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BANK PROTECTION AND RIVER TRAINING (AFPM) PILOT PROJECT

FAP 21/22

COPY NO. 8

INTERIM REPORT

Volume III : Annex 2 - 10

- Annex 2: Morphological Analysis Using Satellite Imagery
Annex 3: Present State of Scour Study
Annex 4: Hydrologic Data
Annex 5: Simplified Mathematical Model for Investigation
of Recurrent AFPM Measures
Annex 6: Filter Test Rig
Annex 7: Examples of Survey Drawings
Annex 8: Subsoil Investigation Results
Annex 9: Economic Aspects
Annex 10: 1-Dimensional Modelling

JULY 1992



CONSULTING CONSORTIUM FAP 21/22

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

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ANNEX 2

MORPHOLOGICAL ANALYSIS USING SATELLITE IMAGERY

MORPHOLOGICAL ANALYSIS USING SATELLITE IMAGERY

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1 GENERAL

In the present Project an extensive study component is included that uses satellite imagery to improve the understanding of morphological processes in braided rivers with fine sand as bed and bank material. This study serves two purposes:

- (1) to provide the best possible predictions of where the test works to be built under FAP 21 have to be sited;
- (2) to improve the prediction methods for where bank erosion is to be expected, as the basis for decisions as to where protective measures may be needed.

Some background for this study is provided hereafter.

Within the framework of FAP 21 bank protection works will be built along the Jamuna River. These protection works should also be "tested" by the river, hence they have to be built at sites where over the coming years the river will attack its banks. So for the selection of the sites of the test works it is required to predict the location of serious bank erosion over the coming years.

In fully meandering rivers the location of bank erosion can fairly well be predicted at this moment, and using a sequence of maps or satellite images also the celerity of bank erosion can be deduced. Recently this has resulted in the development of numerical models for the prediction of changes in planform of meandering rivers (e.g. Johannessen and Parker, 1985 or Crosato, 1991). In addition methods have been developed to predict the occurrence of cutoffs in meandering rivers (Klaassen & van Zanten, 1989). Although the reliability of the predictions with these tools is still fairly limited, they represent a major step forward as far as the planning of bank protection works along meandering rivers is concerned. The development of these tools has been possible because of the improved understanding of processes active in meandering rivers.

For braiding rivers the insight into the controlling processes has not yet reached such a level that predictive models can be developed. What is lacking to start with is a full understanding of the processes that are active in such a braided river with fine sand as bed and bank material. Processes that have been identified are e.g. bank erosion along curved reaches, cutoffs that seem to be controlled by the cutoff ratio and the geometry of the bifurcation and a number of other items (see Klaassen & Masselink (1992) and Annex 4 of the second Interim Report of the Brahmaputra River Training Studies).

Hence, bits and pieces of the complete understanding are gradually being assembled like relations for bank erosion as a function of the channel characteristics and the factors governing natural cutoffs under the Jamuna Bridge project (Klaassen & Masselink, 1992), the notion of stable and moving chars and the life time of very aggressive curved channel reaches under the Brahmaputra River Training Studies.

Improved understanding of the controlling processes in this type of braided rivers is also very much needed for the FAP 22 component of the Project. It is felt that Active Flood Plain Management can only be feasible if the strategy is to strengthen favourable developments and AFPM measures are selected that are working in conjunction *with* the riverine forces and *not against* them. Hence they should be used at times and at places where enforcement of natural tendencies leads to a favourable development. To identify the preferable locations and the appropriate times a good insight in the morphological processes is needed.

The above is the rationale for carrying out a fairly extensive analysis of a sequence of satellite images, aiming at improving the insight into channel processes and their importance for bank erosion processes. Broadly speaking the present study can be divided in two parts, notably:

- (i) one part dealing with the identification of the best sites for constructing the test works, and
- (ii) another part dealing with processes in Jamuna type of rivers, under which a.o. the following topics are being studied:
 - the celerity of bank erosion in relation to channel characteristics (as a continuation of the work carried out within the framework of the Jamuna Bridge Project and Brahmaputra River Training Studies).
 - the occurrence of cutoffs, in relation to the cutoff ratio and the geometry of the bifurcation.

The approach followed is discussed in this annex and some results are presented. The study of the processes active in a braided river with fine sand as bed and bank material is not yet completed, so some conclusions are only preliminary, in particular as to the interpretation of the results obtained. In a later stage more definitive conclusions will be drawn, probably to be published as a separate Technical Report.

In the following first the available data on which the further analysis is based are described briefly. Next some preliminary results of the analysis of morphological processes in the Jamuna River are presented. Finally an analysis is made of the recent river morphological processes near the pre-selected sites for the test works. The emphasis is on the probability that in the near future the test works would be "tested" and the predictability of the future conditions. Furthermore recent developments in the river are reviewed to verify whether other test areas can be identified that are also promising in this respect.



2 AVAILABLE DATA

For the present study mainly satellite images are being used. In addition, however, also cross-sections measured by BWDB were processed. Furthermore some sounding charts as prepared by BIWTA are being used. In the following sections these data are described.

2.1 SATELLITE IMAGES

2.1.1 Selection

The study discussed here, based on the comparison of satellite images of the Jamuna River for successive years for the period 1973-1992, is essentially the same as the approach used in the Jamuna Bridge Project and later in the Brahmaputra River Training Studies.

For the analysis mostly LANDSAT images were selected. Partly these images are (older) MSS images (with resolution of 80 x 80m) and partly the more recent TM images (with a resolution of 30 x 30m). The origin of the data is diverse: part had already been acquired under the Jamuna Bridge Project, some processed tapes were provided by FINNMAP, other processed data were obtained from FAP 19 and finally a number of tapes with the digital material was bought from either EOSAT in the USA or the National Remote Sensing Center in Thailand. In addition a number of Japanese MOS images were acquired.

An overview of the images used and their origin is provided in Table 2.1-1. In total 35 images became available (including the ones already processed under the Jamuna Bridge Project) and if required CCT's (Computer Compatible Tapes) were purchased. The images relate to both the lower reach of the Jamuna River (148(138)/43, downstream of Bahadurabad) and the more upstream reach (148(138)/42). See Fig. 2.1-1 for the coverage of the different images.

In line with the Jamuna Bridge Project here, apart from the "normal" criteria for the selection of an image (quality, cloud cover), an additional criterion was used. By preference images were selected in which on the date of the image being acquired, the stage at Sirajganj was around 6.50m + PWD (PWD stands for Public Works Datum, corresponding to an agreed standard level in Bangladesh). This was done to truly isolate the changes in channel location due to bank erosion and not to mix it up with changes in channel width for different stages. It should be stated here that the objective in this respect could not be reached fully because of the limited availability of the imagery. Table 2.1-2 provides an overview of the water levels at the stations Bahadurabad, Sirajganj and Baruria Transit (confluence with Ganges River). The average water level at Sirajganj is almost 7.00m, and varies between 6.16 and 7.68m. Overviewing the images available it can be stated that out of a period of 20 low-water seasons (1972/73 through 1991/92) 15 images became available, notably:

- 1972/73
- 1975/76 through 1979/80 (5 consecutive low-water seasons)
- 1982/83 through 1986/87 (5 consecutive low-water seasons)
- 1988-89 through 1991/92 (4 consecutive low-water seasons)

hence including three periods with fairly long records of 4 to 5 years. For two intervals no images were available (1973/74 through 1974/75 and 1980/81 through 1981/82), while the 1987/88 was finally not purchased to allow for the processing of the latest years.

Winter	Date	Sensor	Scene	Status and remarks
72/73	730221	MSS USA	138-42	Completed
	730221	MSS USA	138-43	Completed
73/74	-----	MSS USA	138-42	*** Not available
	-----	MSS USA	138-43	*** Not available
74/75	-----	MSS USA	138-42	*** Not available
	-----	MSS USA	138-43	*** Not available
75/76	760110	MSS USA	138-42	Completed
	760110	MSS USA	138-43	Completed
76/77	770209	MSS USA	138-42	Completed
	770209	MSS USA	138-43	Completed
77/78	780222	MSS USA	138-42	Completed
	780222	MSS USA	138-43	Completed
78/79	790226	MSS USA	138-42	Completed
	781225	MSS USA	138-43	Completed
79/80	800116	MSS USA	138-42	Ordered and received
	800116	MSS USA	138-43	Ordered and received
80/81	-----	MSS Tha	138-42	*** Not available
	-----	MSS Tha	138-43	*** Not available
81/82	-----	MSS Tha	138-42	*** Not available
	-----	MSS Tha	138-43	*** Not available
82/83	830205	MSS Tha	138-42	Ordered 23-04-92
	830205	MSS Tha	138-43	Ordered 23-04-92
83/84	840223	MSS Tha	138-42	Completed
	840223	MSS Tha	138-43	Completed
84/85	850225	MSS Tha	138-42	Ordered 23-04-92
	850225	MSS Tha	138-43	Ordered 23-04-92
85/86	860220	MSS Tha	138-42	Completed
	860220	MSS Tha	138-43	Completed
86/87	870207	MSS Tha	138-42	Completed
	870207	MSS Tha	138-43	Completed
87/88	880210	TM Tha	148-42	*** Cancelled
	880210	TM Tha	148-43	*** Cancelled
88/89	890303	MOS Jap	51.84W	Completed
	890303	MOS Jap	51.85W	Completed
	890111	MOS Jap	51.85E	Completed
	890111	MOS Jap	51.86E	Completed
89/90	900206	MOS Jap	51.84W	Ordered 10-06-92
	900206	MOS Jap	51.85W	Ordered 10-06-92
	900130	TM Tha	148-43	Completed
90/91	-----	MOS Jap	51.84W	*** Not available
	910424	MOS Jap	51.85W	Completed
	910321	TM Tha	148-43	Completed
91/92	920308	TM Tha	148-42	Completed
	920308	TM Tha	148-43	Completed

Table 2.1-1 Overview of images used in the present study

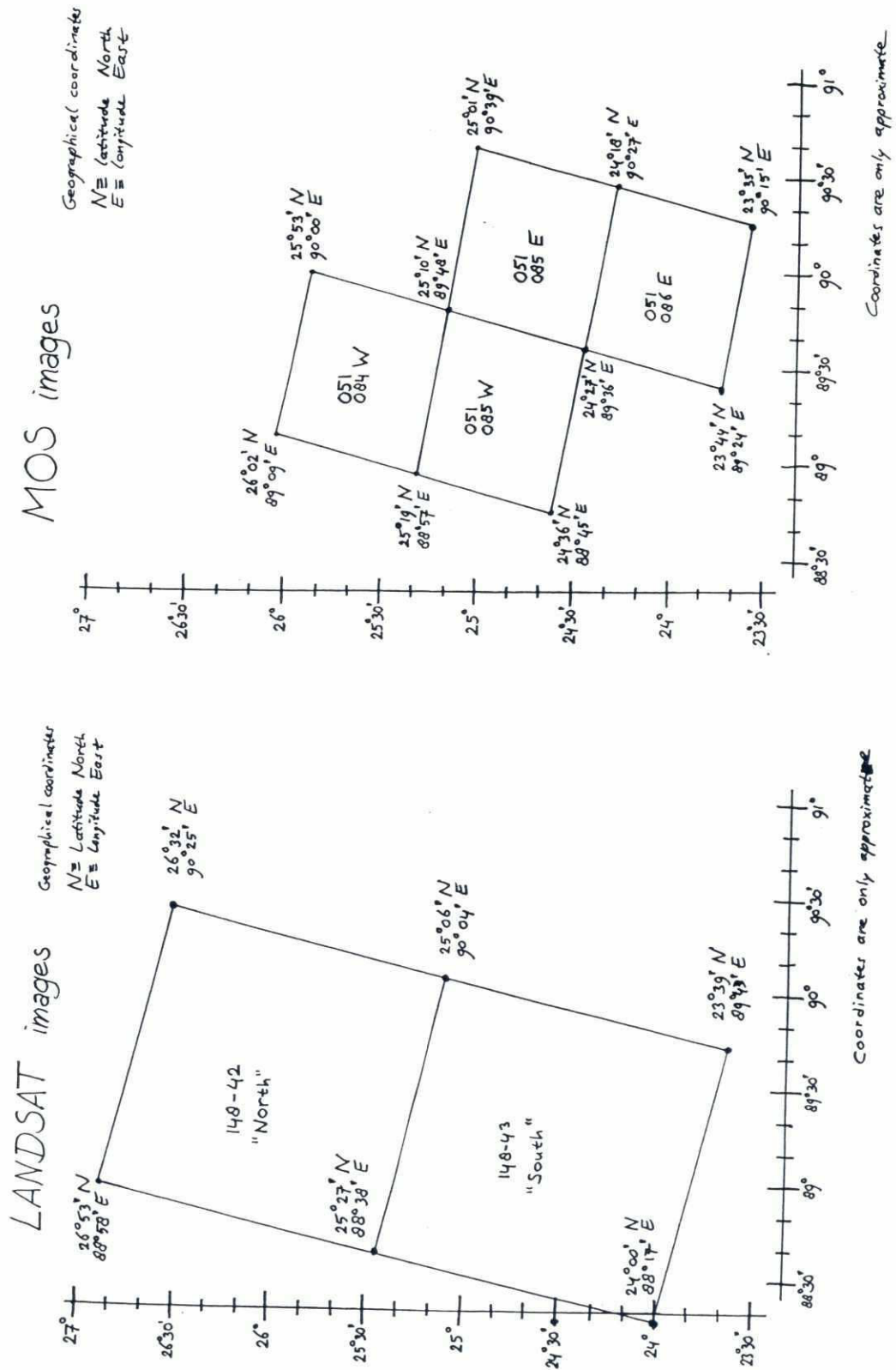


Fig. 2.1-1 Coverage of satellite images

		Water Level (m+PWD)		
Low flow Season	Date	Bahadura- bad	Sirajganj	Baruria (91.9L)
72/73	21-2-73	13.09	6.64	2.51
75/76	10-1-76	13.54	7.14	2.63
76/77	09-2-77	13.09	6.62	1.99
77/78	22-2-78	13.07	6.46	1.96
78/79	25-12-78	13.99	7.24	2.64
	26-2-79	13.11	6.20	1.86
79/80	16-1-80	13.78	6.96	2.14
82/83	05-2-83	13.23	7.00	1.80
83/84	23-2-84	13.12	6.16	1.88
84/85	25-2-85	13.49	6.72	1.78
85/86	20-2-86	13.38	7.04	2.12
86/87	07-2-87	13.30	6.94	2.24
87/88	10-2-88	13.44	7.10	2.25
88/89	11-1-89	14.01	7.84	2.68
	03-3-89	13.94	7.69	2.31
89/90	30-1-90	13.69	7.66	2.23
	06-2-90	13.75	7.65	2.04
90/91	21-3-91	13.69		
	24-4-91			
91/92	8-3-92	13.76		
Average		13.45	6.92	2.15

Table 2.1-2 Water Levels at the dates acquiring the satellite images

2.1.2 Processing

The processing of the CCT's with digital information was done by the National Aerospace Laboratory in The Netherlands by means of the RESEDA processing system. The tapes were read and data were subsequently geometrically corrected and resampled. In doing so the original pixel size of either 80 or 30m was corrected to about 50m. The geometrical correction was done on the basis of some ten points which could be identified on the image of the year 1977 and from which the coordinates could be obtained from topographical maps. The accuracy of this correction was evaluated and the inaccuracy is not greater than the size of the pixels after resampling. All the other images were corrected using the 1977 image as a "base map".

The next step in the analysis is the classification. Conform the Jamuna Bridge project, for the present study four classes were distinguished, i.e. water, bare land, sand and vegetation. The classification was done as the basis of band 5 and band 7. Fig. 2 provides the classification diagram applied for the image for the year 1977. This classification diagram is different from the diagrams for other years, but the shape of the diagram is similar. The following colour code is used:

- blue : water
- grey : bare land
- orange : sand
- green : vegetation

In earlier studies (e.g. Roque, 1983) the class "bare land" was not identified separately and it may be argued that this class can be mixed up with e.g. wet sand. It was found, however, that deleting this class resulted in substantial loss of information on geomorphological features.

Changes over the years in water and land distributions are obtained by comparing the classification for different years. In total 16 possibilities are present, based on the four identified classes. Compared to the Jamuna Bridge study a slightly different code was used to indicate changes. The applied code is indicated in Table 2.1-3

How this table should be read is shown with an example: if in a previous image an certain pixel was identified as vegetated land and in the next image as water then on the figure showing the differences, this pixel is shown with a red colour. Water that has remained surface water is indicated as dark blue. Furthermore, the following remarks should be made:

- "No changes" are in Table 2.1-3 "on the diagonal".
- Some changes (e.g. sand in water and bare soil in water) have been indicated with the same colour. The reason for doing this was to obtain a picture that could be interpreted relatively easily.

It is important to have insight into the accuracy of the full procedure used here to obtain changes in land and water distribution. Within the framework of the Jamuna Bridge Study these inaccuracies were analyzed and it was found that differences are anyhow significant when they are larger than about 200m. This figure was obtained after analyzing all errors involved. A value of 200m corresponds to 0.8mm on scale 1:250,000.

		Next image			
		Water	Sand	Bare soil	Vegetated soil
Previous image	Water	Blue	Light yellow	Very light grey	Pale green
	Sand	Pink	Yellow	Light grey	Light green
	Bare soil	Salmon rose	Brownish yellow	Grey	Green
	Vegetated soil	Red	Brown	Dark grey	Dark green

Table 2.1-3 Applied colouring code for multi-temporal images

2.1.3 Results

Primary results of the present remote sensing study are geometrically corrected, classified images of the Jamuna River for particular years. In total 30 of these images have been produced or are still in the process of being produced, 15 for the South and the North image respectively. For the status of the processing as per 1 July 1992 reference is made to Table 2.1-1. From these images "difference-images" can be prepared conform the method described in the preceding section. It is foreseen that in total 22 of these multi-temporal images will be prepared. About half of them have been produced as per 1 July 1992. In addition some special products were prepared, notably (1) an image where all channels were plotted over the period 1973-1987, giving all channels different colours and (2) an image where the probability of occurrence of a major channel in the period 1973-1987 is indicated.

In a further step the primary results, as laid down in the products described above, are being analyzed, conform what is indicated in Section 1. Results of this analysis are discussed in the following chapters.

2.2 BWDB CROSS-SECTIONS

2.2.1 Introduction

Apart from planform data, also cross-sectional characteristics of the Jamuna River were studied. This was done on the basis of soundings which have been carried out by the Bangladesh Water Development Board (BWDB) since 1966/67. The analysis carried out was mainly aiming at the identification of any consistent changes in channel location, and as such is an extension of the work by Masselink (1989) and Technical Report No. 2, on the state.

The present analysis expands on the work by Ullah (1987), who compared soundings of 1965/66 with more recent ones from 1983/84, and the work carried out under the Jamuna Bridge Project, where the complete series of soundings were analyzed. By including all available soundings over the whole period, a better insight is obtained into the fluctuations over the years in relation to any trends in bed levels and channel locations. In this analysis also the most recent years until 1990 are included.

2.2.2 Available Data

Cross-sectional soundings of the Jamuna River have been made since 1965/66. Initially these measurements were done every year, but they were interrupted in 1970 due to the independence War. In 1975/76 the soundings were taken up again, although not every year the Jamuna River was included in the countrywide morphological measurements. In the period between 1965/66 and 1989/90 for most of the cross-sections in total some 12 soundings are available for analysis.

The soundings were carried out during the low flow season. Both the submerged areas and the dry islands (locally called "chars") in between the braids were sounded. The bed levels are related to the PWD (Public Works Datum). The horizontal distance was measured from monuments. Sometimes the monuments got lost either by bank erosion or by other causes. Consequently for a number of cross-sections it is not possible to make a quantitative comparison of channel location in the horizontal direction.

Initially the distance between the cross-sections was 4 miles, but gradually more cross-sections have been introduced over the years. Fig. 2.2-1 provides an overview of the sounded cross-sections. In the present study twelve cross-sections were selected for detailed analysis. An example of some measured cross-sections is given in Fig. 2.2-2.

