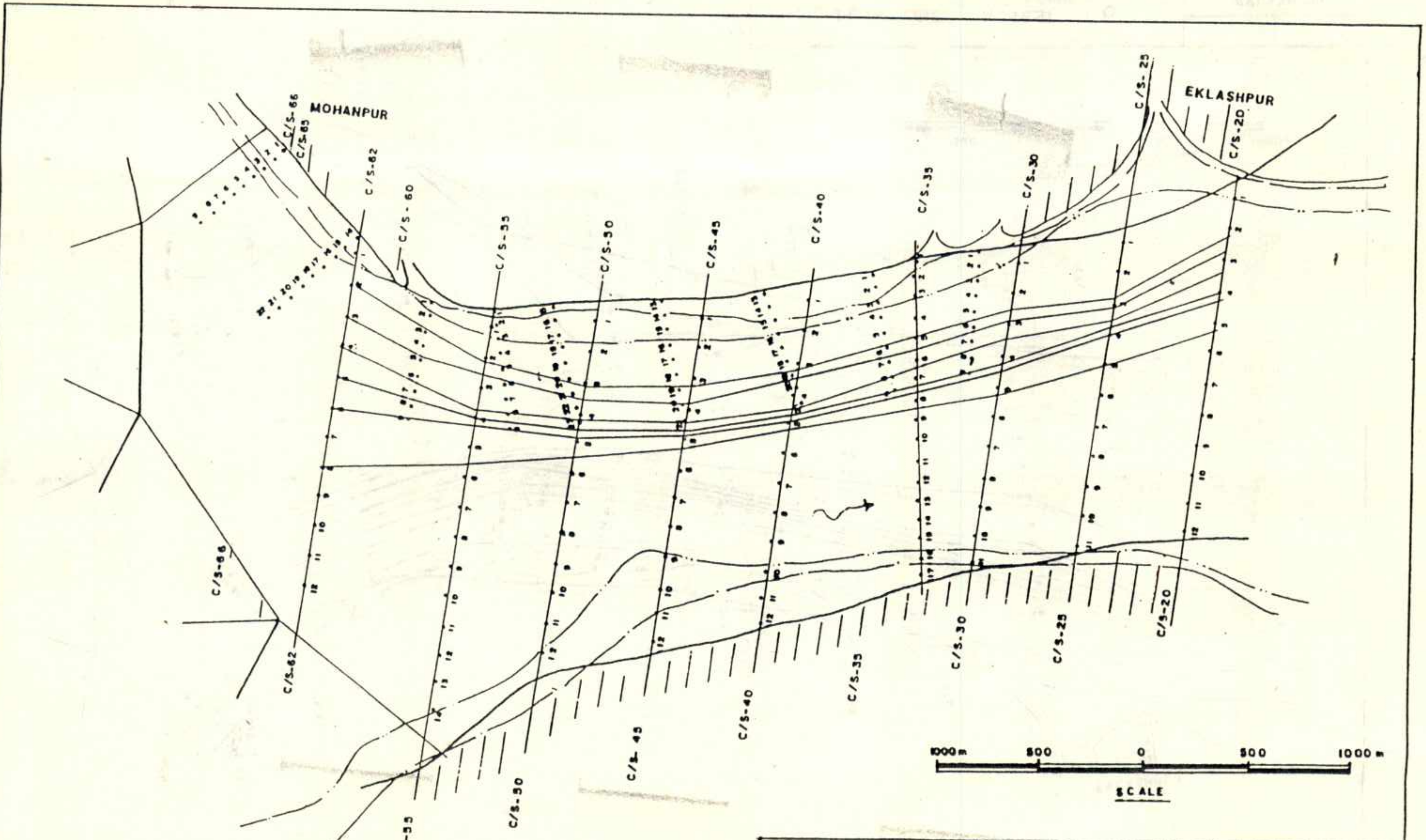


FLOW LINES IN TEST T_{10}

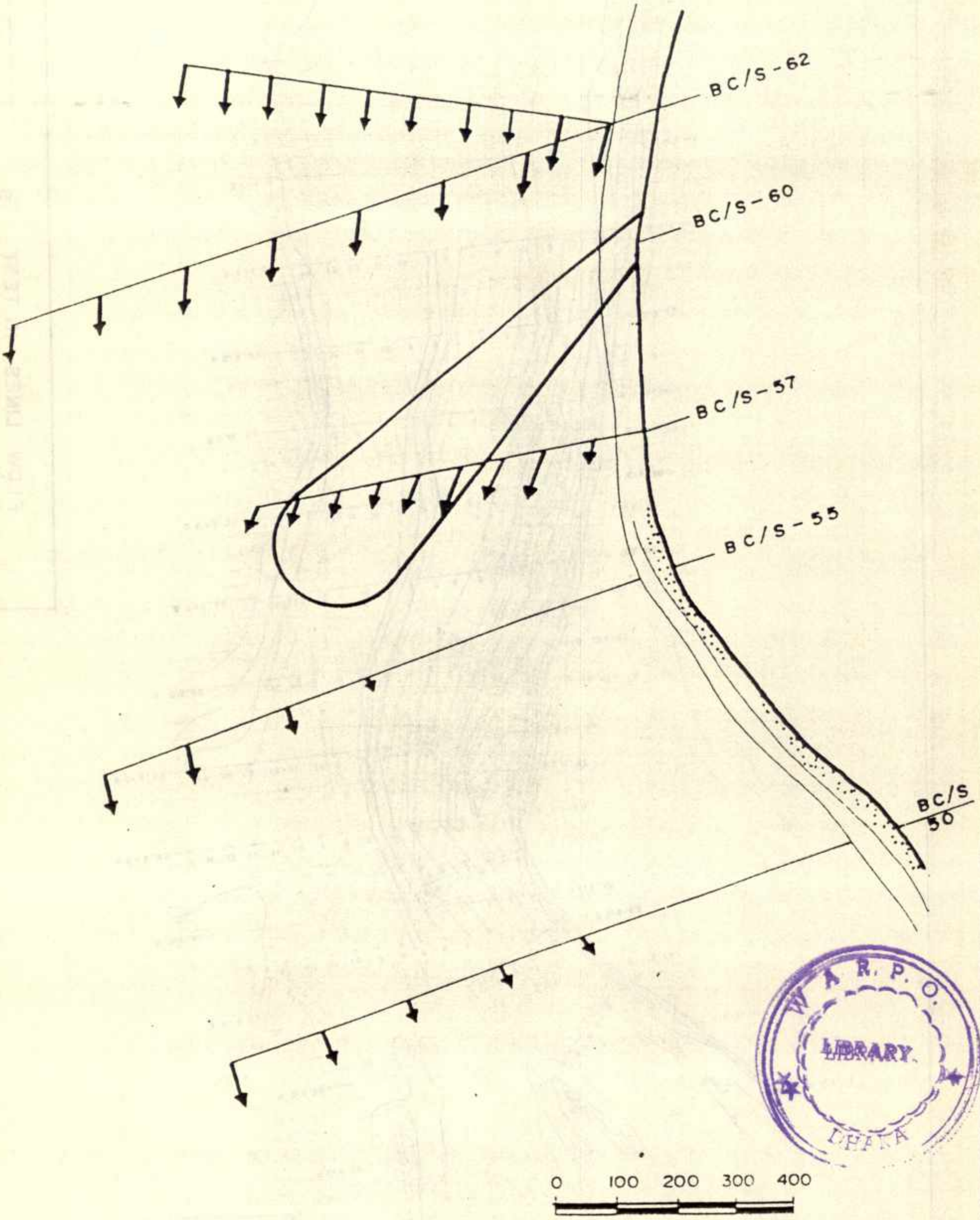
EKLASHPUR

RIVER RESEARCH INSTITUTE

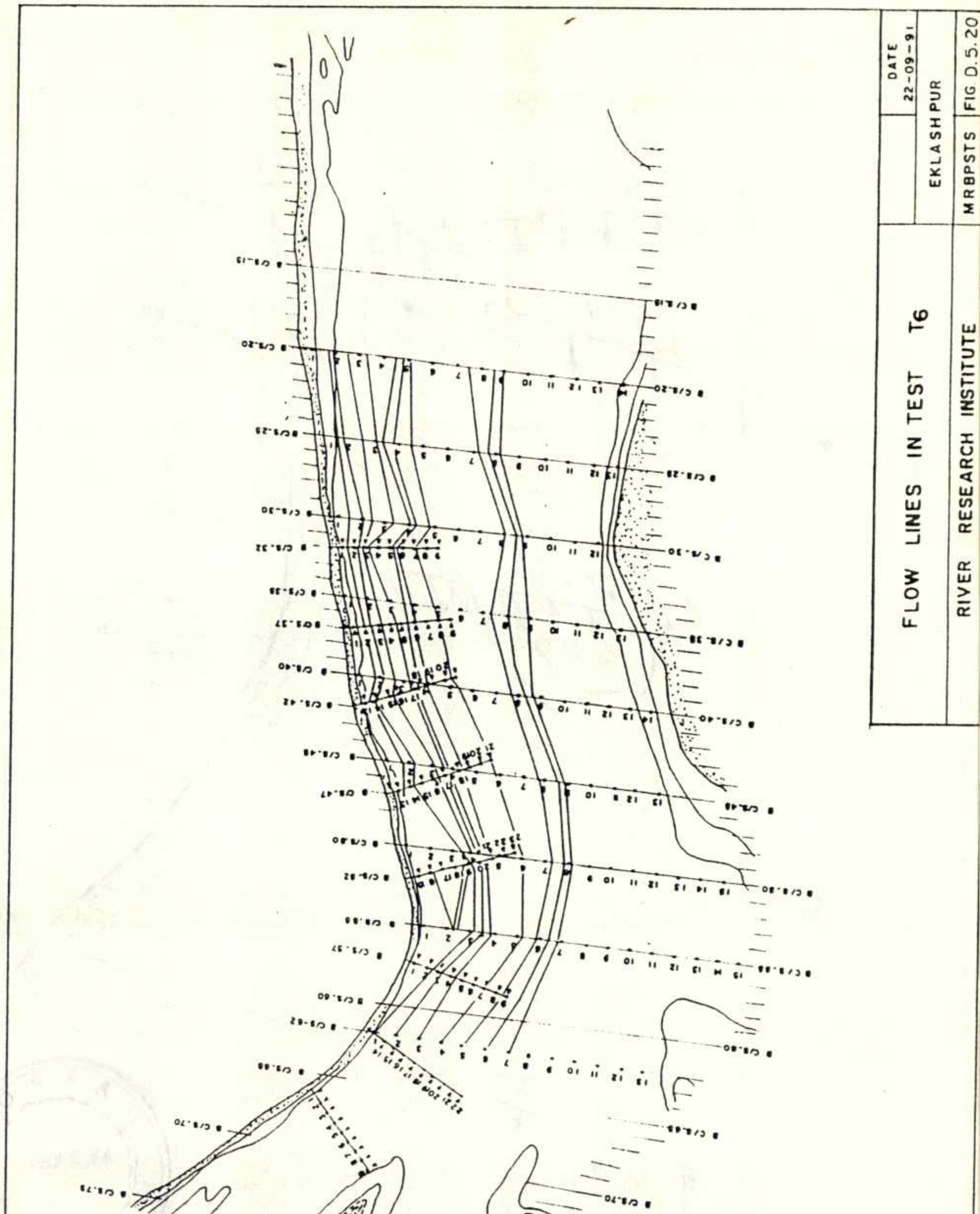
MRBPSTS | FIG.D.5.17

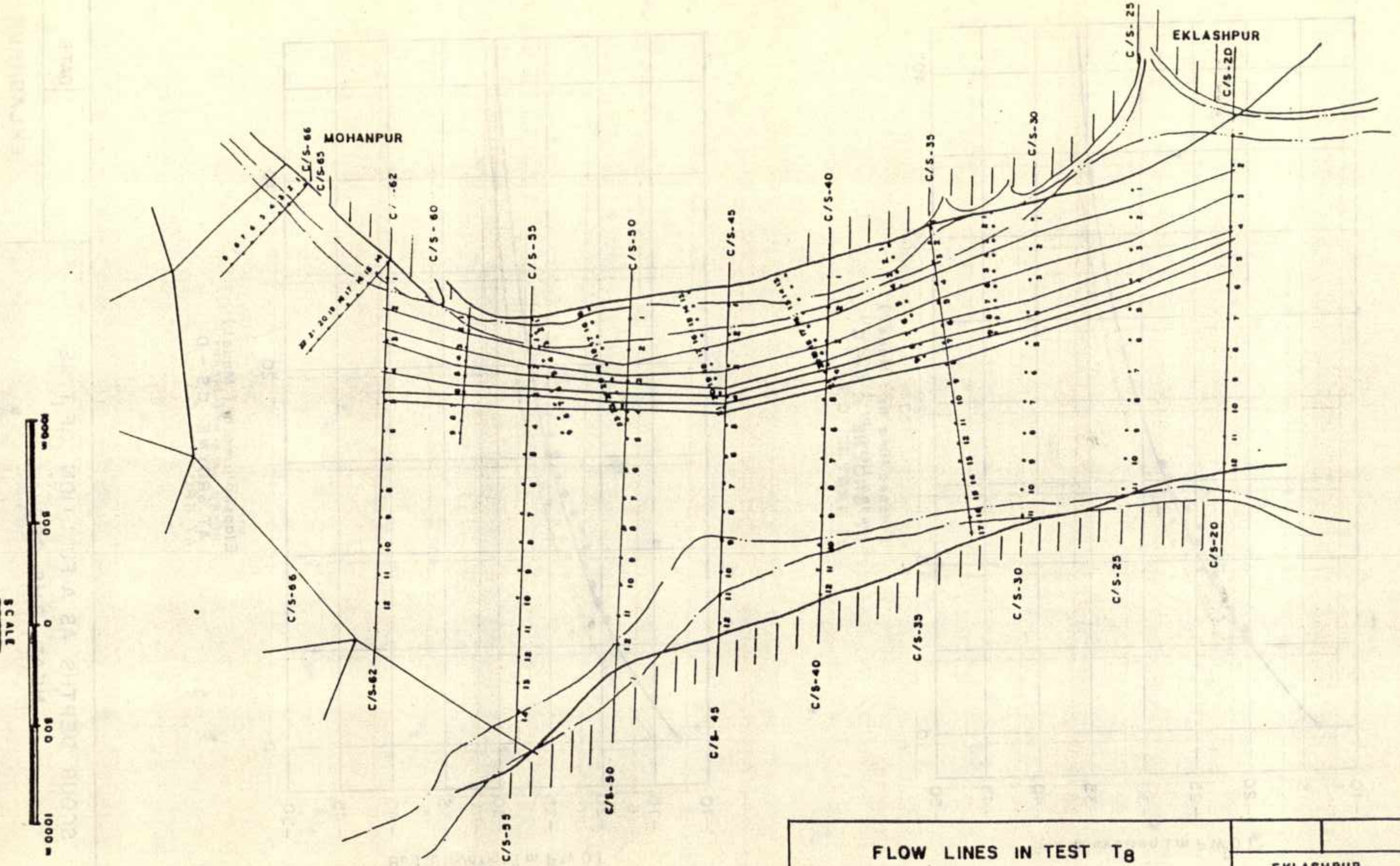


FLOW LINES IN TEST T_{12}	EKLASHPUR
RIVER RESEARCH INSTITUTE	



FLOW VELOCITIES AROUND THE GROYNE IN T6		DATE 21-09-91
	EKLASHPUR	
	MRBPSTS	FIG. D.5.19


FLOW LINES IN TEST T6
RIVER RESEARCH INSTITUTE
DATE
22-09-91
EKLASHPUR
MRBPSTS
FIG D.5.20
SCOUR DEPTH
FLOW VELOCITY
0
500
1000 m
SCALE