2.2.3 Processing

For the study of any cyclicity in the behaviour of the river system, a computer program was used that was developed for the study of cross-sections of the Jamuna River within the

framework of the Jamuna Bridge Project. This program uses the sounded data as input, and subsequently the following processing is being done per cross-section:

- for three discharge levels stages are plotted in the sounded cross-sections, and channels and sand bars (chars) are differentiated;
(i) 44,000 m³/s (bankfull stage); (ii) 12,000 m³/s (intermediate) (iii) 4,000 m³/s (average low flow)
- the total wet area for each of the three discharge levels is computed;
- going from the left (Eastern) bank to the right (Western) bank monument the cumulative conveyance is computed as a function of the distance to the left bank monument; in a second step this relationship is made

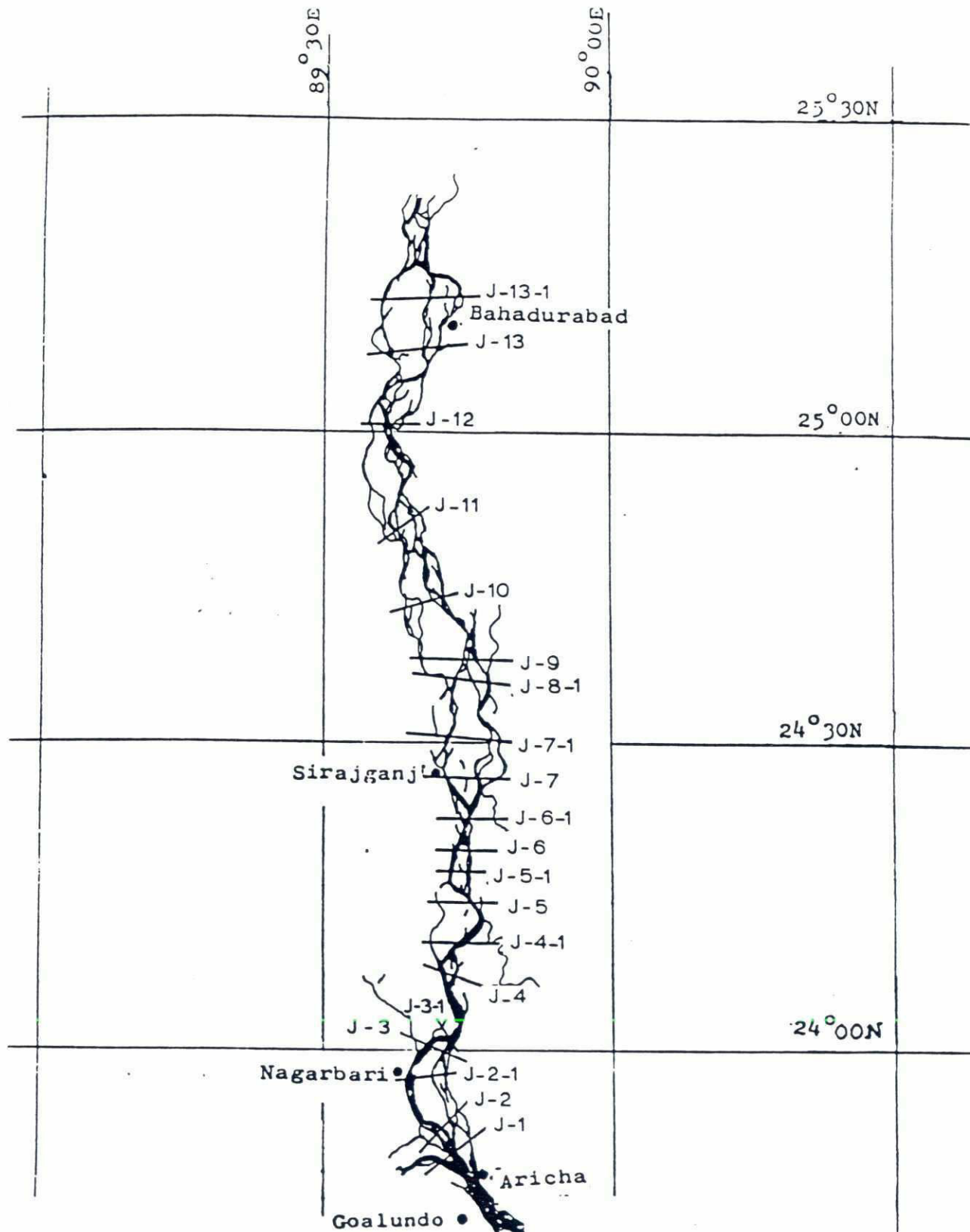


Fig. 2.2-1 Cross-sections measured by BWDB in the period 1966 to 1990

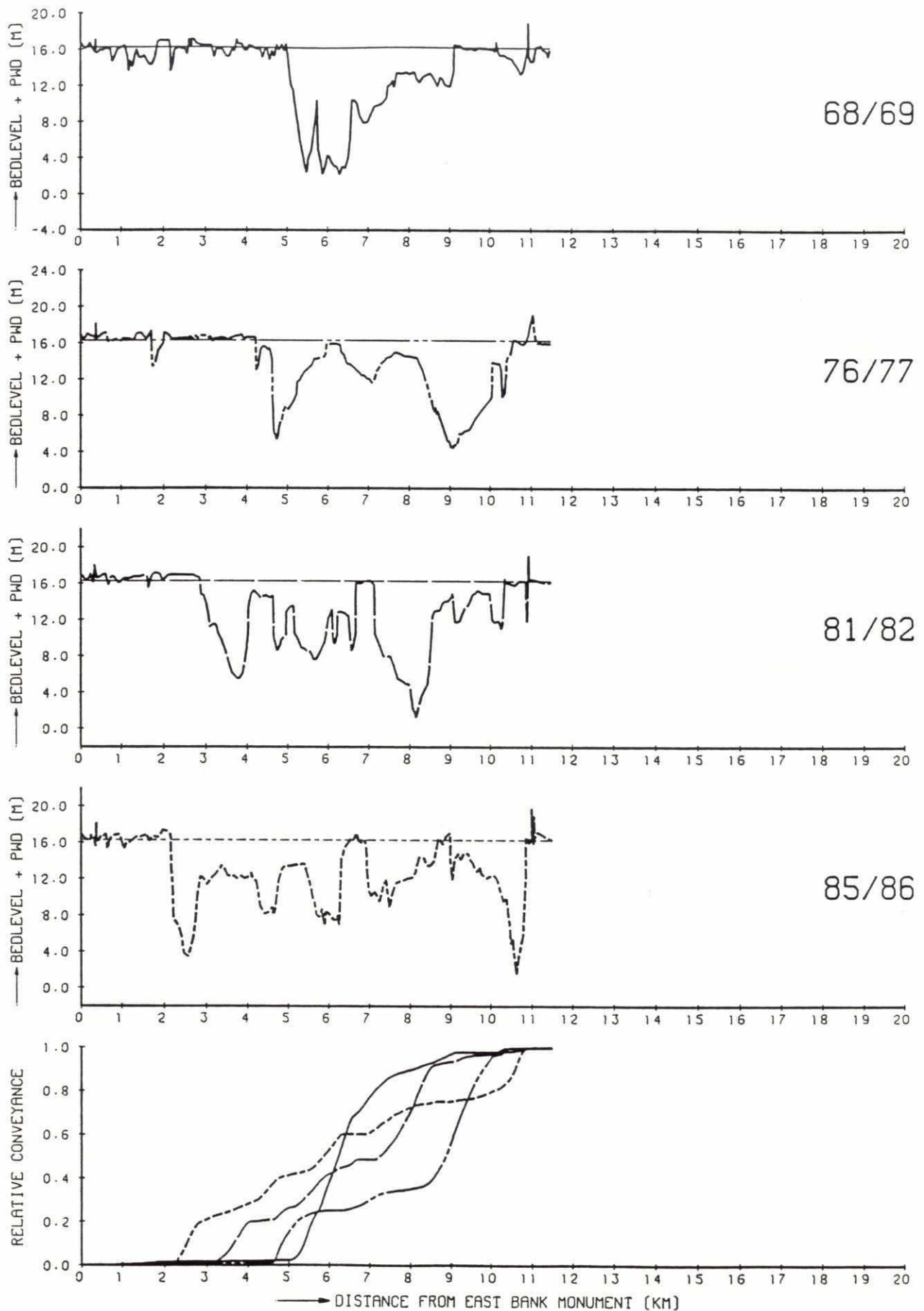


Fig. 2.2-2 Example of cross-sections measured in the Jamuna River

dimensionless by dividing it by the total conveyance for that particular cross-section and stage (for an example of the result see Fig. 2.2-2;

- the width and the average levels of the individual channels in each cross-section is determined, together with the average char level, the average channel level averaged over all channels, and finally the average bed level of the whole sounded cross-section, including both the parts above the water and the submerged parts.

2.2.4 Analysis

The analysis of the cross-sectional data is based on fitting formula (1) to the location of the 50% conveyance points in a cross section in different years :

$$Y = Y_{avg} + c_1 \cdot \sin[c_2 \cdot (T_2 - T_0) + c_2 \cdot (T_0 - T_1)] \quad (2.2.1)$$

in which

Y	distance of the 50 % conveyance point	(m)
Y_{avg}	the average distance of the 50% conveyance points with respect to a selected reference year	(m)
c_1	amplitude	(m)
c_2	$2 \pi/T$	(year-1)
T	period	(year)
T_0	reference year	(year)
T_1	start sinus function	(year)
T_2	year of the prediction of the 50% conveyances point	(year)

After selection of c_1 , c_3 and c_4 the value of c_2 has been determined by a minimalisation procedure. The minimalisation parameter for the iterative parameters fitting A is defined as:

$$A_i = \sum_{i=1}^N [Y_{i,observed} - Y_{i,calculated}]^2 \quad (2.2.2)$$

The average migration, Y_{avg} , with respect to the migration in the selected reference year was first estimated as the average of the given data points. For a good curve fitting it was necessary to multiply Y_{avg} with coefficient c_4 , if the given data points did not cover one complete cycle with a minimum and a maximum value of the 50% conveyance point. And consequently also the value of the average migration had to be adjusted. This coefficient varied between 0.6 and 2.4, but in all cross sections minus the cross sections with these extreme values between a narrow range of 1.0 to 1.4.

Next the amplitude Ampl was estimated as the difference between the maximum migration minus the minimum migration in the points, which are given in the graphs for different years. For an optimal curve fitting it was necessary to multiply Ampl with a coefficient c_5 . This coefficient varied between 0.75 and 1.1.

First this curve fitting procedure has been applied to each cross section separately. From these data an average period was estimated at 22.8 years. Next this procedure has been repeated with a constant period for all cross sections, see example in Fig. 2.2-3.

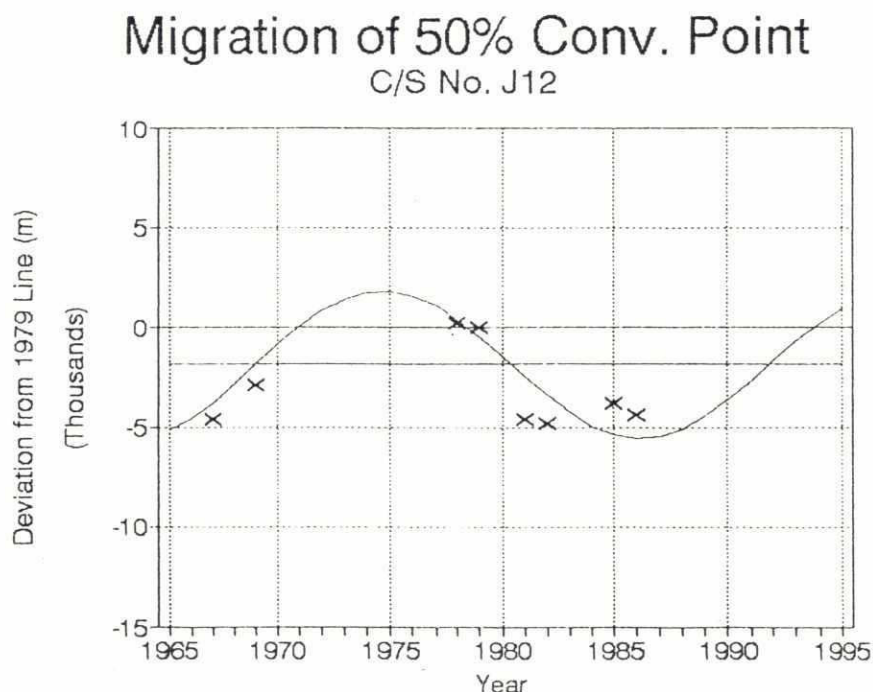


Fig. 2.2-3 Example of cyclic behaviour

The amplitude of the cyclic behaviour is not constant in different cross sections : the minimum amplitude is about 1000 m and the maximum almost 6000 m. In cross sections J6 and in J11 the amplitude is so small that the cyclic behaviour becomes obscured by the scatter in the data points.

If an average period of 22.8 years is assumed, then next the phase of the sine function in the subsequent cross sections is evaluated. By adding and subtracting of a whole cycle in some cross sections an increasing or decreasing sequence of phase values in subsequent cross sections is obtained. This means that the available data is not sufficient to determine if this assumed cyclic behaviour propagates in upstream or downstream direction, see Fig. 2.2-4. The determined downstream propagating cyclic behaviour has the following tentative characteristics: a period about 22.8 years, a wave length of about 80 km and a downstream propagation velocity of 3.5 km per year.

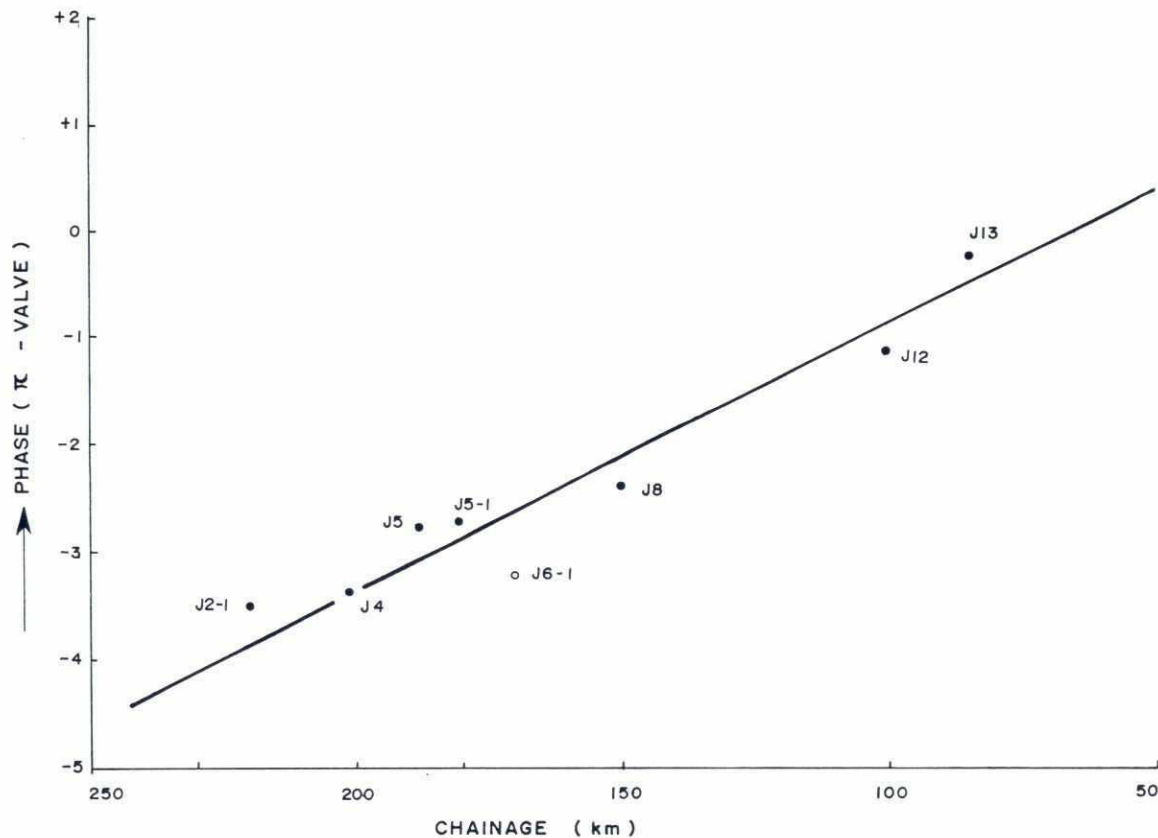


Fig. 2.2-4 Phase as a function of the chainage

The general conclusion is that not sufficient data of cross sections of the Jamuna river is available for this very special analysis of the assumed cyclic behaviour of the 50 % conveyance point. For this analysis the distance between the cross sections should be smaller than the distance of the available cross sections. Therefore it can not be proved with this analysis that this cyclic behaviour really exists. If the results of this analysis are combined with the results of the analysis of the satellite images, then the conclusion is that for the time being a cyclic behaviour of the conveyance points in the cross sections cannot be proven.

	66/67	67/68	68/69	69/70	76/77
77/78	78/79	80/81	81/82	84/85	85/86
J2-1	n.a.	19/2	5/5	29/11	4/4
27/12	8/12	10/1	13/1	n.a.	1/4
J4	12/3	n.a.	23/2	7/1	9/6
24/1	28/12	16/3	27/1	2/1	19/4
J5	30/1	16/2	27/1	23/1	10/11
17/4	7/1	11/3	5/2	7/1	1/5
J5-1	n.a.	17/1	13/1	30/1	21/11
11/4	13/1	24/2	9/2	n.a.	7/5
J6	14/2	24/12	10/1	3/3	26/11
18/1	4/2	12/2	12/1	13/5	3/4
J6-1	n.a.	8/12	24/11	18/1	10/12
28-3	24/1	15/1	5/12	-	23/11/86
J8	14/2	1/2	13/5	4/6	6/1
10/3	18/2	16/11	11/1	26/1	19/5
J11	n.a.	16/12	11/4	3/5	25/2
31/1	6/4	8/3	7/2	10/2	22/4
J12	25/12	n.a.	16/3	13/3	n.a.
13/1	16/4	2/4	7/12	15/2	4/5
J13-1	12/12	26/2	4/2	24/1	9/4
1/5	n.a.	1/2	n.a.	16/5	1/1
J16	24/10	12/11	22/11	10/11	20/10
27/11	27/5	15/2	26/1	3/7	25/7

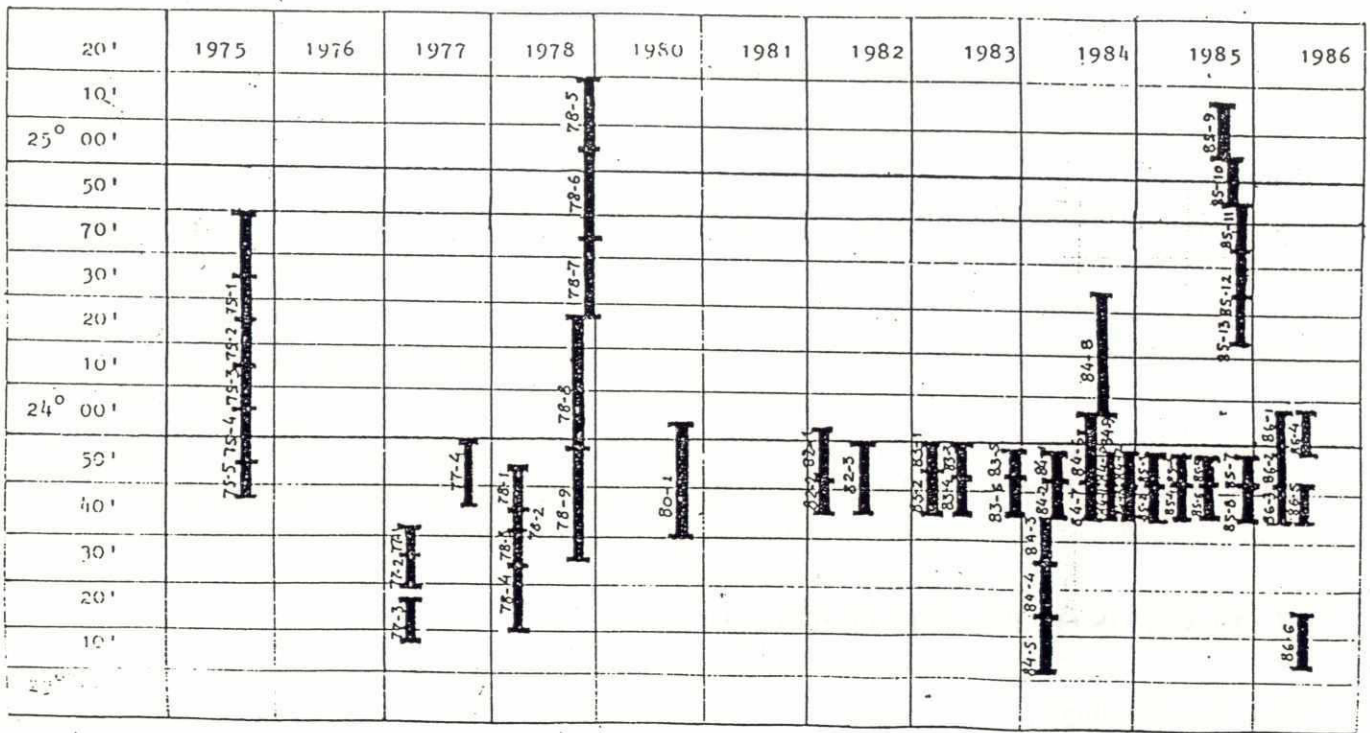
Table 2.2-1 Jamuna River BWDB cross-sections analyzed

2.3 BIWTA SOUNDINGS

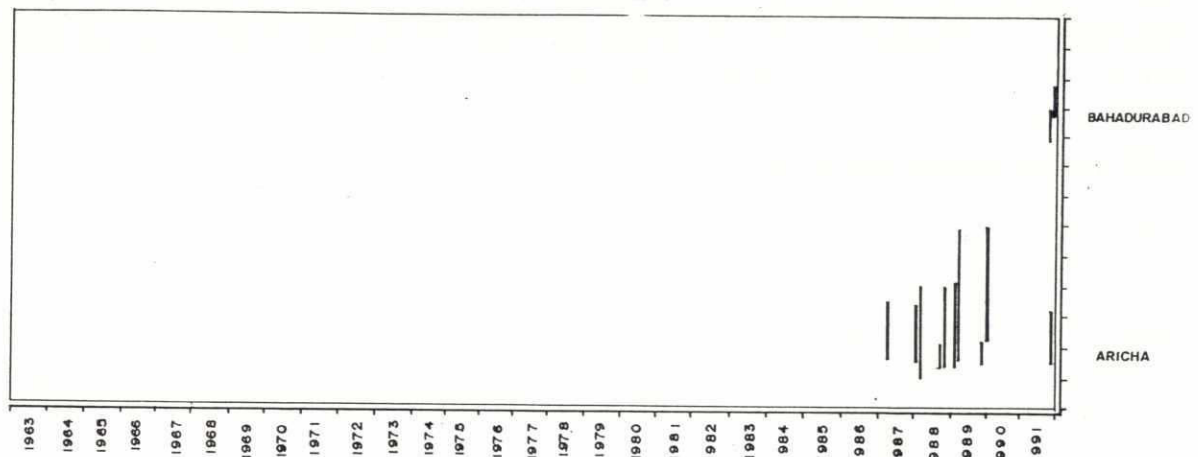
In addition to the BWDB soundings of the BIWTA were used. These soundings provide a more complete coverage of the deeper channels of the Jamuna River, but their frequency is much lower. See Fig. 2.3-1 where an overview is given of the available BIWTA soundings. The surveys are carried out to produce navigational charts. The surveys are made with echo-sounding equipment and a Decca main chain positioning system. Survey charts are usually plotted on scale 1:25,000 or 1:50,000 and lanes are plotted at 250m distances. Only three full coverage of the Jamuna River are available and two more partial ones. Especially the soundings in the years 1978 and 1985 are very useful for the satellite study of the river processes.