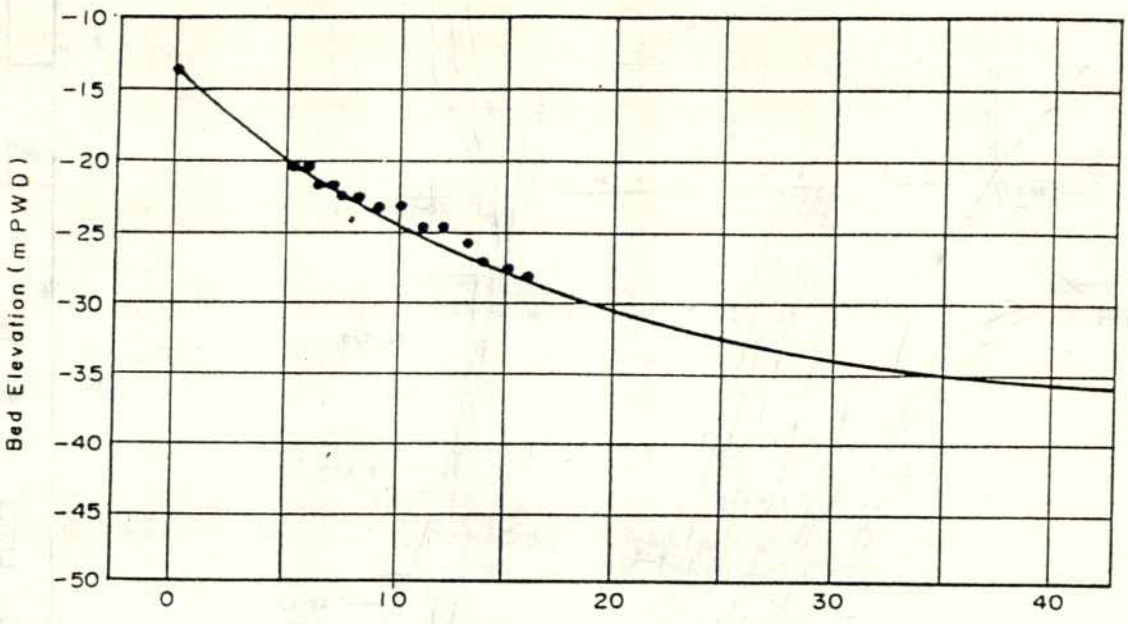
FLOW LINES IN TEST T_8

RIVER RESEARCH INSTITUTE

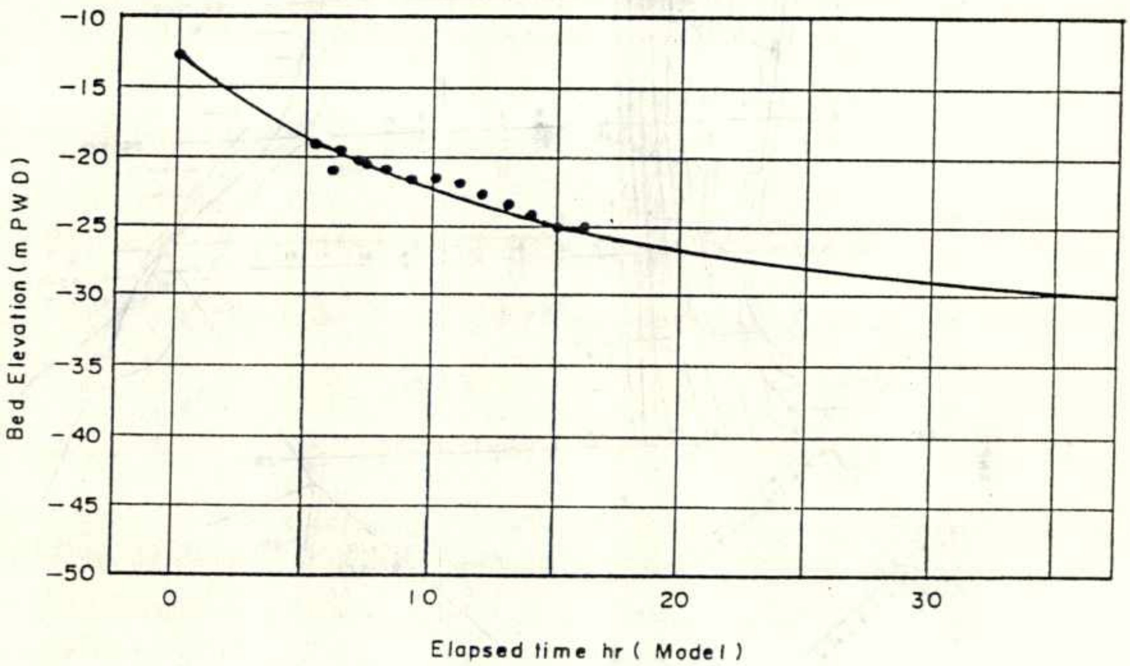
EKLASHPUR

MRBPSTS | FIG. D.5.21

269



AT GROYNE CS – C



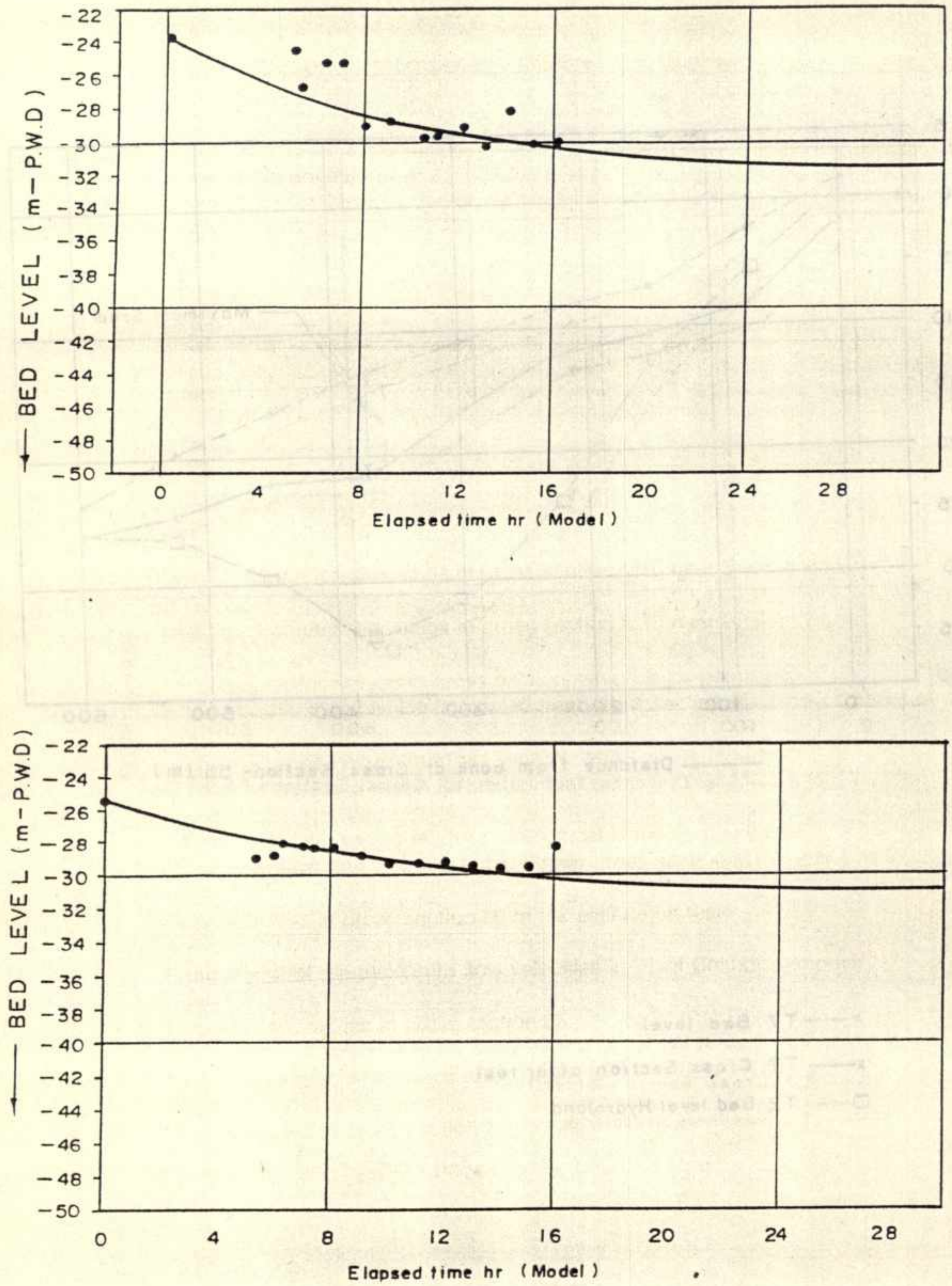
AT GROYNE CS – D

SCOUR DEPTHS AS A FUNCTION OF TIME NEAR THE GROYNE		DATE.
	EKLASHPUR	
	MRBPSTS	FIG. D. 5. 22

216

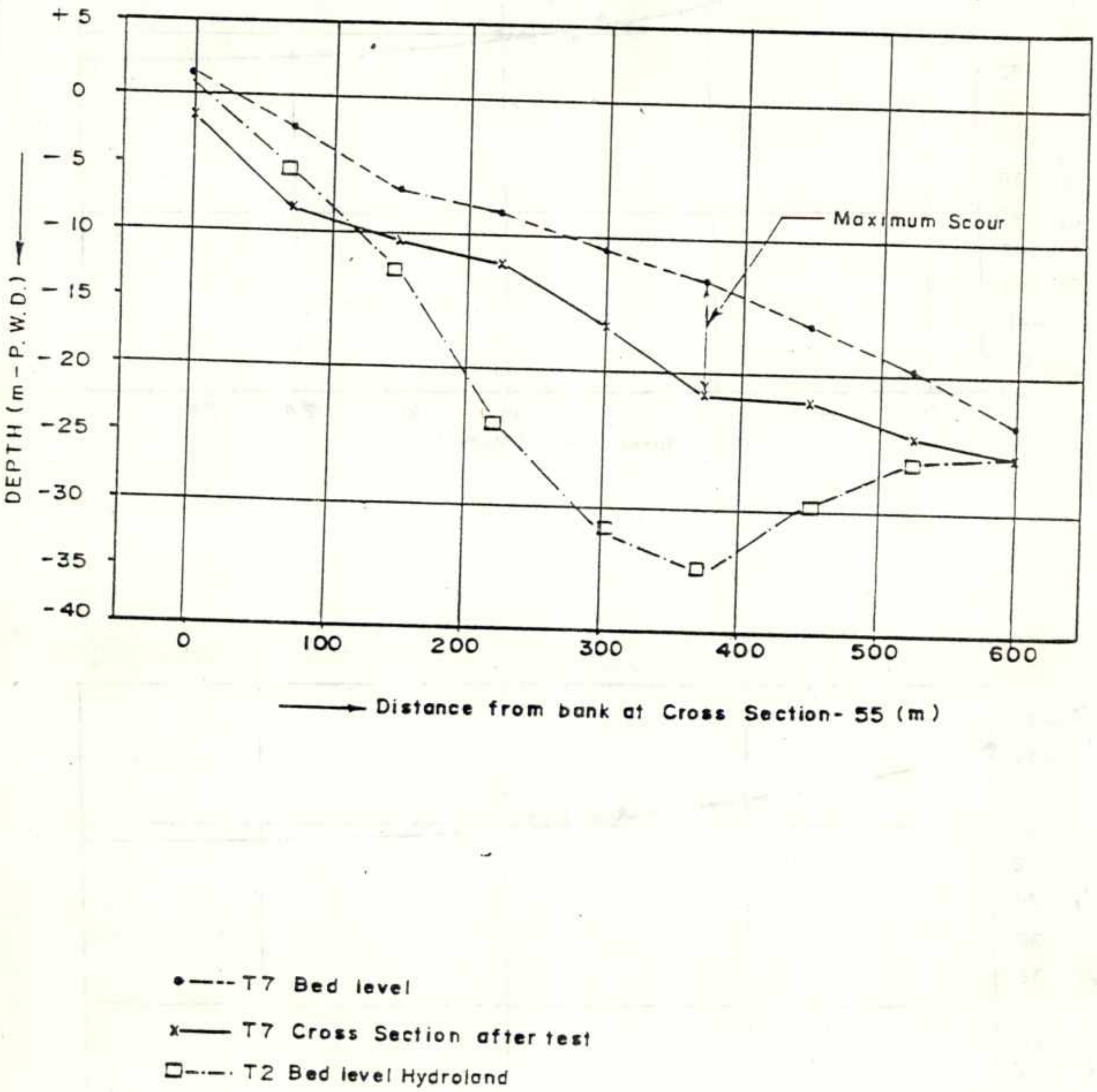
APPENDIX D/1

SCALING PROCEDURES



SCOUR DEPTHS AS A FUNCTION OF TIME IN CROSS SECTION	-	DATE
	EKLASHPUR	
	MRBPSTS	FIG. D. 5. 23

2120



BED LEVEL IN CROSS SECTION 55 IN TEST T2 AND T7		DATE.
	EKLASHPUR	
	MRBPSTS	FIG. D. 5.24

APPENDIX D/1: SCALING PROCEDURES

Contents

TABLES

122

D/1.1 **Introduction**

The scaling procedures, which were applied for the scale selection tests of the three physical models of Bhairab Bazar, Chandpur and Eklashpur, are described in more detail in this Appendix. The scaling of a two-dimensional depth-averaged flow field as part of an alluvial river in a scale model is treated separately (Section D/1.2) from the scaling procedure for reproducing the local scour processes (Section D/1.3), because the reproduction of these phenomena requires different scale conditions and scale laws.

The application of the scale conditions to the flow field model of Bhairab Bazar (Section D/1.4) and the local scour models for Chandpúr (Section D/1.5) and Eklashpur (Section D/1.6) is demonstrated by the results of the calibration tests.

The measured parameters in a scale model are equal to the value of that parameter in the prototype divided by a scale factor. The scale factors should satisfy scale laws and scale conditions. The most important scale laws and scale conditions are treated in Section D/1.2.

D/1.2 **Scale laws and scale conditions**

The geometric reduction of the prototype dimensions to those of the scale model means that all other parameters have to be scaled down to a certain extent. The various scales are determined from the physical phenomena involved.

The scale (or scale factor) n_x of any parameter x is defined by

$$n_x = \frac{\text{value of x in prototype}}{\text{value of x in model}} = \frac{x_p}{x_m}$$

Definition of n_x in this way implies that in most cases $n_x > 1$. This facilitates mental arithmetic.

There are two important methods for deriving the scales for the various parameters involved :

(i) dimensional analysis, which gives dimensionless parameters such as Froude number, Reynolds number, etc.

(ii) mathematical description of the physical phenomena involved, from which scale relations can be obtained.

In general, the second method is to be preferred because it may give insight into the magnitude of scale effects. Scale effects are said to be present if n_x is not only a function of other scales but also varies in the model. Scale effects hinder correct interpretation of model results.

The two methods to determine scale relations can be illustrated by the flow over a sill, see Figure D/1.1. Dimensional analysis leads to two dimensional products, i.e.

- the Froude number[1]

[1] both numbers have to keep the same values in prototype and model.

2026

$$F = u/\sqrt{g\,h}$$

in which

F = Froude number (-)
g = acceleration by gravity (m/s^2)
h = water depth (m)
u = depth averaged flow velocity (m/s).

- the Reynolds number Re,

$$Re = \frac{u\,.\,h}{\nu}$$

in which

Re = Reynolds number (-)
u = average flow velocity, (m/s)
h = water depth, (m)
ν = kinematic viscosity. (m^2/s)

As $n_g = n_\nu = 1$ this leads only to the trivial solution $n_u = n_h = 1$ (i.e. full-scale). Because viscosity does not play a large role in the flow over a sill, it is acceptable to take $n_{Re} \neq 1$. Then the single scale relation

$$n_u = \sqrt{n_h}$$

is obtained. However, no insight is obtained into what happens if $n_u \neq \sqrt{n_h}$. This can only be obtained from the mathematical description of the flow-problem, again neglecting the influence of viscosity.

From Figure D/1.1 it follows that (Bernoulli law)

$$H = h + u^2/(2.g)$$

If $s = u^2/(2.g)$ is taken for convenience, then it can easily be shown that

$$\frac{n_H}{n_h} = \frac{1 + \dfrac{n_s\, s_m}{n_H\, h_m}}{1 + \dfrac{s_m}{h_m}} = \frac{1 + n_F^2\, \dfrac{s_m}{h_m}}{1 + \dfrac{s_m}{h_m}}$$