2.4 OTHER DATA

Within the frame work of the present study also a number of other data were used either explicitly or implicitly. These data can be found in the various reports that have been published over the last years. Data on bed material, sediment transport and resistance to flow were be found in the various reports of the Jamuna Bridge Project, Phase 1 and Phase 2. Data on bank material and also on sediment transport were found in the various reports published as part of the Brahmaputra River Training Studies.



JAMUNA RIVER



PADMA RIVER

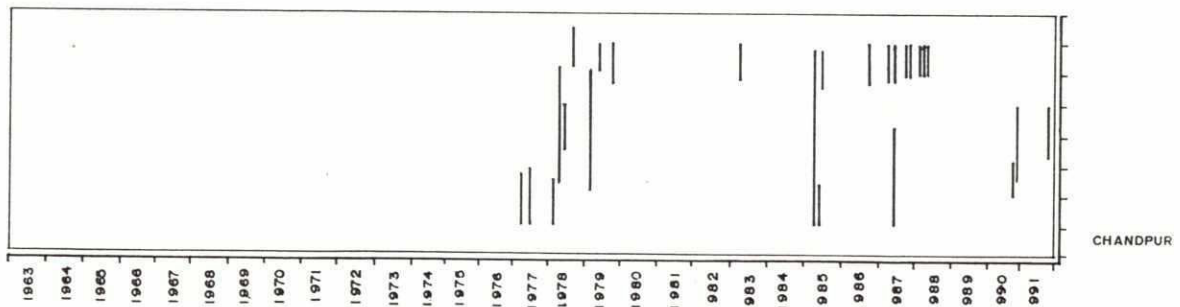


Fig. 2.3-1 BIWTA soundings available for the present study

3 PLANFORM CHANGES

3.1 GENERAL

In this Chapter some results are presented of an analysis of the available data on the planform and on planform changes of the Jamuna River, in particular of the satellite images. The results presented here are only preliminary, because the analysis is not yet finalized. For the time being the emphasis is on bank erosion rates and bifurcations. Other phenomena like the rapid narrowing or widening that can be observed in the Jamuna River have to be studied later. To develop a descriptive model of these changes in planform, ultimately all processes that are active in the Jamuna River should be identified, investigated and understood. The results presented here are followed by some remarks relating to the development of a predictive model of the processes in a braided sand-bed river like the Jamuna River.

3.2 ANALYSIS OF BANK EROSION RATES

3.2.1 Methodology

The aim of the analysis of bank erosion rates is to develop a relation between bank erosion and relevant channel characteristics, that can be used for predictional purposes. The analysis is actually a continuation of the work carried out within the framework of the Jamuna Bridge Project and later research carried out at Delft Hydraulics (see Masselink, 1989 and Klaassen & Masselink, 1992). These studies have yielded a relation between the relative erosion rate E/W and the relative radius of curvature:

$$\frac{E}{W} = f\left(\frac{R}{W}\right) \quad (3.2-1)$$

Where E = yearly erosion (m), W = channel width (m), and R = radius of curvature (m). The relation proposed by Klaassen & Masselink (1992) is inclosed here as Fig. 3.2-1. It can be observed that in this figure the scatter is substantial.

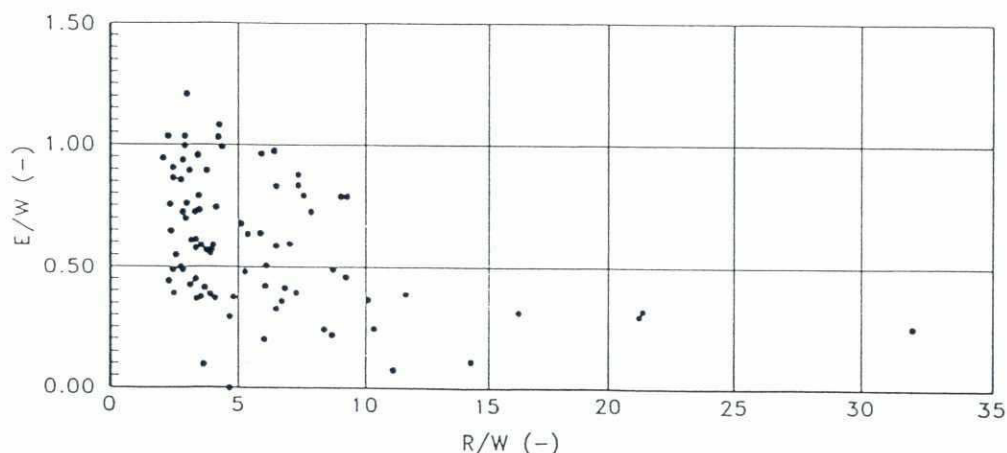


Fig. 3.2-1 Relative bank erosion rate versus relative radius of curvature (Klaassen & Masselink, 1992).

In the present study it will be attempted to reduce the scatter by including more parameter that may be of importance, notably the depth of the channel and the upstream curvature of the channel. The rationale for including the water depth is that it is felt that there may be a difference in behaviour of recent channels and channels that have been present for quite some time. Recent channels seem to be characterized by small width/depth ratio's, because they are still in the process of adapting to the local conditions. Inclusion of the effects of the upstream radii of curvature is a way to take into account the retarded adaptation of the flow in bends.

3.2.2 Selection of Data

Considering the above, two conditions had to be fulfilled to include change-detection images in the present analysis:

- (1) change-detection images of two consecutive years should be available (see below), to enable the determination of the bank erosion rate;
- (2) soundings should be available of the same reach, to allow for the determination of the depth of the channel.

During the time of the analysis only two change-detection images were available that fulfilled both conditions, notably 1977/78S and 1986/87S. Note that these images were also used by Masselink (1989). Regarding the availability of sounding data still the following can be stated: These sounding data were taken from the cross-sections. Quite often the location of these cross-sections did not correspond with the exact bend location. When a considerable difference existed between the width in the cross-section and the width at the bend locations (as measured from the satellite image) a correction was made.

The width was corrected in such a way that the real flow area was kept constant. In some cases however this procedure could not be applied because of the different planform.

From these years bends were selected, using the following criteria:

- Only migrating bends, no shifting bends.
- No bifurcations or confluences in the vicinity of the bend, these were supposed to have an important influence on the planform changes and were studied later on in more detail.
- No widening or narrowing on both sides of the channel. The fact that the same process (erosion or sedimentation) acted on both sides of the channel was assumed to be caused by a large change in discharge. The bends were therefore no longer considered comparable.
- Minimum width should be 0.5mm on the satellite image prints, 125m in reality. This is the smallest distance one can measure with reasonable accuracy.
- Depth data should be available.

Only sixteen bends met all the above mentioned requirements.

3.2.3 Analysis

From the selected 16 bends the width and radius of curvature were obtained from both consecutive single-year classified images, and the depth was obtained from the soundings. The erosion was derived from the change-detection image. These data were plotted in several graphs to investigate possible relationships. Results are presented in the Figs 3.2-2 through 3.2-4. These figures are discussed hereafter.

In Fig. 3.2-2 the relative erosion E/W is plotted versus the radius of curvature R , where the labels added to the data points refer to the width/ water depth (W/h) ratio of the channel considered. At first sight this figure looks very promising. Two trends are visible:

- a larger radius of curvature gives a lower relative erosion rate;
- width-depth ratio increases with higher radius of curvature and also with lower relative erosion.

The first trend observed is mainly caused by the width/radius relation, see below.

The other trend is dubious and might be considered spurious. The W/h trend in horizontal direction is again caused by the width/radius relation mentioned above. The separation in vertical direction is caused by the using of width in both ratios. A large width value will cause low relative erosion (E/W) and a large width/depth ratio (W/h). So the observed trend has a spurious relation. That only width causes the trend in the W/h labelling can be seen from Fig. 3.2-3. This is the same figure as Fig. 3.2-2, but labelled for depth instead for width/depth ratio. No trend can be observed in this graph.

In previous investigations e.g. Nanson & Hickin (1986) and Masselink (1989) the relation between relative erosion and relative curvature proved to be very important. For the selected data set of 16 bends a figure was prepared along the same lines. The results is presented here as Fig. 3.2-4.

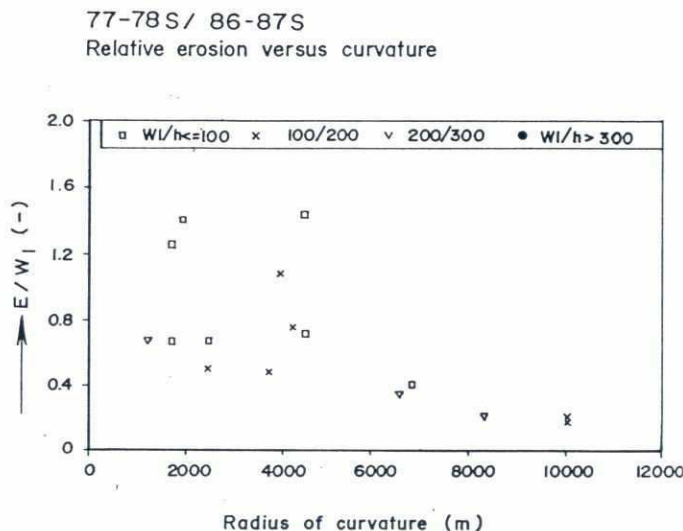


Fig. 3.2-2 Relative erosion versus curvature, labelled for width/depth ratio.



77-78 S / 86-87S

Relative erosion versus curvature

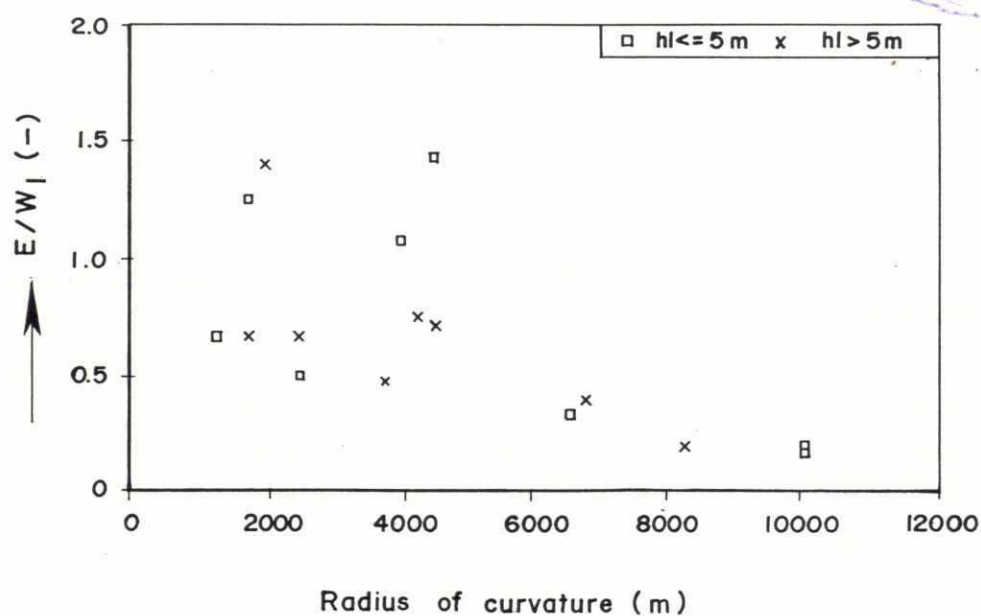


Fig. 3.2-3 Relative erosion versus curvature, labelled for depth

77S-78S/86-87S

RELATIVE EROSION VERSUS RELATIVE CURVATURE

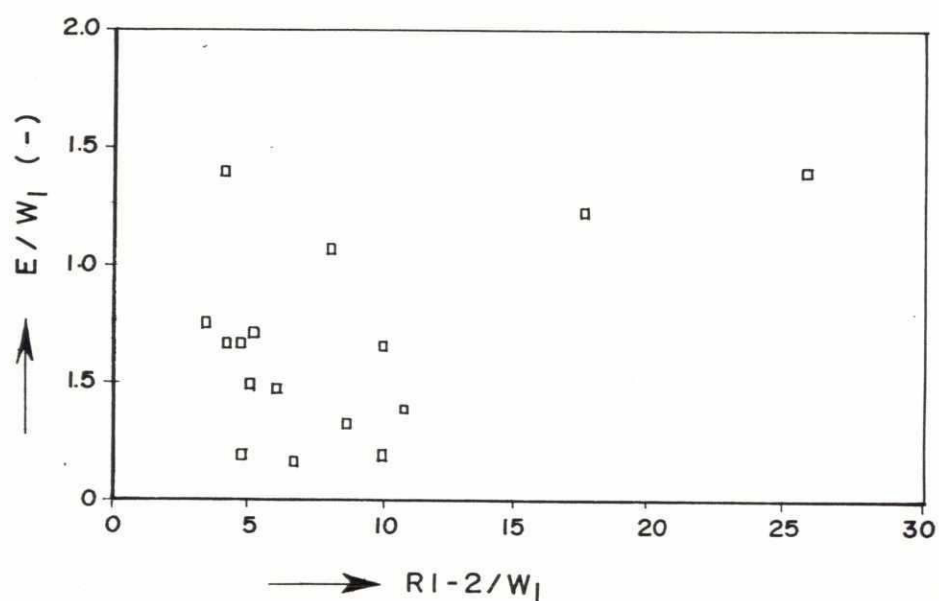


Fig. 3.2-4 Relative erosion versus relative curvature

Most striking is the decrease in relative erosion with an increase in relative curvature as reported in those studies. In the present analysis a similar relation was not found. The opposite trend seems even visible, increase in relative erosion with an increase in relative curvature.

Probable explanations for the discrepancy between the trend observed by Masselink (1989) and in this study are discussed below:

- The small number of locations used in this investigation. The only two points that visualize the opposite trend are taken from the same, relatively small and deep channels, so the significance can be doubted. Adding new data to the investigation could perhaps clarify this point.
- The trend observed here is the trend one might expect to find when using width in both ratios. Again the relation might be considered as spurious.
- Masselink used a two year average R/W. No average was taken when in one year R/W was greater than 5 and in the other it was smaller than 5. In that case the latter was selected. In this analysis the first year R/W was chosen, because that was the initial situation from which the erosion started. This seems to be a more logic choice: If one wants to make predictions, one is also forced to start from this situation.
- Subjective criteria in obtaining the parameters could also play a part.

3.2.4 Conclusions and Recommendations for Further Study

It seems that for the time being it is not possible to improve on the predictor developed by Masselink (1989), see also Klaassen & Masselink (1989), by including width/depth ratio of the channel. This conclusion has to be varified once more data are available from other years.

An aspect which was not studied until now was whether improvements into the way the radius of curvature (which is usually varying through the band) is included in the analysis can be improved. This will be done in the near future.

3.3 ANALYSIS OF BIFURCATIONS

3.3.1 Introduction

The main objective of this analysis is to gain insight in the apparently erratic process of channel abandonment. It was assumed that the bifurcation is an important factor in this process, because the water and sediment distribution determines whether a channel will scour or will die gradually (see Klaassen & van Zanten, 1989). Because the geometry of the bifurcation is very important for this distribution, this abandonment was studied giving due attention to the geometry of the bifurcation, as well as to some other channel properties, in

particular the cutoff ratio (for a definition see Klaassen & van Zenten, 1989). Abandonment in this analysis is the abandoning of one channel in favour of the other, as a result of which the bifurcation disappears. A shift of one (or two) of the bifurcation channels to a new course was not considered as an abandonment, because the bifurcation itself remained.

3.3.2 Selection of Bifurcations and Further Approach

Bifurcations were only studied when they met the following conditions:

- They should result in two channels. Bifurcations that result in three (or more) channels are fairly complex and are not included in this analysis.
- The inter-channels char should be of considerable size, more than 5 km² and preferably classified on the satellite images as bare soil with (some) vegetation.

Once a bifurcation was included in the analysis based on the above criteria, the main stream direction was determined by drawing the channel centre lines in the upstream channel. Then the tangent was constructed at the bifurcation point, and after this the angles of the two bifurcated channels with this line were obtained. Channels deviating to the right hand side of the main stream are defined to have a positive angle, hence the left hand sided deviating channels having a negative angle.

Besides the angles (α_1 , α_2) the following parameters were determined from the satellite image prints:

- | | |
|---------|--|
| L1, L2; | Length of the individual channels, from bifurcation to confluence, measured along the river. |
| D ; | Distance from bifurcation to confluence, measured along a straight line. |
| W1, W2; | Average width of individual channels, used to give an idea about the size of the different channels. |

The numbers indicate whether the left (1) or right (2) channel is concerned (looking in downstream direction).

The analysis to be discussed hereafter has the following limitations:

- The determination of both the exact bifurcation location and the main-stream direction is subject to a certain personal vision, the derived parameters and thus no real objective parameters.
- The Jamuna is a very dynamic river, a lot of processes are acting at the same time. Channels can for example also be abandoned by bypassing, this being caused by a change in channel configuration (e.g. a channel shift) upstream, leaving the bifurcation without water. This process is not accounted for in the present study.

- The width is the only measure for the size of a channel. Better measures (e.g. the conveyance) would also need depth data. These data however are not sufficiently available.

3.3.3 Analysis

In total 14 bifurcations, dating from the period 1976-1991, were studied. Only two of these bifurcations were located on the northern image. This can be explained from the fact that the northern part of the Jamuna River is more dynamic. This results in more complex bifurcations that do not meet the requirements for selection. Some of the bifurcations existed for 13 years whilst others disappeared within 2 years.