Hence $n_H = f\{n_h, n_s \text{ and } s_m/h_m\}$. This simplifies to $n_h = n_H = n_s$ if $n_s = n_h$ is assumed. The latter is nothing more than the Froude condition.
Thus the mathematical description not only leads to the scale condition but also gives quantitative insight into possible scale effects.

It is convenient to divide scale relations into two groups :

(i) Scale conditions, from which deviations can be made although scale effects might then be introduced;

(ii) Scale laws, which are to be fulfilled. These are usually based on definitions that are similar in prototype and model. An example can be taken from the Chezy equation which, in fact, defines the Chezy parameter C:

$$u = C\sqrt{(h.i)}$$

C = Chezy coefficient	$(m^{\frac{1}{2}}\ s^{-1})$
h = water depth	(m)
i = water level gradient	(-)
u = averaged flow velocity	(m/s)

This leads to the scale law

$$n_u = n_C . n_h^{1/2} . n_i^{1/2}$$

This scale relation must be fulfilled : the Chezy coefficient has to be defined in the same way in model and prototype.

The distinction is important in practice, since a compromise has to be found amongst the various scale relations when selecting the individual scales. Scale conditions and scale laws must then be distinguished in order to find a combination of scales different from the trivial solution of a full-scale model.

The two examples given above, the Bernoulli and Chezy examples, illustrate the use of two different rules for parameters. In these equations the rule of product of parameters has been employed: if $z = x.y$ then $n_z = n_x.n_y$.

This can easily be shown using the foregoing equations.

In the equations of the examples the rule of sum of parameters is valid: if $z = x + y$ then $n_x = n_y$ leads to $n_z = n_x = n_y$.

Scale effects are present for $n_x \neq n_y$

D/1.3 <u>**Flow field**</u>

In a wide, alluvial river the flow field can often be schematized to a two dimensional depth-averaged flow, because in general the width of a cross section of the river is much greater than the depth. This flow field is governed by the continuity equation, a flow equation along a flow line, and a flow equation perpendicular to a flow line.

With the basic rules for the sum and the product of parameters for the determination of scales the following scale conditions are obtained from the flow equation along a flow line:

$$\frac{dh}{dt} + \frac{d}{ds}\left(\frac{u^2}{2g}\right) + \frac{dh}{ds} + \frac{u^2}{C^2\,h} = 0$$

in which

s = ordinate along a flow line (m)

t = time (s)

This formula is a simplified formula for a horizontal bed level.
From this equation and the basic scale relations it can be demonstrated that $n_F = 1$ in which F = Froude number.

In the same way the following relation follows from an other part of the flow equation:

$$n_C = \sqrt{\frac{n_L}{n_h}}$$

This formula is called the roughness condition and this condition is important for the reproduction at scale of a depth averaged flow field.

In case of geometric similitude $n_l = n_h$ the scale of the Chezy coefficient is $n_C = 1$. In general this requires a very flat cemented model bed of the alluvial river. A model bed from sand is often too rough and the Chezy coefficient in the model, C_m, is too low.

Turbulence

A turbulent flow will be generated if the Reynolds number
Re > 600 to 1000.

In the prototype the flow is completely turbulent, and to prevent scale effects in the flow field the flow in the model should be turbulent too.

Near the banks of the model this condition cannot be fulfilled sometimes, see for example the following situation:

$h = 0.05$ m, $\nu = 1.1\ 10^{-6}\ m^2/s$ and $u = 0.03$ m/s.than Re = 1360
This can be considered as the limit for turbulent flow.

D/1.4 Local scour

A first condition for the reproduction of the local scour process in a physical model is correct modelling of the three dimensional flow field. This can be achieved in a non distorted model and $n_F = 1$.

In the local scour models of Chandpur and Eklashpur the most important parameter of the local scour hole is the maximum scour depth. From extensive tests for bridge pier scour with and without upstream supply of sediment the following empirical relationship for the reproduction of the maximum scour depth has been determined:

$$n_{ys} = n_h \quad \text{if } u_m > 2.\ u_{cr}$$

This means that the maximum scour depth does not depend on the flow velocity in the model, if $u_m > 2.\ u_{cr}$. Only the time period in which this maximum scour depth will be reached, will decrease if the model flow velocity increases.

With this relation the scale factor for the scour depth has been calculated for each model, Chandp model, Section D/1.6 and Eklashpur model, Section D/1.7.

In a distorted model, as is Bhairab Bazar model, this relationship for the maximum scour depth can n be applied, because the three dimensional turbulent eddies in the flow field are influenced not only b the depth scale but also by the length scale. This means roughly for the Bhairab Bazar model 60 < n_y < 300. A further refinement of this condition has been deduced from a semi empirical expression for th maximum scour depth according to Dietz. First the formula of Ahmad has been examined, because thi formula has been used in the analysis of the measurements and in the Geomorphologic Study.

In the prototype the maximum depth of a local scour hole can be estimated by applying, for example the formula of Ahmad:

$$y_s = k \cdot u_0^{0.67} \cdot h_0^{0.67} - h_0$$

h_o	=	initial water depth, just upstream of the local scour hole	(m)
k	=	coefficient, (empirical)	(-)
u_o	=	depth averaged velocity, just upstream of the local scour hole	(m/s)
y_s	=	scour depth	(m)

In this formula the scour depth is a function of the water depth and the flow velocity. In a scale model the maximum flow velocity is 2 or 3 times the critical flow velocity for the initiation of motion and hence the parameter u should be replaced by (u - u_{cr}). In the prototype the difference between u and (u - u_{cr}) disappears as u >> u_{cr}. This means that the formula of Ahmad can not be applied in a scale model and that this formula should not be used for the derivation of scale relationships.

The formula of Dietz for the equilibrium scour depth however can be used in prototype circumstances and in a scale model. The formula of Dietz can be used to derive scale conditions in a situation of a two dimensional flow passing a sill or a bed protection without supply of sediment from upstream of the scour hole (so called clear water scour):

$$n_{y_s} = \frac{n_{(u_0 - u_{cr})}}{n_{u_{cr}}} \cdot n_{h_0}$$

in which

u_{cr} = the critical flow velocity (m/s)

In the Bhairab Bazar model the following characteristic values have been determined:

$$n_{(u-u_{cr})} = \frac{1,7 - 0.26}{0.31 - 0.14} = 8.5$$

$$n_{u_{cr}} = \frac{0.26}{0.14} = 1.8$$

n_{h_0} - 60

After substitution of these values the scale factor of the scour depth is: n_{ys} = 280. This value has been used in the analysis of the scour measurements in the Bhairab Bazar model.

D/1.5 Bhairab Bazar model

For the scale selection of the model discharge two tests have been designed. The characteristic data for these two tests, which have been used to define the flow field, flow velocities and the bed topography in the situation of 1989, are given in Tables D/1.1 and D/1.2.

Test T1 has been designed to reproduce the flow field in the model, therefore n_F = 1. From the measurements it is concluded that the model bed is too flat to fulfil the roughness condition. In test T2 the model discharge has been increased.

TABLE D/1.1 TEST T1, CHARACTERISTIC VALUES IN CROSS SECTION M18 NEAR *BHAIRAB BAZAR*

parameter	unit	prototype	scale factor	model
width	m	800	300	2.67
depth	m	16.2	60	0.27
hydr. radius	m	15.5	60	0.26
velocity	m/s	1.7	7.74	0.22
cross. area	m²	12,940	18,000	0.72
discharge	m³/s	22,000	39,000	0.16
slope water level	-	0.000022	0.35	0.00006
Chezy-coeff.	$m^{\frac{1}{2}}/s$	~~90	1.64	~~55
Froude number	-	0.138	1.0	0.138

~~Ke~~

TABLE D/1.2 TEST T2, CHARACTERISTIC VALUES IN CROSS SECTION M18 NEAR BHAIRAB BAZAR

parameter	unit	prototype	scale factor	model
width	m	800	300	2.67
depth	m	16.2	60	0.27
hydr. radius	m	15.5	60	0.26
velocity	m/s	1.7	5.41	0.31
cross. area	m^2	12,940	18,000	0.72
discharge	m^3/s	22,000	98,500	0.22
slope water level	-	0.000022	0.09	0.00023
Chezy-coeff.	$m^{\frac{1}{2}}/s$	~~90	2.24	~~40
Froude number	-	0.138	0.71	0.195

In the design of the test T2 the parameters of the roughness condition have been estimated as follows:

$C_{prototype}$ = 90 $m^{0.5}$/s
C_{model} = 40 $m^{0.5}$/s
n_l = 300
n_h = 60

From the Chezy equation follows C_{model} = 48 $m^{0.5}$/s:

u = 0.31 m/s, R = 0.12 m and $I_{average}$ = 0.00034.

The White Colebrook formula gives C_{model} = 48 $m^{0.5}$/s if it is assumed that the hydraulic radius = 0.12 m and the hydraulic roughness k_s = 0.003 m. This seems to be a realistic assumption.