After entering the data in a spreadsheet, several graphs were made to facilitate the analysis. Some of these graphs are presented here as the Fig.s 3.3-1 and 3.3-2. Fig. 3.3-1 shows the sinuosity of a bifurcated channel, plotted against its angle. The sinuosity is defined here as the channel length (L) divided by the distance (D). In case the bifurcated channel was abandoned the following year, a special notation was used.

Fig. 3.3-2 shows the ratio of relative length of a bifurcated channel against its angle. Relative length is defined here as length of the channel divided by the length of the other channel. So $L1/L2$ is plotted against $\alpha1$ and $L2/L1$ is plotted against $\alpha2$. Again the abandoned channels were given a certain label.

3.2.4 Conclusion and Recommendations for Further Study

From an inspection of these two figures the following tentative conclusion can be drawn:

- The angle (α) indicating channel deviation from the main stream direction, seems to be a more important factor in channel abandonment than sinuosity or relative length.
- Abandonment seems only to take place when the angle (α) is larger than approximately 40° .

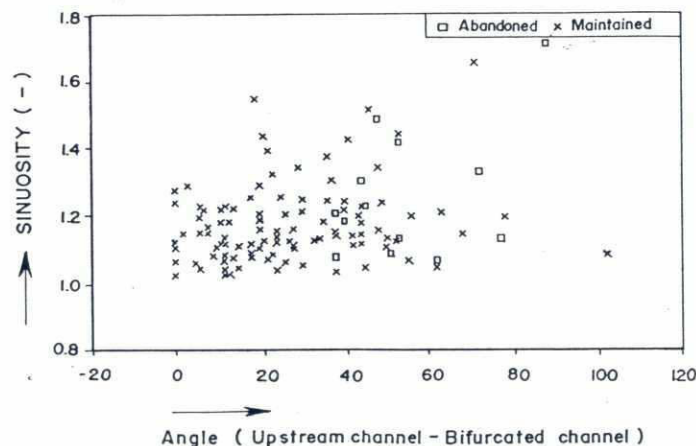


Fig. 3.3-1 Abandonment as a function of the sinuosity and the angle of the downstream channel

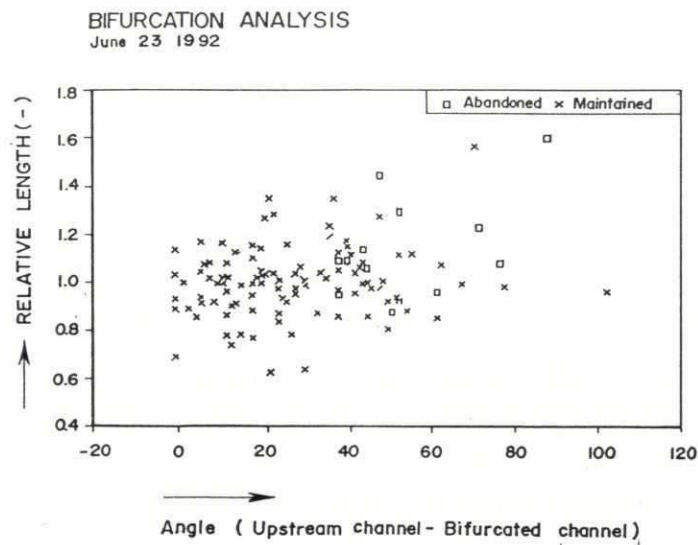


Fig. 3.3-2 Abandonment as a function of the angle of the downstream channel and the relative length.

In addition the following remarks are made:

- (1) The relative length corresponds to the cutoff ratio as used previously by Klaassen & van Zanten in their analysis of the cutoff phenomena. It seems that in Jamuna type of rivers the cutoff ratio plays only a minor role.
- (2) The angle of the downstream channel of a bifurcation is an important factor for the distribution of the sediment distribution at the bifurcation. Hence this analysis seems to indicate that the sediment distribution has a larger effect than the cutoff ratio.
- (3) Changes in channel configuration greatly affect bifurcations. This troubled the analysis a great deal. Upstream changes may lead to a change in main stream direction or even abandonment. Downstream changes may disturb the parameters L & D. Both changes trouble the analysis and limit the validity of the above mentioned conclusions.

This exploring investigation will be extended in the near future with:

- New bifurcations, when more satellite data of subsequent years will become available.
- More attention to the behaviour in time, to see how long it takes for a certain channel to be abandoned.

3.4 OTHER ANALYSIS

In the above two phenomena were studied in some detail. To be able to do this a selection was made, setting aside too difficult conditions. In a further stage it will be required to investigate also these more difficult cases. The same holds for precesses that have not been

studied until now. One example of this is the narrowing and widening of channels. In the Jamuna River it is regularly observed that single channels gain importance, but the understanding into this phenomenon is virtually absent. Hence more studies are needed to be able in due time to predict with a fair accuracy the behaviour of a braided river with sand as bed material over the coming years.

3.5 TOWARDS A PREDICTIVE MODEL

The processes in the Jamuna River are so complex that they sometimes seem almost random. Therefore, it is important to reflect upon the predictability of these processes. The predictions are based on observations and on the availability of constituent elements for a predictive model. Both put limits to the predictability, as will be discussed below. Based on this discussion the prospects of the development of a predictive model are indicated.

The observations do not provide complete information, and measurements are usually only estimates of the true state. In particular, the following problems can be mentioned:

- The resolution (pixel size) of the satellite images is on the order of 50m.
- There are no data on the morphological changes during a flood. Only the net end result can be seen on the satellite images taken in the dry season.
- Cross-sections are not available for every channel.
- Even when a cross-section is available for a certain channel, it is not necessarily representative because of the strong non-uniformity of the channels.
- Inundated banks cannot be distinguished from eroded banks on the satellite images. Hence, higher stages produce spurious bank erosion on the images.

These sources of inaccuracy deteriorate the input of (the concepts of) the predictive model, and hence decrease the quality of the predictions.

The derivation of mathematical predictive model based on the underlying mechanisms involves many problems, due to the accumulation of residual effects in the integration over time and space, the possibility of chaotic behaviour, the interaction at larger scales with other dynamic systems, and the necessity of including exceptional events that can be disregarded in predictions for shorter time spans.

The formal integration to larger scales results in a mathematical model consisting of a set of coupled partial differential equations. It reduces an infinite number of degrees of freedom to a number for which solving the equations is feasible. The procedure is not a direct integration from a small to a large scale, but a series of integration steps to ever higher levels in a cascade of scales. Due to the non-linearity of the system, each step produces closure terms for which additional relationships must be induced from observations. Examples of such relationships are eddy viscosity models in the integration over turbulent fluctuations, and velocity profiles in the integration over depth or width. The choice of additional relationships

also determines whether the result of the integration is a bar-approach model or a channel-approach model. This stepwise integration procedure implies that information on processes on smaller scales is lost, and that descriptions on larger scales are added as premises for further deduction. The errors of the additional relationships are residual effects. They may cancel out on larger scales, but they may also accumulate and eventually become dominant, so that the model derived by formal integration cannot predict the actual phenomena. The main aim of the observations of phenomena on larger scales is therefore a careful determination of the additional relationships for closure.

The resulting mathematical model with only a few degrees of freedom can also have the property that small deviations accumulate and eventually lead to totally different predictions. Since the recognition of this chaotic behaviour of deterministic systems, the term 'chaos' has got a more restricted meaning in physics. It no longer refers to randomness in general, but to a special type of unpredictability, related to a sensitive dependence on initial conditions. Chaos in river morphology is still a hardly explored subject. There are indications of chaotic behaviour in models for the development of bed topography and meander bend migration (Mosselman, 1992), but whether these models really exhibit such behaviour is still to be investigated.

As the final mathematical predictive model has not been formulated yet, it is not possible to quantify the predictability formally. From experience the Consultant estimates that the limit for bank erosion predictions is about 2 or 3 years. Five years ahead seems unrealistic in general, though some local phenomena are more persistent, e.g. major confluences, major bends and channels aligning active tectonic faults.

On larger scales, the dynamic system of a river interacts with the dynamic systems of (1) climate, (2) geological activity and (3) vegetation. Human interference (4) can also be regarded as an interaction with another dynamic system. These interactions are discussed below:

(1) Climate:

Interactions with climate are not taken into account. Possible effects would be a change in the discharge hydrograph due to changes in precipitation distribution, and a change in boundary conditions at the confluence due to sea level rise.

(2) Geology:

Bangladesh lies in one of the most active tectonic zones of the world (Coleman, 1969). It is generally recognized that major avulsions are controlled by tectonic migration of the Brahmaputra which is controlled by major faults or fractures in the earth's crust. However, there are strong indications that also the channel patterns within the present river course are influenced by tectonic activity. This could be one of the reasons that the river seems to have preferred courses.

(3) **Vegetation:**

Vegetation affects the bank erosion of smaller streams, but the larger the river, the less the influence will be, Roots are only present in the upper part, and bank materials can be sheared away by the flow up to depths of 10m (Coleman, 1969). The insignificance of vegetation is supported by Masselink (1989), who did not find any correlation between bank erosion and the presence of vegetation.

(4) **Human interference:**

Human interference can produce significant changes of the river, but this does not generate an additional source of unpredictability. By building river training structures, the river system becomes less free and hence more predictable.

Predictions covering a long period are also made difficult by the possible occurrence of exceptional events. An earthquake could completely alter the present-day trends and may result in a completely new orientation of the course of the river (as suggested by Coleman (1969) for the major shift of the Brahmaputra River from its former Old Brahmaputra course to the present Jamuna course).

Still however the above does not include that a predictive model cannot be developed. The characteristics of such a model would however be more complicated than the deterministic models presently in use. It seems that in this case a predictive model would consist of a combination of deterministic elements and probabilistic approach. Its constituent elements are starting to emerge. Some of these elements are typically the graphs shown in the Section 3.2 and 3.3 of this Annex. Hence the probability of occurrence of a certain phenomena (like and abandonment to take place) is a function of some local parameters but can also be expressed as a certain probability distribution function the characteristics of which are linked to these local parameters.

The following conceptual model can be visualised: The present planform is determined, e.g. from satellite images. Then the probability of changes in the planform during the next flood are evaluated yielding possible planforms after one year with corresponding probabilities of occurrence. Then again these new planforms are evaluated and possible further developments during the next flood are indicated. This leads to a new set of possible planforms after 2 years, again with related probabilities of occurrence. This procedure can be repeated for a number of floods. The final result is a set of possible developments to the river system with their probability of occurrence. These can then be used e.g. for a study of how probable bank erosion at certain site over the coming years will be. In fact the above approach was used in evaluating the conditions in the test areas in Chapter-4.

The conceptual model proposed above is described as a combination of a deterministic and a probabilistic approach. It is clear from the above description that initially (in the first years) the deterministic part is the most influential in determining how the planform of the river will change. After some years however the probability of occurrence is gradually taking over.

This implies that after some years (possibly some 5 years) the model will merely provide estimates of the state the river may be in, and becomes comparable to a statistical description of the river characteristics. In the first years however, the deterministic approach yields results that are steered from the existing planform characteristics and will provide useful results for planning preventive or remedial measures, e.g. in the case that a channel may become particularly aggressive over the coming years. Hence such a model will be very helpfull in deciding on any bank protection works (FAP 21) and any remedial measures to mitigate the attack of the river (as studied under FAP 22).

It is against this background of limited predictability and the gradual emergence of a conceptual model that may predict the behaviour of the river over the coming years, that the following discussion on the conditions at the test areas have to be appreciated.

4 STUDY OF TEST AREAS

4.1 TEST AREAS

In the Technical Report No.1 (Consulting Consortium FAP 21/22, 1992) the following areas for test river training structures were pre-selected:

- Kamarjani
- Bahadurabad
- Chandanbaisa
- Nakalia

The pre-selection was based on various criteria, one of them being the certainty-of-attack criterion. This criterion is evaluated here in more detail based on information (in particular satellite images) on the recent behaviour of the river system near the proposed test areas. This information became available over the last months as a result of the remote sensing study. In addition to this reassessment, the possibility of other sites is reconsidered on the basis of the new satellite images. The approach is outlined in Section 4.2. The actual analysis is described in Section 4.3 and the conclusions and recommendations are given in Section 4.4.

4.2 APPROACH

The evaluation of the certainty-of-attack criterion requires reliable predictions of future river geometry. These predictions are based on observations and on concepts for a predictive model.

Observations of the behaviour of the river is mainly inferred from satellite images and river cross-sections surveyed by the Morphology Division of the Bangladesh Water Development Board. These available observations are described in Chapter 2 of this Annex.

The predictive model aimed at is not available at this very moment, but some of its constituent elements are understood by now and computational rules are becoming available (see the Sections 3.2 and 3.3). These have been used instead in the present analysis. Ultimately the constituent elements of the model are based on fundamental laws describing water motion, sediment transport, bed erosion and sedimentation, and bank erosion and accretion. They are formulated for small volumes of water and sediment, and then integrated over time and space to larger scales. In their present form however they consist of fairly straight forward prediction graphs. Two types of approaches can be used: (1) bar approach and (2) channel approach.

In the bar approach, the pattern changes are taken to be determined primarily by changes of a submerged bed topography. What typically can be predicted using the bar approach is the gradual movement downstream of a system of bars and channels as present near Kazipur. Hence also the location of the main attack and how it changes in time can be predicted with this approach.

In the channel approach, the patterns are schematized as a network of migrating bends, interconnected at bifurcations and confluences that are also migrating. Within this network, new channels can be created and existing channels can be abandoned. Typically the channel approach can be used for the prediction of bank erosion along curved channels, the probability of the occurrence of cutoffs, etc.

The reality will be a combination of bar and channel processes, so that neither of the approaches separately will be complete. However, constructing a model which includes both approaches will be very difficult.

In addition to the small-scale motion of water and sediment, two elements are to be kept in mind: the hypothesis of cyclicity and the hypothesis of preferred river courses, both suggested by Masselink (1989) and Klaassen and Masselink (1992). The cyclicity of the 50% conveyance position is presently being investigated within the present framework (see Chapter 2) and a period of about 20 to 25 years seems to be found. The preference for certain river courses to which the river regularly returns might exist because the sediment deposited in former abandoned channels is eroded more easily than the surrounding sediments, like Gilvear and Harrison (1991) find for the river Tay in Scotland. It could also be related to active tectonic faults, which is currently being investigated by in cooperation with the Municipal University of Amsterdam.

4.3 ANALYSIS

4.3.1 General

In this Section the conditions at the proposed test areas are assessed in more detail. The emphasis is on the recent developments and attempts are made to extrapolate this recent behaviour to predict the most probable behaviour of the river system at the proposed test areas over the years to come. In particular the probability of continued bank erosion is evaluated and estimates of this bank erosion are made for the near future. This is presented in one figure for each test area in which some possible developments are indicated together with the probability that a certain development may take place.

The probability of the future developments and of continued bank erosion at the test areas is presented here to underscore the uncertainties involved in the prediction of the future

developments. By including predictions of the behaviour of the system during the flood of 1992, it will be possible after the flood to check on the accuracy of the predictions. It may be expected that the analysis of the satellite images as presently being carried out will result in improved possibilities for predicting the future behaviour of the river system. This will lead to better predictions and hence to guidelines when and where protective measures will be needed.

4.3.2 Kamarjani

Kamarjani and the Manos Regulator are situated on the right bank at an approximate latitude of 25°22'.

Cross-sections of this area are coded J-15. From 1973 to 1978, the river pattern here became more and more ragged, but the banks near Kamarjani and the Manos Regulator were not under attack. The river had regained a simpler appearance in 1984 with a bifurcation at about 4 km downstream of the Manos Regulator. Upstream of this bifurcation, widening and associated sedimentation produced an area of dissected chars. Further sedimentation caused these chars to coalesce and to integrate with the island downstream in 1986, thus shifting the bifurcation 12 km upstream. Island growth by lateral accretion deflected the bifurcated channels outward. This induced severe bank erosion, for which the right embankment had to be retired. This is the background of the massive bank erosion at Kamarjani and the Manos Regulator after 1984. However, the corresponding migration rate has decreased now, and there seems to be a tendency that the eastern anabranch could become more important than the one near Kamarjani. Hence it is likely that the erosion may diminish in the years to come.

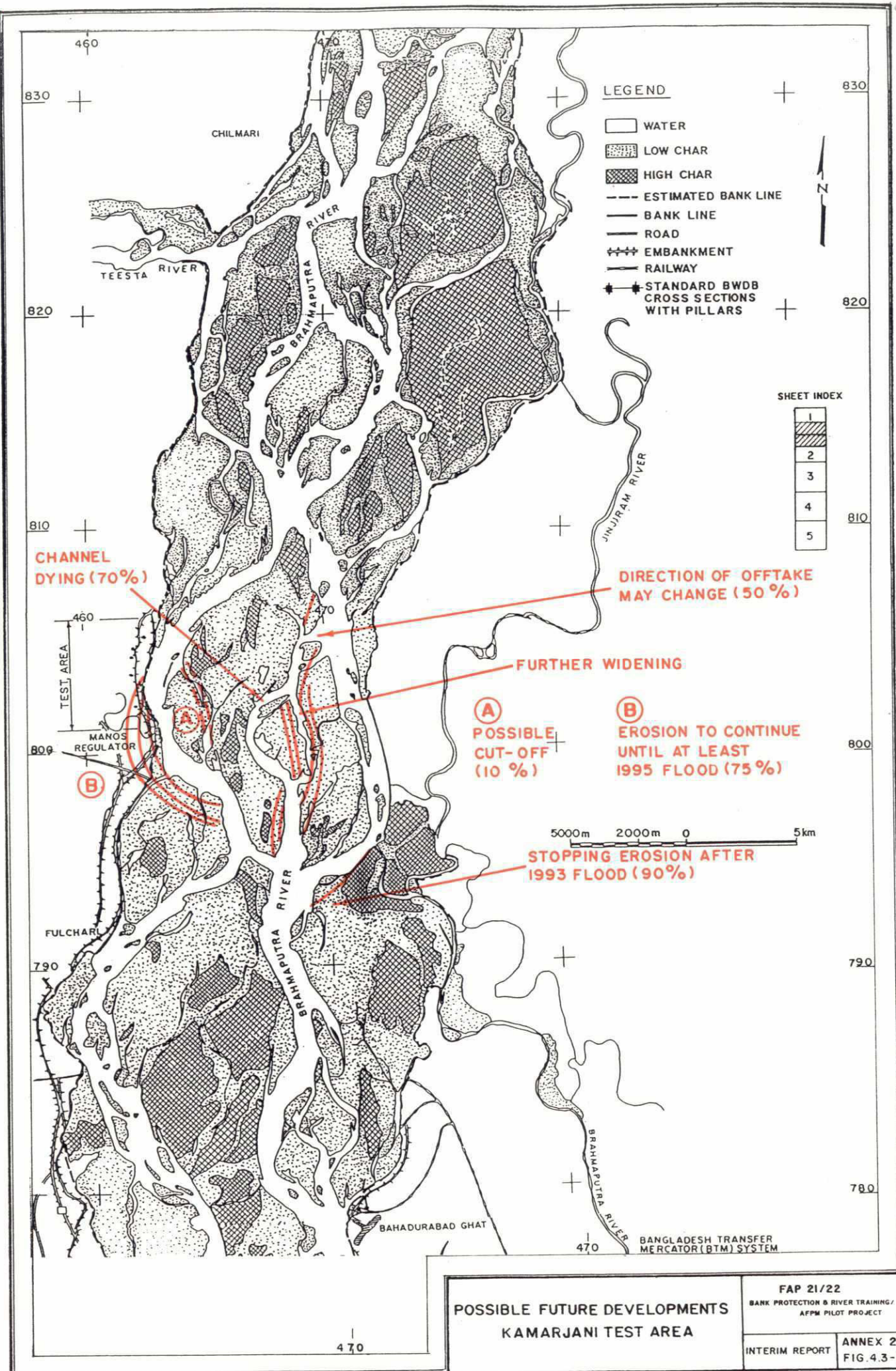
A cyclic repetition of channel patterns at Kamarjani could not be detected from the images. Rather, the impression of overall widening was dominant. It is interesting to note that in 1991 the island had been split in two by a nodelike crossing lying at the same location as the bifurcation of 1984, which supports the hypothesis that the river has preferred courses.

The processes prevailing from 1977 to 1984 can be understood best from the perspective of the bar approach, but for the behaviour from 1986 to now, and hence for predictions, the channel approach is more appropriate.

Based on the 1992 planform derived from the 1992 image and on the changes over the period 1989-1992 (see also Fig. 2.2-1 of the main report) an assessment was made of possible future developments at the Kamarjani test area. The results of that assessment are presented in Fig. 4.3-1.

The following observations are made:

- (1) In 1992 a mid char channel had developed that acts as a cutoff channel for the Eastern channel near Kamarjani. If this channel develops further the possibility exists



**POSSIBLE FUTURE DEVELOPMENTS
KAMARJANI TEST AREA**

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

INTERIM REPORT **ANNEX 2**
FIG.4.3-1

that this channel is going to attract more water at the expense of the Western channel. This may especially be the case if the minor channel West of the cutoff channel would be abandoned. Larger channels usually have smaller slopes, hence they tend to attract more water in a braided system.

- (2) The cutoff channel has only a good probability to remain in existence if the direction of the offtake changes slightly. It is expected (at a 90% probability level) that the cutoff channel will continue to widen over the coming years. The curved pattern may induce the formation of a bend opposite to Kamarjani in due time.
- (3) Near Kamarjani two possible cutoffs can be identified. The probability that they will develop into a real cutoff is estimated to be very low because of the angle of the offtakes from the main stream direction.

Hence it is concluded that with a 75% probability the bank erosion near Kamarjani will continue at least until 1995. The rate of bank erosion at the Kamarjani site for the period 1992-1995 is also indicated in Fig. 4.3-1. It is felt that this rate will hardly decelerate. As a consequence it may be expected that the Manos regulator will be eroded away before a test structure can be built. Developments after 1995 are increasingly difficult to predict, but it is estimated that the bank erosion will still continue, although probably at a slightly lesser rate.

4.3.3 Bahadurabad

Bahadurabad lies on the left bank at latitude $25^{\circ}09'$. Cross-sections of this area are coded J-14. In 1973, there was only one main channel here. It had a strong meandering twist at about 10 to 15 km upstream of Bahadurabad Ghat, outflanking into the left bank of the Jamuna. The river became more braided in the following years. Severe bank erosion had occurred upstream of Bahadurabad in 1976 as a combination of channel migration and widening associated with char formation. This bank erosion produced an embayment just upstream of Bahadurabad Ghat. In 1984 a more orderly planform had appeared, mainly consisting of two anabranches on either side of an elongated island. These anabranches persisted from then to 1992. Since 1990, an important channel has been impinging on the bank South of Bahadurabad Ghat, whereas the embayment which was formed between 1973 and 1976 silted up. The change-detection images of 1989-1991 and 1991-1992 show a continuation of this trend which can be expected to persist during the next two years. The bank downstream of Bahadurabad Ghat seems therefore a most suitable area for bank protection test works.

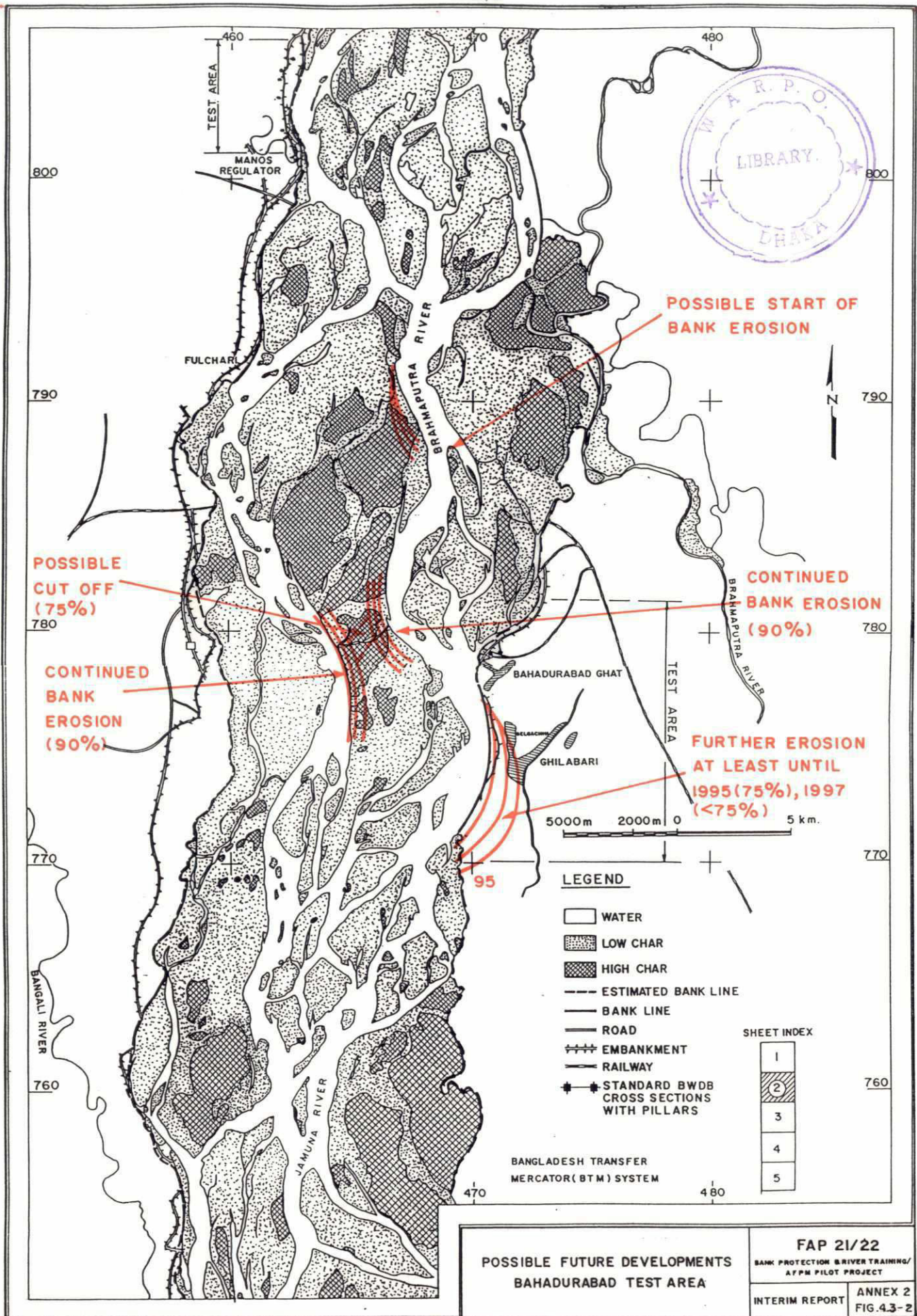
A cyclic repetition of channel patterns at Bahadurabad could not be detected from the images. Rather, the impression of widening prevailed. A return to a course which had been abandoned previously could not be recognized in the immediate vicinity of Bahadurabad. The channel approach offers the best base for predictions here.

Based on the 1992 planform derived from the 1992 image and changes over the period

1989/1991-1992 (see also Fig. 2.2-2 of the main report) an assessment was made of possible future developments at the Bahadurabad test area. The results of the assessment are presented in Fig. 4.3-2.

The following observations are made:

- (1) Upstream of Bahadurabad two channels are present that have developed fair bends. One channel is located at in the Eastern channel system and is in fact the main reason for the erosion downstream of Bahadurabad. The other channel is more connected to a system of mid char channels and bars.



- (2) It is expected that both channels will continue to erode, hence in time increasing the probability of a short-cut to develop, see Fig. 4.3-1. It is estimated that such a short cut will have taken place in 1995 with a 75% percentage of probability.
- (3) In the past channels have existed at the possible future location of this short cut and all of them carried water from the Eastern reaches of the river to the Western reaches.
- (4) If such a development would occur it would gradually reduce the rate of erosion south of Bahadurabad.

The probability of continuing bank erosion at Bahadurabad is estimated at 75% in 1995 and less than that for the years to come.

4.3.4 Chandanbaisa

Chandanbaisa is situated on the right bank at latitude $24^{\circ}48'$. Its cross-section is coded J-11. Throughout the years, two anabranches can be identified here. In 1973, both anabranches lay far from Chandanbaisa, but in 1976 a bend had already migrated to the vicinity of this town. The two anabranches had considerably shifted Westward in 1984. As a result, a relatively wide and braided anabranch aligned the right bank near Chandanbaisa from 1984 to 1991. Bars propagating through this anabranch exposed and protected the bank at Chandanbaisa alternately, thus offering favourable conditions for building test river training structures. In 1991, however, substantial sedimentation had occurred along the whole stretch of the bank. Char fragments grown together downstream of Chandanbaisa deflect the main flow thread Eastward. Upstream of Chandanbaisa, a channel which dissects the medial char seems to gain importance. If this trend continues, then the main body of the flow will impinge on the bank upstream of Chandanbaisa, thus causing significant bank erosion. This trend cannot be counted upon too surely, though, so that the certainty-of-attack criterion is not satisfied very well here.

A cyclic repetition of channel patterns at Chandanbaisa could not be detected from the images. Again, the impression of widening and increased braiding prevailed. A comparison of the Southern images from 1973 and 1989 offers an impressive illustration of this trend. A return to a preferred course could not be found in the immediate vicinity of Chandanbaisa.

The sedimentation and char growth in this area can be understood best from the perspective of the bar approach, but the channel approach is the best base for a prediction of bank erosion upstream of Chandanbaisa.

It should be noted that char erosion might even occur in a period in between the dates of two consecutive images which show char accretion. It is possible that scour occurs during the flood, after which substantial deposition takes place during the falling stage of the hydrograph.

Based on the 1992 planform derived from the 1992 image and changes over the period 1991-1992 (see also Fig. 2.2-3 of the main report) an assessment was made of possible future developments at the Chandanbaisa test area. The results of the assessment are presented in Fig. 4.3-3

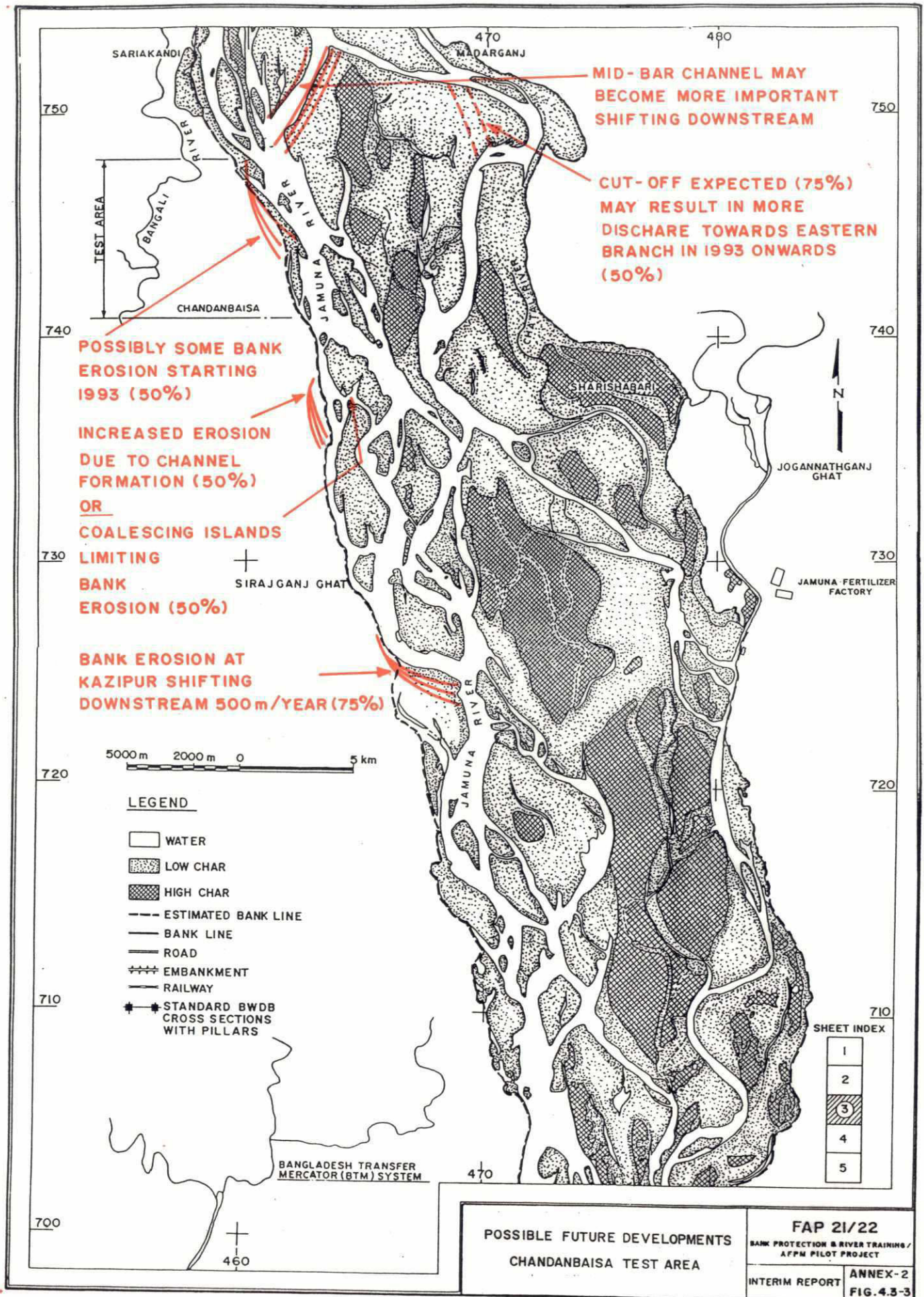
The following observations are made:

- (1) At present most of the discharge is passing along the Western side of the river. The bend near Madarganj on the left side will probably be cut off in 1992 or 1993. The possibility exists (50%) that this will induce the flow to shift gradually from the Western bank to the Eastern bank.
- (2) A mid-bar channel upstream of Chandanbaisa seems to gain importance. Because it is directed towards the Western bank this channel may induce bank erosion in the test area.
- (3) More downstream there will probably be no bank erosion owing to the coalescing islands formation mentioned before.

The possibility of bank erosion starting in 1993 is estimated to be about 50%.

4.3.5 Nakalia

Nakalia lies on the right bank at latitude $24^{\circ}01'$. Cross-sections of this area are coded J-3. Downstream of Sirajganj, the Jamuna River is less braided and tends to assume a meandering planform. Despite the presence of chars and secondary channels, the main flow thread can easily be identified. In 1973, the river bifurcated near Nakalia around a char of less than 20 km². This char grew later into a more stable, vegetated island, with two anabranches on either side of it. Further growth shifted the bifurcation point about 15 km upstream into the vicinity of Betil. From 1973 to 1984 the Western anabranch was dominant. Severe bank erosion near Nakalia created an embayment in this period. In 1991 and 1992, however, the Eastern anabranch became more important, and the embayment formed previously was filled with sediment. The 1989-1991 and 1991-1992 change detection images show that the erosion is less than about 300m per year. Meanwhile, the Eastern channel between Chauhali and Aricha became much more active, and it is suggested to consider a test area along the left bank as an alternative. When doing so, one should anticipate the cutoff of the large bend which is likely to occur within two years. This alternative meets the preference expressed in the Terms of Reference for test sites on the left bank. Another option is that the test area is maintained on the right bank near Nakalia, with an additional guiding structure on the left bank to deflect the main flow thread to the West. This additional structure could be upstream of the present bifurcation in the area of Betil, or (better) further downstream, forcing channel creation through the island. For the latter option, the meandering planform of 1978 can serve as a guideline. It has a regular shape complying with the natural characteristics of the river.



A cyclic repetition of channel patterns at Nakalia could not be detected from the images. One might expect otherwise because the river is meandering here. It should be noted, however, that the wave length decreased. In 1978 and 1984 there were only five half-meanders between Sirajganj and the confluence with the Ganges, whereas there are six half-meanders in 1992. The decrease of wave length can be explained from a decrease of discharge, because in 1992 part of the discharge is conveyed by the western anabranch.

Again, significant widening occurred. The widening is such that the river between Sirajganj and the confluence with the Ganges might even lose the meandering appearance and become a fully braided river. A comparison of the southern images of 1973 and 1989 strongly suggests this possibility. A return to a course which had been abandoned previously could not be detected. The most appropriate base for predictions here is the channel approach.

Based on the 1992 planform derived from the 1992 image and changes over the period 1991-1992 (see also Fig. 2.2-4 of the main report) an assessment was made of possible future developments at the Nakalia test area. The results of the assessment are presented in Fig. 4.3-4.

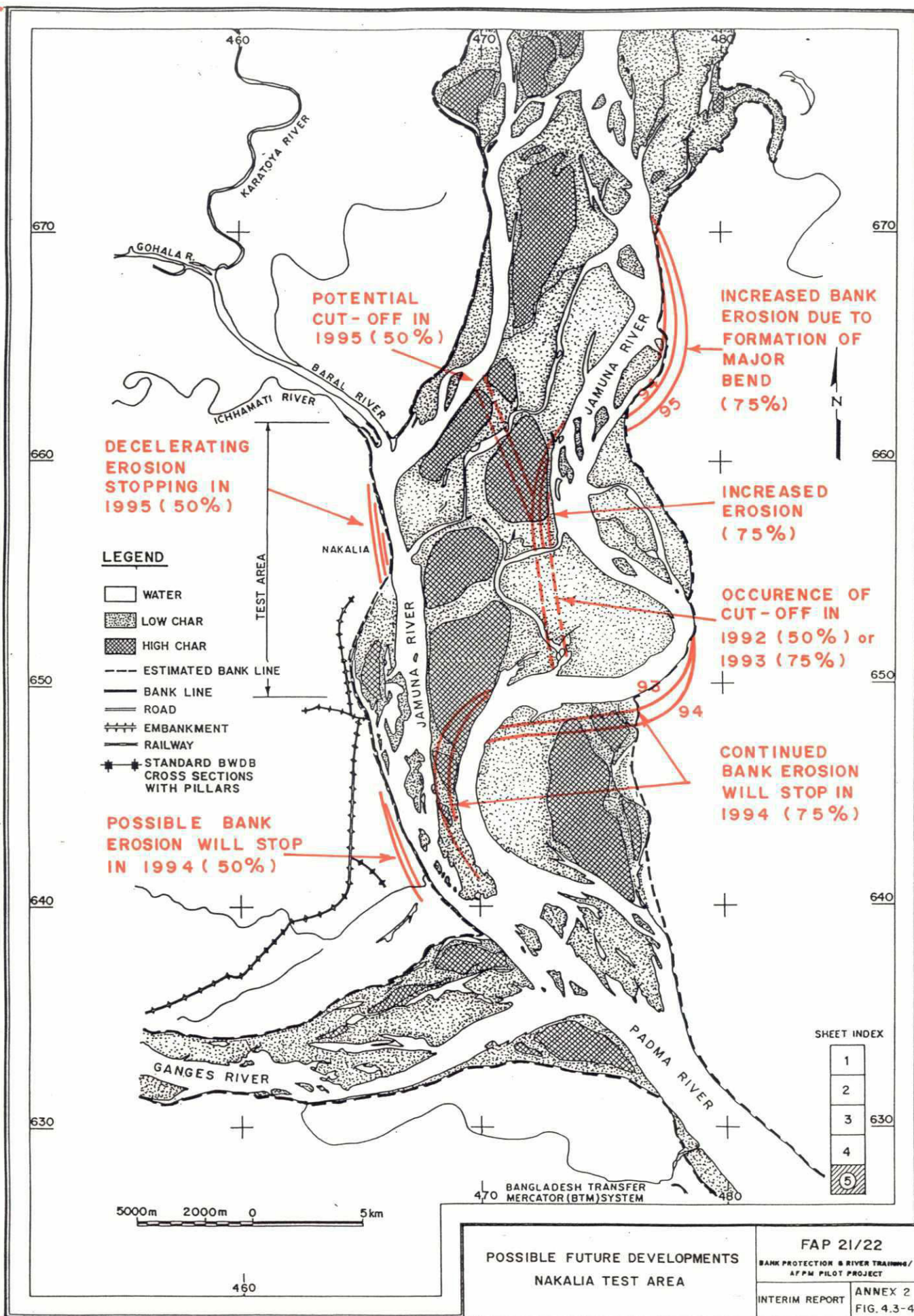
The following observations are made:

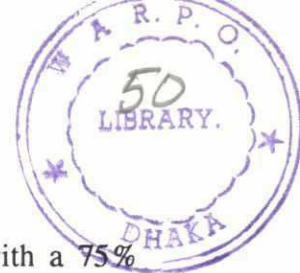
- (1) At present, a large bend exists opposite and slightly downstream that will experience a cutoff shortly. The probability that this cutoff will occur in 1992 is about 50% and that it will have materialized in 1993 is 75%.
- (2) Upstream of this bend there is another bend which is supposed to develop over the coming years.
- (3) This will trigger accelerated bank erosion just upstream of the cutoff discussed under (1). The effect of this bank erosion will be that the probability of a cutoff upstream will increase, see Fig. 4.3-4. This may lead to a reduction of the discharge in the Western channel and hence a reduction of the bank erosion rate at Nakalia.
- (4) An other development may also influence the relative importance of the Western channel. The Eastern channel may gain importance due to the cutoff foreseen and the related movement of the bend in upstream direction. This may cause this channel to draw more water and this may result in a decrease of the bank erosion near Nakalia.