If these values are substituted in the roughness condition, it can be concluded that in this model the roughness condition is almost fulfilled in test T2. The increase in the model discharge has resulted in an improvement of the flow field: roughness condition fulfilled and no serious deviation in the flow field by deviating from the Froude condition. In a flow field model of a river reach a deviation from the Froude condition can be accepted, because the variation in the water level along a reach is normally rather small. Only near the head of a groyne or near an abutment of a bridge the local acceleration of the flow results in a considerable local depression in the water level. If the Froude number in the model is too high, than this depression will be relatively too deep in the model. In the Bhairab Bazar model this will be acceptable, because this effect has a very small influence on the two parameters of interest: the flow velocities and local scour depths.

Also the scale effect on the gradient in the water level in a cross section, which is located in a river bend, is relatively small. A deviation in the model of this gradient is acceptable.

In test T2 downstream of the Railway Bridge the average flow velocities in the model fulfil the condition $u > 2\ u_{cr}$ and the local scour process is reproduced at $n_{y,s}$ = 280. Upstream of the railway bridge the width of the river is more than 800 m and consequently the flow velocities are lower than downstream of the railway bridge and the local scour holes are not well reproduced. However this area is of no interest for the alternatives, with exception of the upstream groyne. The concentration of the flow lines near the head of groyne is sufficient to induce a local scour hole at scale in these tests with an upstream groyne.

D/1.6 <u>Chandpur model</u>

The geometric length scale of the undistorted models of Chandpur and Eklashpur is 150. The average channel depths are 20 to 25 m in the prototype and consequently 0.13 to 0.17 m in the model. The maximum scour depth in the model will be between 0.30 and 0.40 m. The width of the main channel is around 10 m in the model. The total length of the model is about 30 m. The model size has been selected taking into account the constraints by model discharge and the available model area. The size of the model suits its purposes: to study the flow field and the local scour holes for alternative solutions.

Tests for calibration

For the scale selection of the model discharge three tests have been designed.
The characteristic data for these tests, which will be used to define the flow field, flow velocities and the bed topography in the situation of 1991, are given in Tables D/1.3, D/1.4 and D/1.5. In test T1 the model discharge was scaled according to the Froude condition to reproduce the flow field. The value of the water level gradient is affected by the wide inflow section the narrow area near Nutan Bazar and the wide outflow area therefore C_{model} has been estimated without using the Chezy equation.

In test T1 no bed erosion could be observed, as expected. Therefore the model discharge has been increased in T2 and T3. It has been checked that this increase in the model discharge did not induce unacceptable changes in the flow pattern.

TABLE D/1.3 T1, CHARACTERISTIC VALUES IN A CROSS SECTION, WHICH IS PERPENDICULAR TO THE FLOW LINES AND WHICH IS LOCATED NEAR CROSS SECTION M1-1 NEAR CHANDPUR

parameter	unit	prototype	scale factor	model
length	m	4500	150	30
width	m	1100	150	7.33
depth	m	32.7	150	0.22
hydraulic radius	m	31	150	0.21
water level in Chandpur	m+PWD	5.40		0.036
velocity: main	m/s	1.9	12.2	0.16
char	m/s	0.8	12.2	0.06
cross. area	m^2	36,000	22,500	1.33
discharge:total	m^3/s	130,000	270,000	0.255
section		70,000		
slope water level		0.00001	0.3	0.0003
Chezy-coeff.	$m^{\frac{1}{2}}/s$	~~125	2.1	~~60
Froude number	-	0.11	1	0.11

২০১

TABLE D/1.4 T2, CHARACTERISTIC VALUES IN CROSS SECTION, WHICH IS PERPENDICULAR TO THE FLOW LINES AND WHICH IS LOCATED NEAR CROSS SECTION M1-1 NEAR CHANDPUR

parameter	unit	prototype	scale factor	model
length	m	4500	150	30
width	m	1100	150	7.33
depth	m	35	150	0.23
hydraulic radius	m	33	150	0.22
water level in Chandpur	m	5.40	150	0.036
velocity: main char	m/s m/s	2.3 0.8	9.76 9.76	0.24 0.08
cross. area	m²	38,500	22,500	1.71
discharge:total section	m³/s	130,000 90,000	219,500	0.410
slope water level		0.00001	0.14	0.00007
Chezy-coeff.	$m^{\frac{1}{2}}/s$	~~125	2.1	~~60
Froude number	-	0.128	0.8	0.163

TABLE D/1.5 T3, CHARACTERISTIC VALUES IN CROSS SECTION, WHICH IS PERPENDICULAR TO THE FLOW LINES AND WHICH IS LOCATED NEAR CROSS SECTION M1-1 NEAR CHANDPUR

parameter	unit	prototype	scale factor	model
length	m	4500	150	30
width	m	1100	150	7.33
depth	m	35	150	0.23
hydr. radius	m	33	150	0.22
water level in Chandpur	m+PWD	5.40		0.036
velocity: main char	m/s m/s	2.3 0.8	7.84 7.84	0.30 0.10
cross. area	m^2	38,500	22,500	1.71
discharge:total section	m^3/s	130,000 90,000	176,500	0.510
slope water level		0.00005	1	0.00005
Chezy-coeff.	$m^{\frac{1}{2}}/s$	~~90	1.64	~~55
Froude number	-	0.128	0.63	0.204

The total river discharge is 130,000 m^3/s and it is estimated that only 90,000 m^3/s flows through the modeled section of the river. It is remarked that during this high discharge the right and the left bank are flooded.

During the test program also other discharges have been selected in the range of 0.410 and 0.510 m^3/s:

discharge velocity: main	m^3/s m/s	90.000 2.3	204,500 9.09	0.400 0.2
discharge velocity: main	m^3/s m/s	90.000 2.3	200,000 8.89	0.45 0.26

By changing the model discharge also the scale of the flow velocities is changed as indicated above.

For a detailed calibration of the flow field the bamboo screen had to be adjusted in such a way that the correct flow velocity distribution is obtained in cross section 50 5 to 10 m downstream of the bamboo screen. This flow velocity distribution has been computed with the mathematical model, see Table D/1.6.

TABLE D/1.6 FLOW VELOCITIES IN CROSS SECTION 50 IN THE CALIBRATION TESTS

distance from the right bank of char in cross section 50		flow velocities calculated			
prototype	model	prototype	test T1	test T2	test T3
m	m	m/s	m/s	m/s	m/s
0	0.00	0.8	0.07	0.08	0.10
220	1.47	1.1	0.09	0.11	0.14
580	3.87	1.4	0.11	0.14	0.18
720	4.80	1.4	0.11	0.14	0.18
910	6.07	1.55	0.13	0.16	0.20
990	6.60	1.7	0.14	0.17	0.22
1080	7.20	1.9	0.16	0.19	0.24
1130	7.53	2.1	0.17	0.22	0.27
1210	8.07	2.3	0.19	0.24	0.29
1300	8.67	2.5	0.20	0.26	0.32
1570	10.47	2.8	0.23	0.29	0.36
1820	12.13	2.7	0.22	0.28	0.34

An important aspect of these calibration tests is to check the deviation from the roughness condition and whether the initiation of sediment transport upstream of the scour hole can be observed. After the first three preliminary tests a model discharge of 0.410 m3/s was selected. For a more accurate calibration T2 was repeated and the results are compared with the results of the mathematical model:

- flow lines:
  The flow lines in T2 show a rather good similarity with the calculated flow lines. However, in the mathematical model the upstream right channel is more pronounced than in the scale model. In the scale model the radius of the curvature of the flow lines is smaller than in the mathematical model due to the schematization in the mathematical model and due to the high roughness of the scale model bed.