Based on the above observations, it is expected that the bank erosion at Nakalia will continue to reduce and may stop (with a 50% probability) in 1994.

4.3.6 Other possible areas

It is clear from the previous subsection that the certainty-of-attack criterion is nowhere fully fulfilled at the four pre-selected test areas. Hence the Consultant has studied the recent developments in the river system to identify other areas which might have a larger probability of being attacked from 1994 onwards. The following areas have been reviewed :





(1) **Right bank near Kazipur**

At present there is an important migrating bend in this area. It is expected with a 75% probability that this bend will continue to erode near Kazipur.

(2) **Left bank opposite to Kamarjani**

The Eastern anabranch is becoming more important here, conform Section 2.1, and attacks the left bank. However, present estimates are that this bank erosion will reduce or possibly even stop after the 1993 flood.

Left bank at Jamuna Bridge

Severe erosion occurs along the left bank at the proposed site of the Jamuna Bridge. Considering the river planform, this bank erosion can be expected to continue, unless a cutoff along the Western bank would develop.

Left bank between Chauhali and Aricha :

This alternative has been discussed in subsection 4.3.5. It seems worthwhile to keep this possible location in mind and to verify the developments at that location after the 1992 flood.

4.4 CONCLUSION AND RECOMMENDATIONS

At **Kamarjani** and the Manos Regulator, the severe bank erosion after 1984 was related to the formation of a large island which deflected its anabranches outward. The corresponding migration rate may decrease in future, but substantial bank erosion will continue (at a 75% probability level) until at least 1995.

The change-detection images of 1989-1991 and 1991-1992 show a continuation of bank erosion South of **Bahadurabad Ghat** due to the impingement of a channel from upstream. In view of the present geometry of the area, this can be expected to continue during the next two years. The bank downstream of Bahadurabad Ghat seems therefore a suitable area for bank protection test works. Future developments may cause that the bank erosion rate reduces after 1995.

Substantial sedimentation is occurring along the bank near **Chandanbaisa**. Chars downstream deflect the main flow thread to the East. It is plausible that the bank upstream of Chandanbaisa will erode in the near future, but the possibility for this does not satisfy the certainty-of-attack criterion very well.

Bank erosion at **Nakalia** appears to slow down. However, the Eastern anabranch along the opposite bank is much more active, and it is suggested to consider a test area there as an alternative. When doing so, one should anticipate the cut-off of the large bend which is likely to occur within two years. The situation after the 1992 flood will be checked.

Neither cyclicity nor path preference could be seen in the channel patterns at the pre-selected sites. The overall widening is a much more striking phenomenon. This widening might have far reaching consequences for the reach between Sirajganj and the confluence with the Ganges, because the river might lose its meandering appearance and become fully braided here. A comparison of the Southern images of 1973 and 1989 strongly suggests this possibility. An alternative explanation is that the river downstream of Sirajganj changes alternatively between more meandering and more braiding. More studies are needed to elucidate this aspect.

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ANNEX 3

PRESENT STATE OF SCOUR STUDY

ANNEX 3

SCOUR STUDY

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1 INTRODUCTION

This Annex provides a brief description of the scour study carried out to arrive at better predictions of the scour near groynes in outer bends. This study is tackling one of the questions in relation to the prediction of combined scour (see Chapter 2). In this Annex a brief review is given of the facility used, the results obtained and the recommendation arrived at regarding the prediction of scour near groynes in outer bends. More details will be provided in a separate Technical Report that is to appear in October 1992.

2 ON COMBINED SCOUR

Until recently maximum scour depths near river training works in Jamuna type of rivers were usually determined using methods essentially based on the regime approach as proposed by e.g. Ahmad (1953). In these methods only limited use is made of the specific understanding of river processes. In fact when the design methods proposed by Ahmad and others were developed the understanding of phenomena in particular braided rivers like the Jamuna River was only limited.

Since then there has been quite some progress in the understanding of river processes. One dimensional mathematical models have been developed to simulate **general scour**, defined in e.g. RPT/NEDECO/BCL (1990b) as the reaction of the river to changes in its boundary conditions (like deforestation, diversions, sea level rise, etc.). The understanding of **scour in bends** has substantially improved, not only for axisymmetric conditions but also for the conditions in the beginning of a bend, including the so-called overshoot and point bar formation (see Struiksma et al, 1985). Much more insight is obtained in the influence of bank erosion products on the lateral bed profile (see e.g. RPT/NEDECO/BCL (1990b) and Mosselman (1992)). Expressions for **constriction scour** have been developed, including their time-wise development via numerical models or via analytical solutions (see Klaassen & Struiksma, 1988). **Confluence scour** has been identified (Ashmore & Parker, 1983), and special predictors have been developed for Jamuna type of rivers (Klaassen & Vermeer, 1988a). Insight in bedform dimensions in large rivers like the Jamuna River and the related minimum **bedform scour depth** was obtained and their relevance for the design of river training works was indicated (Klaassen & Vermeer, 1988b). Finally a number of physical model studies have been carried out allowing to derive expressions for **local scour** (Pilarczyk, 1984; Breusers & Raudkivi, 1991).

It is challenging to use this improved understanding to arrive at a better prediction of design scour depths for river training works along Jamuna type of rivers. This is especially important when a probabilistic design approach is used, because for all contributions to the total scour depth separate probability density functions can be derived, which can or cannot be correlated.

Indeed, recently some projects have been carried out which are trying to use this increased understanding to arrive at more scientifically based predictions of the scour depths that may occur near river training works that are being planned along the Jamuna and similar types of river. It is interesting to summarize here the approaches taken within these different projects. Especially because it appears that being able to differentiate between the different types of scour and to be able to compute their contribution to the total scour depth is a necessary condition, which is however, not sufficient for establishing the design criteria for river training works.

If the different contributions are estimated separately, then the major issue becomes how to combine them. There is at present no univocal opinion of how to do this and in fact this is the very topic of the present study for the combination of bend scour and local scour around groynes.

Within the framework of the present study a review was made of the approaches taken in three different projects which were carried out in Bangladesh recently and which can be used to illustrate that there is indeed at this moment not one established method to combine these different types of scour. The three projects referred to here are:

- (1) Jamuna Bridge Project (RPT/NEDECO/BCL, 1990a)
- (2) Brahmaputra River Training Studies (Halcrow/DHI/EPC/DIG, 1991)
- (3) Meghna River Bank Protection Short Term Study (Haskoning/DELFT HYDRAULICS/BETS, 1992).

During the review it was observed that different approaches have been used for the different sites and that there is no univocal opinion on the importance of some of the contributions to the total scour. Also the way some of these various contributions were taken into account was slightly different.

The review has led to the following questions to as how to determine extreme scour depths in Jamuna type of rivers and how to combine the different contributions to the total scour:

- (1) Should evaluation of the bend scour be based on observed bend profiles or should the influence of the erosion of bank erosion products be taken into account?
- (2) Is local scour in front of revetments negligible (as assumed in the Brahmaputra Study) or does it have a significant contribution to the total scour (as assumed in the Meghna study) ?
- (3) Is protrusion scour indeed an important phenomena worthwhile to be identified and assessed independently (as done in the Meghna study) in cases where the bank protection works are stand alone structures like in Chandpur ?
- (4) Is bedform scour important and should it be taken into account (as done in the Brahmaputra study) or is its influence negligible near structures where local scour is dominant (as assumed in the Jamuna Bridge and the Meghna studies) ?

- (5) Should local scour (and protrusion scour) be computed including the effect of the constriction scour and the bend scour on the initial water depth (Meghna study) or should it be evaluated separately and simply be added to the combined scour excluding local scour (Jamuna Bridge study) ?

The present study addresses specifically the fifth question and to a smaller extent the first question, leaving the other questions open for further studies.

It should be mentioned here that the prediction methods employed during the different projects are all based on an extensive analysis of field data, during which the combined scour that had been observed in the past was split up in the different contributions to scour. It is therefore felt that any conceptions as to how to combine these contributions will have a minor effect on the predictions made for those particular locations. Application of the approach taken and the methods developed to other locations should be done with care, however, until answers can be given to the questions listed above.

3 APPROACH

The main purpose of this study is to investigate the combined scour near groynes along the outer bank of a bend in alluvial channels and determine how to combine bend scour and local scour, particularly in the case of bank erosion products having a significant influence on the shape of cross-sections in the unprotected situation. The study has to result in recommendation on how to assess the combined scour due to bend flow and groynes. If h_o is the water depth of a straight channel without groynes, h_b is the maximum depth of scour in outer bend and h_l is the maximum scour depth at the head of groyne in a straight channel, then, with the construction of a groyne along the outer bank of a bend, the total depth of scour h_{lb} can probably be expressed as:

$$h_{lb} = f(h_o, h_l, h_b) \quad (3.1)$$

The exact formulation of this relation is not known (see Chapter 1). Experiments in a physical model with mobile bed and banks will help in clarifying this relationship and thus in obtaining a better prediction for the maximum scour to be expected around groynes in bends.

The tests were carried out in an experimental facility that was constructed in the De Voorst experimental station of DELFT HYDRAULICS. The characteristics of the facility are as follows:

- (1) Overall dimensions 9 m long and 6 m wide.
- (2) The height of the walls is 0.60 m.
- (3) A tailgate with the width of 3.0 m is installed at the lower side of facility to set the downstream water level.

- (4) The facility is filled up with sand material having a particle size of $D_{50} = 0.210$ mm.
- (5) A closed system for sediment and water recirculation is used including a sand-trap, a water-storage, a sand pump with $Q = 7.75$ l/s, a clean water pump with a maximum discharge of 50 l/s and two separate pipe lines, see Fig. 3-1.
- (6) A series of pipes with a diameter 45 mm and a length of 0.50 m is used to dampen the turbulence of the flow downstream or where the flow is entering into the facility.
- (7) The groynes were simulated with thin plates. The advantages of this schematization is (i) that they could be handled easily and (ii) the local scour is independent of the scour depth.

The following measuring instruments were used:

- (1) one electro-magnetic flow meter to measure the flow discharge;
- (2) two pilot tubes for measuring the water levels;
- (3) one normal point gauge for measuring the bed levels;
- (4) one electro-magnetic flow velocity meter.

In addition vertical photographs were taken from a point about 7.5 m above the facility.

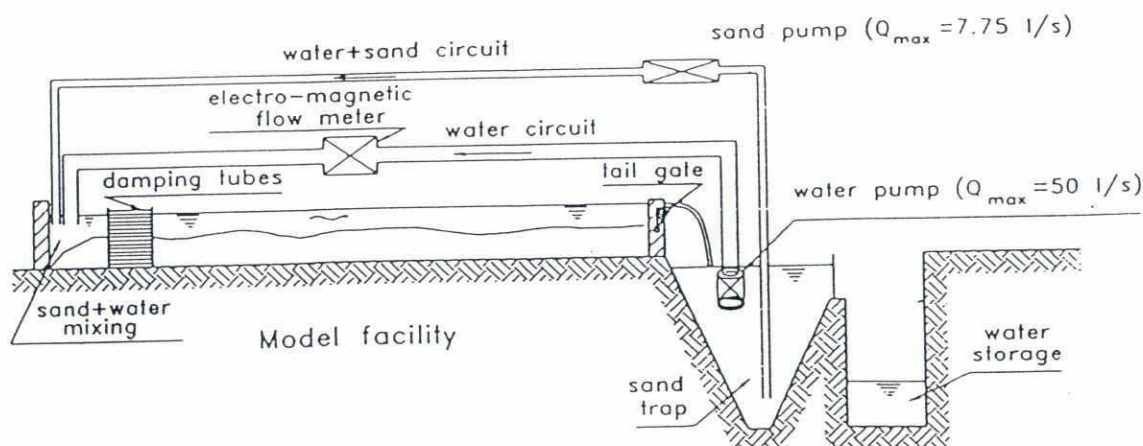


Fig. 3-1 Experimental facility used and water and sediment circuits

In total three series of each five experiments were carried out. The five experiments are described hereafter as five subsequent stages of the investigation (see also Fig. 3-2):

- Stage 1 : Running the model with a straight channel without any groynes. From this test the average water depth h_0 for given discharge Q , width B and slope i is obtained. During this experiment both banks were fixed.
- Stage 2 : Running the model with a straight channel including groynes along one bank with the same Q , B and i . In this stage the aim is to determine the maximum

depth of local scour h_l near the head of groynes and the increase in depth Δh_l ($= h_l - h_o$) due to local scour. In this experiment only the bank without groynes was fixed.

Stage 3 : Running the model with a curved channel, without any groyne and with the same Q , B and i . The aim of stage 3 is to determine the maximum depth of outer bend scour h_b and the increase Δh_b ($= h_l - h_o$) near the groynes. In this experiment both banks were fixed again.

Stage 4 : Running the model with a curved channel and groynes along the concave bank for the same conditions as the previous experiments to find the maximum depth of scour in front of groynes in the bend h_{lb} and the increase due to the combined presence of an outer bend and local scour Δh_{lb} ($= h_{lb} - h_o$). In this experiment only the inner bend was fixed.

Stage 5 : In the last stage (only during the first two series of experiments) the same curved channel with erodible outer bank as in stage 4 was used. The groynes were placed in a retired position some 0.3 m from the out bank. The initial conditions were the same bed topography during equilibrium conditions of the third stage. The purpose of this stage is to study the effects of bank erosion products on the bed topography of bends.

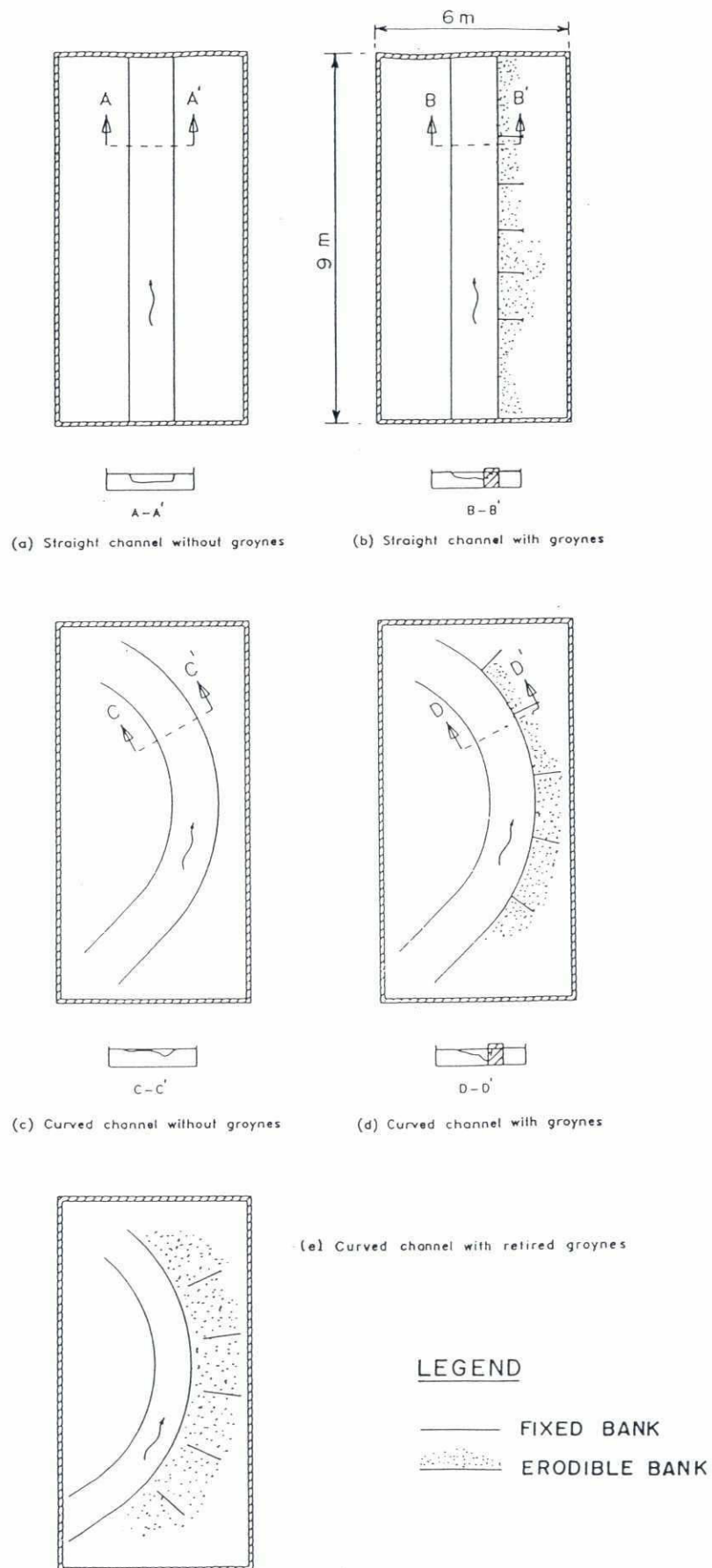


Fig. 3.2 Various stages of test

4 RESULTS

The main results are summarized in the Table 4-1. Furthermore some salient details are provided in the Fig. 4-1 through 4-3, which relate to the first series of experiments only. Similar results are available for the other series.

Test No.	Test Code	Run. time (hr)	i(f) (-)	h(ave)	h(max) in the sections :					v(ave) (m/s)	Fr(ave) (-)	O(ave) (-)	C(ave) (m.5/s)	q(s)*10 ⁶ (m ² /s)
					3	4	5	6	7					
1	SCWG1	73	.0025	.08	.077	.077	.084	.079	.084	.347	.392	.58	23.34	5
2	SCIG1.2	46.5	.00276	.083	.095	.164	.18	.181	.171	.307	.351	.661	24.18	-
3	CCWG1	57.5	.0024	.089	.118	.177	.179	.18	.099	.347	.371	.616	20.9	2.4
4	CCIG1	46	.00267	.11	-	.257	.27	.25	.244	.271	.268	.844	15.09	-
5	SCWG2	49.5	.00322	.064	.066	.066	.066	.066	.063	.544	.697	.599	33.74	7.65
6	SCIG2	95	.00376	.055	.125	.121	.119	.131	.156	.572	.809	.594	46.79	-
7	CCWG2	113	.00345	.076	.093	.163	.225	.136	.129	.496	.583	.793	27.81	23.24
8	CCIG2	48.25	.00367	.091	.141	.246	.305	.27	.268	.415	.454	.967	20.7	-
9	SCWG3	94	.0014	.096	.088	.091	.098	.102	.097	.359	.371	.386	28.43	2.9
10	SCIG3	115	.00236	.071	.1	.114	.127	.163	.177	.415	.514	.484	39.13	-
11	CCWG3	94	.00305	.094	.108	.158	.144	.199	.225	.44	.466	.827	21.37	22.12
12	CCIG3	23	.00463	.082	.115	.229	.253	.272	.258	.498	.6	1.09	24.06	-

Table 4-1 Main results of the test

The test code used in Table 4-1 is explained hereafter:

- SCWG = Straight Channel Without Groynes (Stage 1)
- SCIG = Straight Channel Including Groynes (Stage 2)
- CCWG = Curved Channel Without Groynes (Stage 3)
- CCIG = Curved Channel Including Groynes (Stage 4)
- CCWIG = Curved Channel Without and Including Groynes (Stage 5)

The following observations can be made when inspecting the results in the Table 4-1:

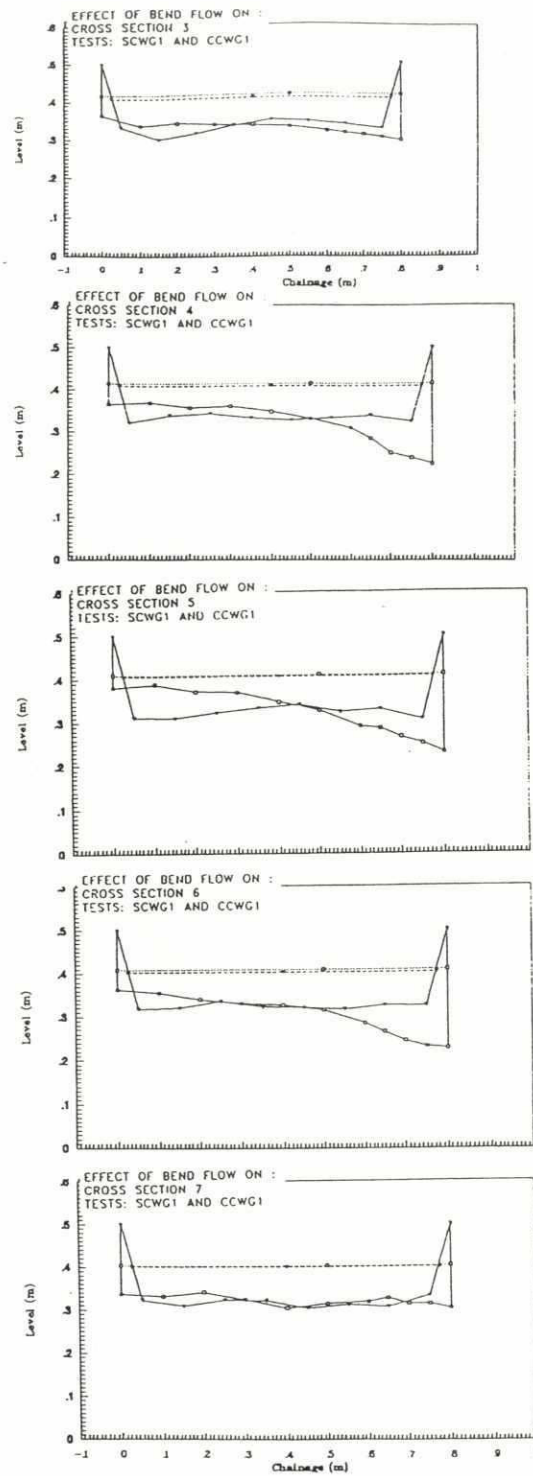
- (1) During the 1st and the 2nd series the slope was fairly constant. During the 3rd series the slope of the different experiments was quite different (varying from 0.0014 to 0.0046). Hence the different experiments are not comparable. For the time being the 3rd series has not been used in the analysis.
- (2) The Chezy coefficient varied between 15 and 28, with the exception of two experiments SCWG2 and SCIG2, which were done during a period when the water temperature in the facility was probably substantially higher than during the other experiments.
- (3) During the experiments the average Froude number varied between 0.35 and 0.58, except for the same experiments SCWG2 and SCIG2.

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Fig. 4-1 presents the results of experiment CCWG1 compared with the results of experiment SCWG1. Cross-section 3 corresponds to the beginning of the bend. It can be observed that a typical bend profile is present in the bend. In cross-section 7 the bed is almost horizontal. This is due to the occurrence of the so-called overshoot in bends and it will be explained in more detail in the forthcoming Technical Report.

Fig. 4-2 compares the experiments SCWG1 and SCIG1, and hence this figure shows the effect of the local scour near the groynes. It is remarked that the maximum local scour near the groynes occurred near in the cross-section where the groynes was due to the use of plates instead of real groynes.

Fig. 4-3 shows the results of the experiment CCWIG1 (Stage 5) compared with the experiment CCWG1. It is observed clearly that the bank erosion during this experiment had a significant effect on the cross-sectional profile.

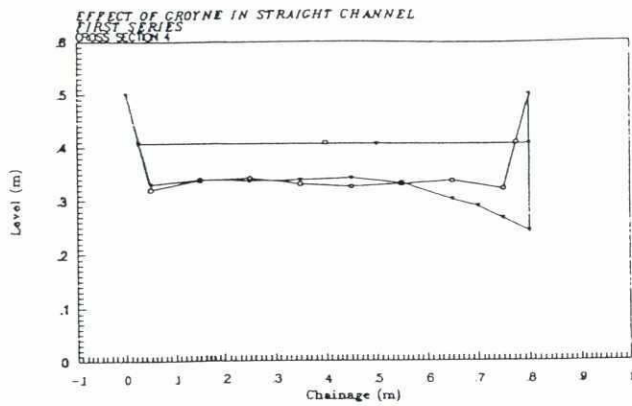


$Q = 0.030 \text{ m}^3/\text{s}$
 $B = 0.96 \text{ m}$
 $D(50) = 0.210 \text{ mm}$

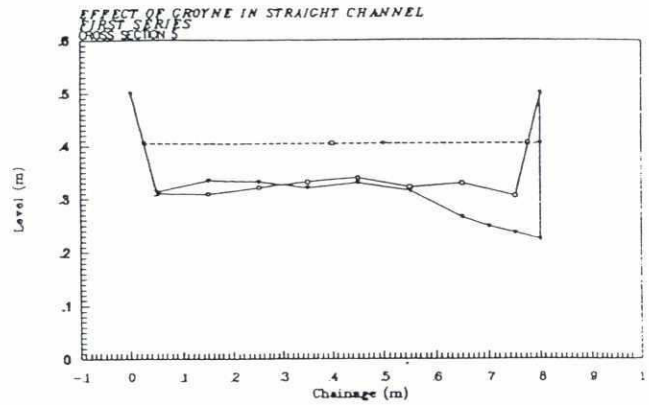
—□— Bed sur in CCWG1 —□— Water sur. in CCWG1 —v— Bed sur in SCWG1 —v— Water sur. in SCWG1

Fig. 4-1

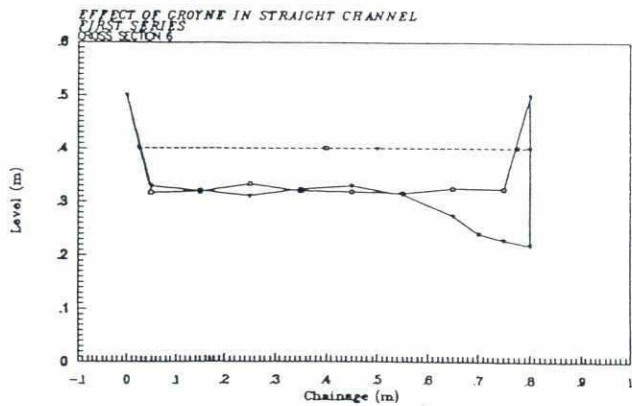
Comparison of cross-sections measured during the experiments SCWG1 and CCWG1 showing the bend scour



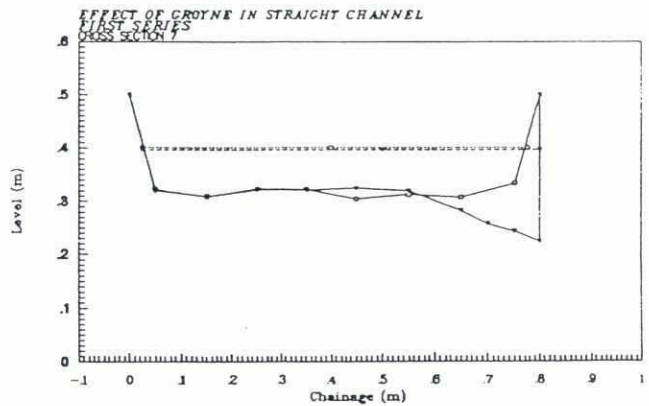
a. Section 4



b. Section 5



c. Section 6



d. Section 7

-□- Bed sur. (SCIG 2) -□- Water sur. (SCIG 2) -v- Bed sur. (SCIG 2) -v- Water sur. (SCIG 2)

$Q = 0.021 \text{ m}^3/\text{s}$
 $B = 0.80 \text{ m}$
 $D(50) = 0.210 \text{ mm}$

Fig. 4-2 Comparison of cross-sections measured during the experiments SCWG1 and SCIG1 showing the local scour

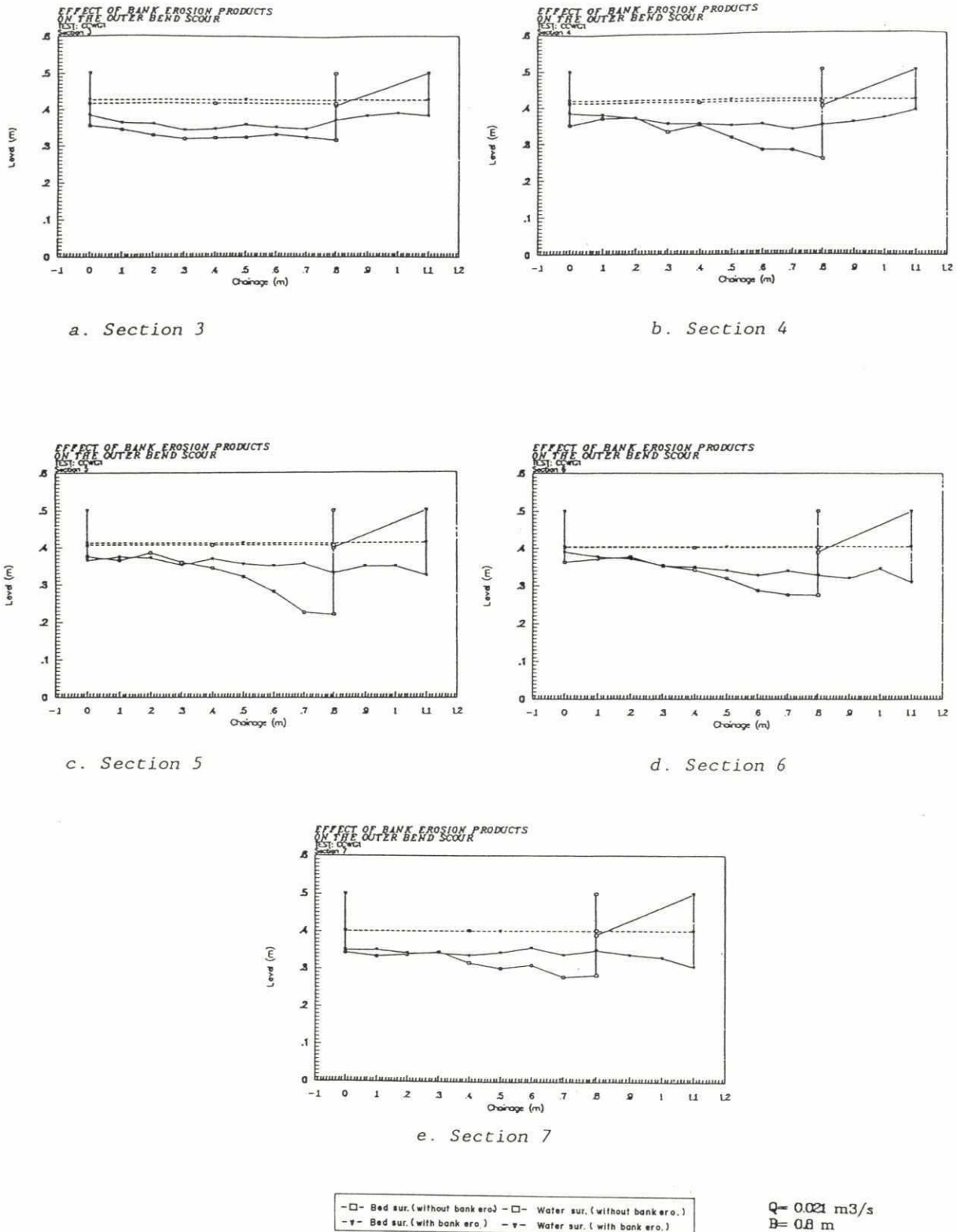


Fig. 4-3 Comparison of cross-sections measured during the experiments SCWG1 and CCWIG1 showing the effect of bank erosion on the cross-sectional profile

5 ANALYSIS

Here the analysis is limited to the relationship to be used for the prediction of combined scour near groynes along outer bends. Fig. 5-1 presents the results of the series 1 and 2 as far as h_o , h_l , h_b and h_{bl} is concerned. In Table 5-1 the values of $\Delta h_l + \Delta h_b$ and Δh_{bl} are determined and a comparison is made between $\Delta h_l + \Delta h_b$ and Δh_{bl} . It is observed that there is a very good correspondence between $\Delta h_l + \Delta h_b$ and Δh_{bl} . It may therefore be concluded that a prediction of the combined scour near groynes in outer bend is obtained when the following formula is used:

$$h_{bl} = h_o + \Delta h_b + \Delta h_l$$

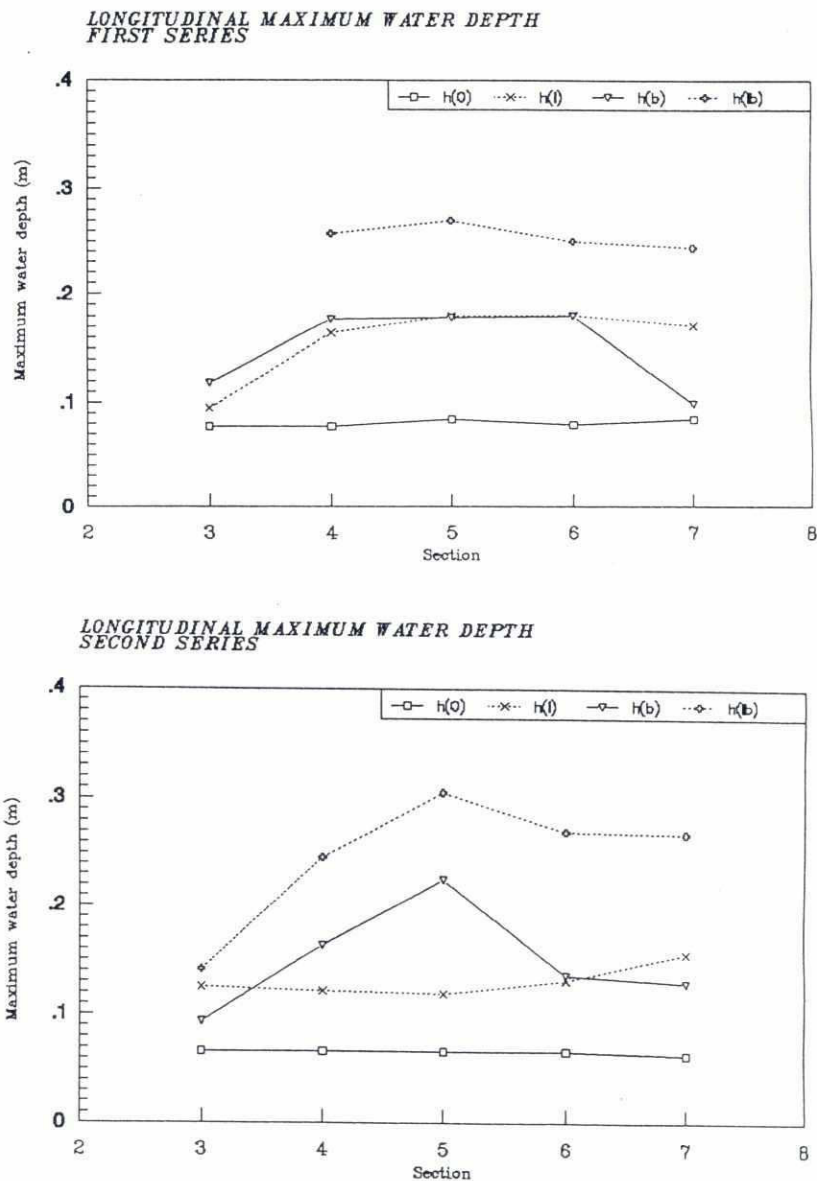


Fig. 5-1 Comparison of the maximum depth measured per cross-section for the different experiments.

a. First series:								
Test: Section	SCWG1 h(o)	SCWG1. 2 h(l)	CCWG1 h(b)	CCIG1 h(lb)	Delta (h(l)	Delta h(b)	Delta h(l+b)	Delta h(lb)
3	.077	.095	.118	-	.018	.041	.059	-
4	.077	.164	.177	.257	.087	.1	.187	.18
5	.084	.18	.179	.27	.096	.095	.191	.186
6	.079	.181	.18	.25	.102	.101	.203	.171
7	.084	.171	.099	.244	.087	.015	.102	.16
b. Second series:								
Test: Section	SCWG2 h(o)	SCIG2 h(l)	CCWG2 h(b)	CCIG2 h(lb)	Delta h(l)	Delta h(b)	Delta h(l+b)	Delta h(lb)
3	.066	.125	.093	.141	.059	.027	.086	.075
4	.066	.121	.163	.246	.055	.097	.152	.18
5	.066	.119	.225	.305	.053	.159	.212	.239
6	.066	.131	.136	.27	.065	.07	.135	.204
7	.063	.156	.129	.268	.093	.066	.159	.205

Table 5-1 Further analysis of the experience during the series 1 and 2

This good correspondence is also shown in Fig. 5-1, where a comparison is made between $h_o + \Delta h_l + \Delta h_b$ and $h_o + \Delta h_{lb}$.

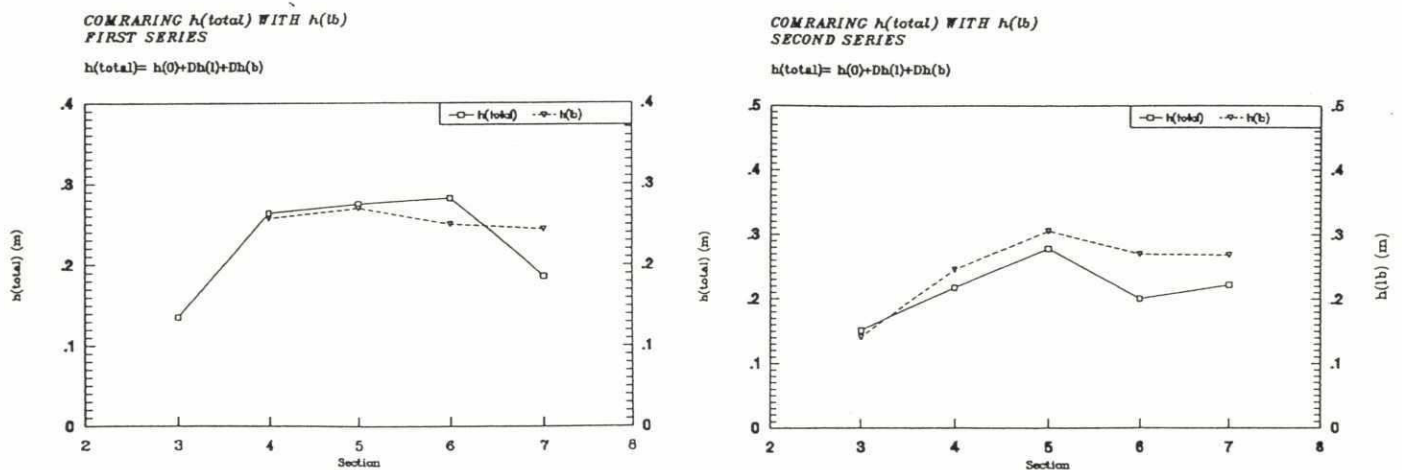


Fig. 5-2 Testing of proposed relationship

6 CONCLUSION AND RECOMMENDATIONS

From the above analysis of the results it was concluded that the approach followed (with series of four comparable test in straight and curved channels, and with and without groynes) has been quite successful and allows for the preparation of recommendations for the design of groynes along outer bends.

The following simple design rule is tentatively recommended :

$$h_{bl} = h_o + \Delta h_b + \Delta h_l$$

where h_{bl} = scour depth near groynes in outer bends, h_o = average water depth for straight channel, Δh_b = additional scour of channel with average depth h_o due to bend scour, and Δh_l = additional scour due to local scour in a straight channel due to groynes.

Furthermore it was demonstrated that in rivers with severe bank erosion like in the Jamuna River, the measured bend profiles should be corrected for the influence of the bank erosion products, before the local scour near groynes in bends is computed according to the above design rule. Finally it is concluded that additional studies both in laboratory channels under controlled conditions and in the field are required to verify the recommendations given on the basis of this exploring investigation.

More detailed conclusions and recommendations are due in the forthcoming Technical Report with a detailed account of the present study.

ANNEX 4

HYDROLOGIC DATA

ANNEX 4**HYDROLOGIC DATA****Table of Contents**

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4	FLOOD FREQUENCY ANALYSIS	4
5	LOW WATER ANALYSIS	5
6	SHORT WATER LEVEL RISE AND FALL ANALYSIS	7

FIGURES :

- A4-1 Discharge distributions in Bahadurabad station
- A4-2 Frequency and duration curves of the water levels
in the gauge station Bahadurabad
- A4-3 Frequency analysis of short rises and falls of
the water level at Bahadurabad

1 INTRODUCTION

The both projects FAP 21/ 22 require hydrologic data for the design of the bank protection measures at the test sites and for the estimation of the effectiveness of river training measures. These hydrologic data are collected and analysed in an hydrologic study, which has been carried out from mid February to the beginning of April 1992. In this period a first selection of the location of possible test sites has been made on basis of the available morphologic data. it is mentioned that the hydrologic study does not include the results of the field surveys near the selected test areas.