- maximum velocity:
  In several cross sections the flow velocity distribution has been measured, and from these distributions the maximum flow velocity has been determined, see Table D/1.7. These velocities are compared with the maximum flow velocities in the mathematical model.

208

TABLE D/1.7 COMPARISON OF THE MAXIMUM FLOW VELOCITIES IN T2 OF CHANDPUR MODEL

cross section	scale model $\hat{u}$	math. model $\hat{u}$
	m/s	m/s
50	2.9	2.6
42	2.9	3.0
32 *	3.9	2.5
29 *	4.0	3.1
24 *	3.8	2.8
17	3.6	3.7
10	3.4	3.4

* in these cross sections the schematization of the bed in the mathematical model is not detailed enough.

From Table D/1.7 it can be seen that at the inflow boundary and the outflow boundary of the model the maximum flow velocity in the scale model is quite similar to the maximum flow velocity in the mathematical model.

A turbulent flow will be generated if the Reynolds number Re > 600 to 1000. The following assumed characteristic values, minimum flow velocity:u = 0.1 m/s, h = 0.15 m and ν = 1.1 10^{-6} m^2/s, will result in Re = 900 > 600. In the area of interest the flow velocity and the water depths will be higher, and therefore the flow will be fully turbulent in this area. The turbulence in the flow will be increased by the ripples on the bed, if the flow exceeds the critical flow velocity (around 0.2 m/s) for the model sand. Only in some shallow areas near the banks the Reynolds number of the flow may be too low. This small scale effect has a negligible influence on the model results and can be accepted.

From the compared flow velocities, flow lines and the roughness it is concluded that in the calibration test T2 the flow field is reproduced in the model with sufficient accuracy.

The size of the model sand is approximately the same as the size of the prototype sand. For a fair reproduction of the local scour holes in the model it is required that the flow velocity upstream of the scour hole is higher than two times the critical flow velocity for the initiation of motion of the sand. This requirement is a reason to deviate from the Froude condition. Consultants prefer to increase the model discharge and to use prototype sand in the model, instead of fulfilling the Froude condition by the selection of light weight material. Probably some properties of the light weight material differ from the sand and therefore new scale effects are introduced.

D/1.7 <u>Eklashpur model</u>

The characteristic data for two calibration tests, which will be used to define the flow field, flow velocities and the bed topography in the situation of 1991, are given in Tables D/1.8 and D/1.9. For the scale selection of the model discharge two tests have been designed. In T1 the model discharge was calculated with n_F = 1. In the next test, T2, the model discharge was increased, to create some sand transport on the model bed.

TABLE D/1.8 T1, CHARACTERISTIC VALUES IN CROSS SECTION 35

parameter	unit	prototype	scale factor	model
length	m	5500	150	36.7
width	m	1500	150	10.0
depth	m	18.3	150	0.122
cross area	m^2	27500	22,500	1.22
velocity	m/s	1.56	12.25	0.13
discharge Q	m^3/s	43,000	275,000	0.160
Q weir 1	m^3/s	0	275,000	0
Q weir 2	m^3/s	8,600	275,000	0.031
Q weir 3	m^3/s	12,900	275,000	0.047
Q weir 4	m^3/s	21,500	275,000	0.078
water level cross sect 35	m + PWD	6.20	150	0.041
slope water. level	-	0.00005	1	0.00005
Chezy-coeff.	$m^{1/2}/s$	~90	2	~45

A remark about the discharge distribution over the weirs:

The preliminary design of the four weirs at the inflow boundary has been based on the field survey of 1989. The main channel geometry, which has been measured in that field survey, has been compared with the main channel geometry, which has been measured in the field survey of 1991. From this comparison it follows that the main channel geometry may have changed in these two years. Therefore the discharge in weir 1 is now estimated to be zero, based on the flow lines, which have been computed with the mathematical model. However, weir 1 has been used in one of the geomorphologic alternatives.

204

TABLE D/1.9 TEST T2, CHARACTERISTIC VALUES IN CROSS SECTION 35

parameter	unit	prototype	scale factor	model
length	m	5500	150	36.7
width	m	1500	150	10.0
depth	m	18.3	150	0.122
cross area	m2	27,500	22,500	1.22
velocity	m/s	1.56	5.2	0.30
discharge	m3/s	43,000	117,000	0.370
Q weir 1	m3/s.	0	117,000	0
Q weir 2	m3/s	8,600	117,000	0.074
Q weir 3	m3/s	12,900	117,000	0.111
Q weir 4	m3/s	21,500	117,000	0.184
water level cross sect 35	m + PWD	6.20	150	0.041
slope water. level	-	0.00005	1	0.00005
Chezy-coeff.	m0.5/s	~ 90	2	~ 45

The total river discharge is 130,000 m^3/s during the 1 in 100 years condition. (see Annex A). It is estimated that only 43,000 m^3/s flows through the modelled section of the river. During this high discharge the banks are flooded.

The discharge distribution in cross section 35 has been calculated by the mathematical model also, see Table D/1.10. These flow velocities should be used for a detailed calibration of the model. In the detailed calibration the required flow velocity distribution will be obtained by small adjustments in the bamboo screen in the upstream boundary of the model.

TABLE D/1.10 FLOW VELOCITIES IN CROSS SECTION 35 FOR CALIBRATION PURPOSES

distance from the left bank		prototype flow velocity	test T1 flow velocity	test T2 flow velocity
m prototype	m model	m/s	m/s	m/s
10	0.07	0.15	0.01	0.03
70	0.47	0.45	0.04	0.09
170	1.13	0.70	0.06	0.13
270	1.80	0.80	0.07	0.15
370	2.47	1.10	0.09	0.21
470	3.13	1.60	0.13	0.31
570	3.80	1.80	0.15	0.35
670	4.47	2.10	0.17	0.40
770	5.13	2.20	0.18	0.42
870	5.80	2.00	0.16	0.38
970	6.47	1.90	0.16	0.37
1070	7.13	1.85	0.15	0.36
1170	7.80	1.85	0.15	0.36
1270	8.47	1.85	0.15	0.36
1370	9.13	1.75	0.14	0.34
1470	9.80	1.45	0.12	0.28
1570	10.47	0.90	0.07	0.17

An important aspect of these calibration tests is to check a possible deviation from the roughness condition and to check the initiation of sediment transport upstream of the scour holes. The initiation of sediment transport can be observed from the ripple pattern of the bed.