The final results of run 6 from FAP 25 have been stored in the data base of the project, but a updating of this study may require a additional input in the next years.

The objectives of the hydrology study is to provide hydrological boundary conditions in conjunction with FAP 25 for the selected sites along the Jamuna river. These objectives include the analysis of a kind of construction window during low water periods and the analysis of quick rises and falls in the water level as a boundary for the study of groundwater flow near the test sites. Unfortunately it was not possible during this study to analyse all required hydrologic boundary conditions as rating curves, wave and wind data. To complete the hydrologic study for this project it is desirable to include these items in an additional study.

2 DATA COLLECTION

A satisfactory network of hydrological stations in the Jamuna river catchment in Bangladesh has been established by the Bangladesh Water Development Board, (BWDB), which is the main source of hydrological data for this project. In general, hydrological data related to the Jamuna River catchment within Bangladesh are available from the early 1960's. A data availability directory has been produced by FAP 25.

The hydrological data of interest for the hydrological study of this project were collected from several sources and stored in a computer database in the Consultant's office. The emphasis was placed on available computerized data, but also some additional data was stored manually.

The rainfall data has been stored in a database and these data can be used as input for the rainfall-runoff module of the 1-dimensional model (NAM module) to run together with the general model or with one of the regional models.

An extensive network of water level gauging stations on the Jamuna river and its distributaries has been established also by the BWDB. The data of some secondary stations have been collected mainly with the purpose to improve, if necessary the 1-dimensional model and to avoid an additional data collection during the study phase of this project.

The accurate location of these gauging stations has been determined recently by various teams using Global Positioning Systems. From the comparison with the results of the previous applied methods it is shown that a more accurate determination of the location of these stations is obtained with a GPS .

The reliability of the zero datum of these stations, especially between right bank and left bank of the Jamuna river is still pending. The preliminary results of the second order bench marks checks indicate a vertical difference of about 0.20 m between the bench marks on the left bank and the bench marks on the right bank.

For the secondary stations the quality of the data was checked by visual inspection of the daily data, computation of the one day gradient (water level difference between 2 days) and comparison with the other stations.

The water levels, which were calculated by the 1-D model for the 25 year period, can be used to determine design conditions near the test areas. These water levels are stored in a database.

Discharge data of different stations in Bangladesh have been collected. The station in Bahadurabad is the most important station along the Jamuna river, because it is the only station where the discharge is currently being measured regularly by the BWDB.

In the Flood Hydrology Study discharges calculated by using a set of improved rating curves with consistent extrapolation characteristics. These discharge data have also been collected for the project database and will be used instead of the uncorrected daily data.

It should be mentioned that the accuracy of the discharge measurements has been estimated at some 20-30 %, especially for the estimation of the maximum discharge. For other locations than Bahadurabad the discharges, which were calculated by the 1-D model for the 25 year period, have been used to determine design conditions near the test areas. These discharges are stored in a database.

The General Model (GM) is a hydrodynamic model of the main rivers in Bangladesh. This model has been developed at the Surface Water Modelling Centre (SWMC) in recent years. Based on the hydrodynamic module of Mike 11 Software, the FAP 25/GM model is a reduced version of the GM with runs for a long period from 1965 to 1989 with a fixed bed.

At this moment the so called run 6 of FAP 25/GM is considered as the final run for the existing situation with a recalibrated model. It is mentioned that the "BRTS Chainage" has been used for run 6 instead of the "GM" chainage used for run 5. To avoid any confusion, only the "BRTS" chainage has been used in this study.

Although the systematic errors between measured and calculated time series at a station will be assessed in detail by FAP 25 in their final report, some quick checks have already been done in this hydrologic study.

The collected hydrologic data have been stored logically into several databases:

Lotus files: Some BWDB data are available in data files, which have been made with a spreadsheet program, Lotus, with 16 water levels gauge stations and 2 discharge stations along the Jamuna river.

General Model database : Data used or calculated by this hydrodynamic model are available in two kinds of databases, which are part of the Mike II software package, which has been developed by the DANISH HYDRAULIC INSTITUTE.

HYMOS database: A hydrological database using HYMOS software has been built and checked to facilitate data interpretation. This software program is a hydrological data management and processing software package developed by DELFT HYDRAULICS. It is pointed out that all the basic data into HYMOS have been introduced from the Lotus files or the Mike 11 databases.

Additional analyses (like rating curves or any other data interpretation) can be carried out easily with the help of this HYMOS database and using the same set of data.

3 TEST AREAS

At the beginning of this hydrologic study it was decided to use the calculated hydrologic data from the FAP 25 / GM-model for an hydrologic analysis for each test area. But because of the results of run 6 were not finalized at the beginning of the study and a BWDB station is located in the vicinity of each test area, an analysis of the most suitable data has been done site by site, keeping in mind that for the modelling activities, to be consistent, the results of the FAP 25 / GM model will **always** be used as boundaries conditions for the determination of the decisive data in design conditions.

Kamarjani

The monthly maximum, minimum and average have been compared with the results of J63 (results of the model for the chainage 63 km), but the differences between observed and

computed water level are too big to be easily explained: an average difference of 0.56m in the water level in a period of several years, but up to 1.4m in annual average. The field measurements of this station cannot be used without a complete investigation of these differences in the water level, however, this is not included in the scope of this hydrologic study. Therefore the computed water level have been used. The computed discharge has been taken from the chainage 67 km (Q67).

Bahadurabad

The observed data has been used for the analysis. For comparison, the computed water levels at the station J85 have also been produced. The difference between observed and computed water levels (J85) is very small in average, 0.06 m, but with a range from -0.36 m to 0.57 m for the annual average difference.

Chandanbaisa

The BWDB station 15J Mathurapara is located 3 kilometer upstream the test area Chandanbaisa. After the finalization of the second order bench marks check a more accurate zero datum of this station can be determined probably. Because only eight years of water level data of this station are available, and to be consistent with the Flood Hydrology Study, the calculated water levels of FAP 25/GM have been used.

Nakalia

The BWDB station Mathura (50.3) is located downstream the test area and the readings of this station have been checked against the water levels in cross section J220. Although the results of the two stations generally match well, this means only a small difference of 0.05 m in average, the differences in yearly average ranges from -0.70 to 0.65 m. The statistical analysis of the observed data may be more accurate the computed data and may be used for the engineering study.

4 FLOOD FREQUENCY ANALYSIS

With a frequency analysis of the available data the design water levels and the design discharges have been established. For these analysis HYMOS software has been used. The recommendations, which were made by FAP 25 for the selection of probability distributions, have been used as guideline for this hydrologic study.

The maximum likelihood method has been used to estimate the parameters.

The 3 parameter log-normal distribution has been selected for the statistical analysis of the maximum annual water levels, and the seasonal discharges. For the maximum annual discharges, the Gumbel distribution has been selected. The design water levels for a selected return period of 2, 5, 10, 25, 50 and 100 year are given for the cross sections near the test areas in table A4-1.

Station name	HYMOS Code	Return period					
		2	5	10	25	50	100
		Water level (in m)					
KAMARJANI	J63	21.60	22.01	22.25	22.52	22.71	22.89
BAHADURABAD	J85	19.82	20.21	20.43	20.67	20.83	20.98
BAHADURABAD	BWDB46.9L	19.78	20.04	20.20	20.42	20.57	20.72
CHANDAN.	J118	16.78	17.18	17.43	17.71	17.91	18.11
NAKALIA	J205	10.99	11.40	11.64	11.93	12.13	12.32
MATHURA	BWDB50_3	10.01	10.46	10.76	11.14	11.41	11.69
MATHURA	J220	9.77	10.13	10.38	10.71	10.96	11.22

Water level (in m PWD)

Table A4-1: Maximum water level for selected return periods

The design water levels should include a safety margin. Safety margins, accounting for the effects of random morphological processes, for model errors, and the likely underestimation of extreme events, due to the shortness of available records of observations, have been proposed by FAP 25 / CAT team for the determination of the design water level.

Safety margins varying from 0.45 m for the 100 years maximum flood level to 0.30 m for the 10 years maximum flood level were proposed for the major rivers. These safety margins have to be **added** to the free board of the bank protections, accounting for wind set-up, wave run-up and others possible safety factors.

From a frequency analysis of the calculated daily discharges during a period of 25 years the magnitude of the discharges corresponding to a selected return period has been determined. A Gumbel distribution has been fitted to the annual maximum discharges, see Figure A4-1. From the graphs it can be seen that the Gumbel distribution fits very well the data points. The design discharges for a selected return period of 2, 5, 10, 25, 50 and 100 year are given for the test areas.

Station name	Return period					
	2	5	10	25	50	100
	Maximum discharge (in m ³ /s)					
KAMARJANI	70251	82722	90979	101412	109151	116834
BAHADURABAD	67621	79144	86774	96413	103565	110663
BAHADURABAD	64267	75670	83220	92759	99836	106861
NAKALIA	56435	64002	69012	75342	80038	84700

Table A4-2: Maximum discharge for selected return period

Only one parameter, as the maximum discharge, is not necessarily sufficient to characterize the flood discharge and the morphologic consequences of a flood discharge. As an alternative parameter the mean discharge during the months July, August and September has been chosen. For the station in Bahadurabad this parameter has been determined from the calculated daily discharges during the 25 year period.

Station name	Return period						Type of Distribution
	2	5	10	25	50	100	
	Mean maximum discharge (in m3/s)						
BAHADURABAD (BWDB 46.9I)	41966	47142	50467	54576	57581	60548	Log normal
	41663	46983	50739	55768	59715	63825	EVT 2

EVT = Extreme value type 2

**Table A4-3: Mean of 3 months maximum discharge
for a selected return period**

A return period means that the considered event will be observed on an average n times in a given period, in which n is equal to the given period divided by the return period. For a good understanding of the hydrologic behaviour of the peak water level and the peak discharge the relative meaning of a return period is essential, or more specific it means that a return period of a flood depends on the parameters, which are chosen and on the selected locations.

From the analysis it can be seen that in 1988 the maximum discharge had a return period of 100 year, but the maximum water level was not so extreme: a return period of only 60 years. The selection of a return period of one parameter in one location is not sufficient to characterize a hydrologic event as a typical design event. It is the reason why long-term simulation with the regional models for the period 1965-1989 is advocated by FAP 25.

The statistical representativeness of the period 1965-1990 for the whole century has been assessed by the Flood Hydrology Study and this analysis indicates that the period 1965-1989 can be considered to be typical for this century and so designs can be based on the statistical analysis of output from this period.

5 LOW WATER ANALYSIS

A kind of a construction window has been defined by the number of days during which the water level is below a certain level with a certain probability.

From the duration curve, the level not surpassed during periods of 60, 90, 120 and 180 days can be extracted for the 10%, 50%, 90% frequencies. The 90% value for 90 days indicates

that in 9 years of a period of 10 years, one can expect during at least 90 days per year a water level below this value, see table A4-4 and also upper graph in Fig. A4-2. With this date a construction window regarding the water level can be determined.

The small variations in the water level during the receding limb of the hydrograph in the dry season are remarkable, therefore the differences between the 10%, 50 % and 90% water level is relatively small. As soon as the monsoon period starts the variation in the water level at a certain date increases from year to year.

Duration in days	Low Water Level (in m PWD)			Date of occurrence of the 90% low water level	
	for the frequency			Start date & 90 % range	End date & 90 % range
	10 %	50 %	90 %		
60 days	13.21	13.54	13.89	362 (351 - 13)	93 (72 - 106)
90 days	13.48	13.81	14.19	347 (340 - 362)	106 (86 - 114)
120 days	13.89	14.20	14.74	329 (318 - 342)	112 (97 - 127)
180 days	14.88	15.43	16.33	300 (287 - 307)	149 (121 - 165)

Day number 1 = 1st January

Table A4-4: Low water analysis at Bahadurabad (BWDB 46_91)

6 SHORT WATER LEVEL RISE AND FALL ANALYSIS

Relatively short rises and falls in the water level can be observed in all water level stations along the Jamuna river during the rising limb of the hydrograph, that is during the start of the monsoon period, see for example Fig. A4-2.

The differences of water levels between 1, 2, 3, 5 and 7 sequential days (the gradients "G1", "G2", "G3", "G5", "G7") have been computed with a frequency analysis from the daily water levels in the 25 year period and the extreme positive and negative values for each day of a year have been calculated.

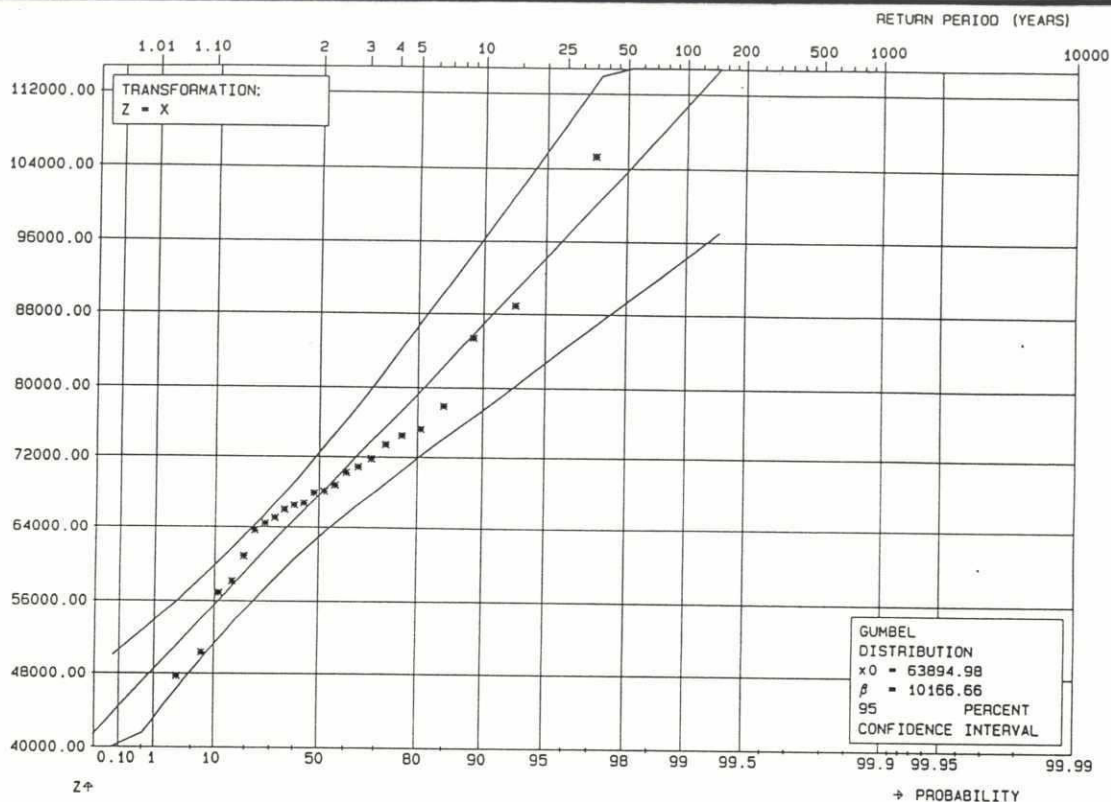
The results of this analysis can be summarized as follows. From the graphs in Fig. A4-3, it is seen that in these gradients per day of a year two different periods can be distinguished:

- * From November to February the water level is changing very slowly with a few centimeter per day without exception.
- * And from April to October the water level can rise or fall suddenly with a few decimeters per day.

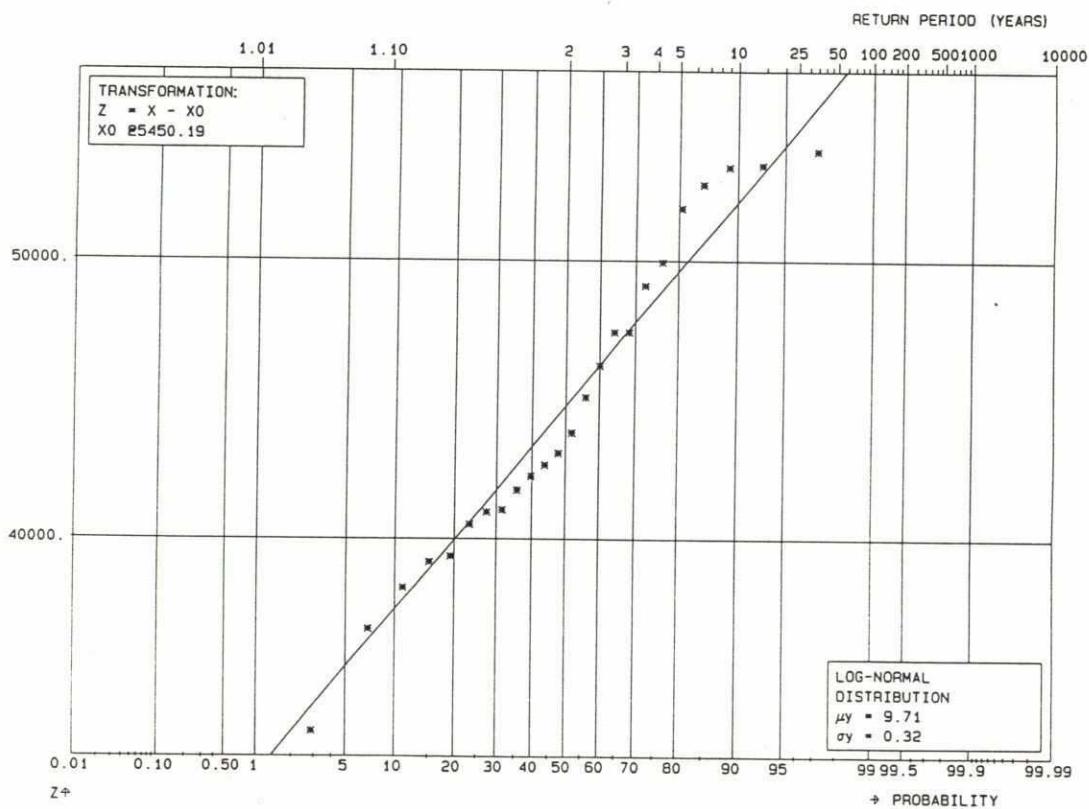
During the rising limb of the hydrograph in April to October the positive gradient is 0.25 to 0.5 m per day. This celerity in rising waterlevels can be continued for an other day with a total rise of 0.5 to 1.0 m. In a period of 5 sequential days the water level can rise with an average celerity of 0.2 to 0.5 m per day.

During the rising limb of the hydrograph the negative gradient is 0.1 to 0.25 m per day, the celerity of the falling water level is continued for a second day and a third sequential day with a total negative gradient of 0.3 to 0.75 m per period of 3 sequential days. For longer periods the average celerity per day is reduced.

The variation in the results is very small between different locations along the Jamuna river.



ANNUAL MAXIMUM DISCHARGE



AVERAGE DISCHARGE OF THE PERIOD JULY - SEPTEMBER

DISCHARGE DISTRIBUTIONS
IN BAHADURABAD STATION

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

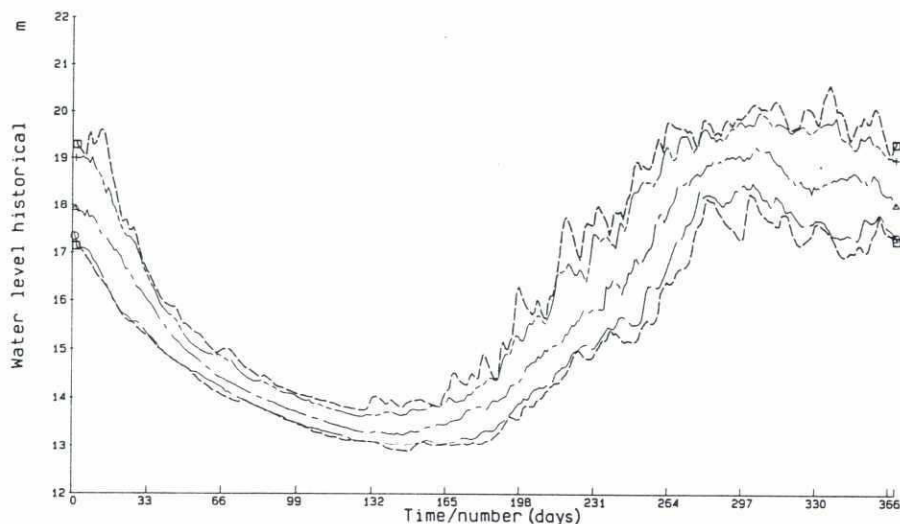
INTERIM REPORT

FIG. A4 - I

80

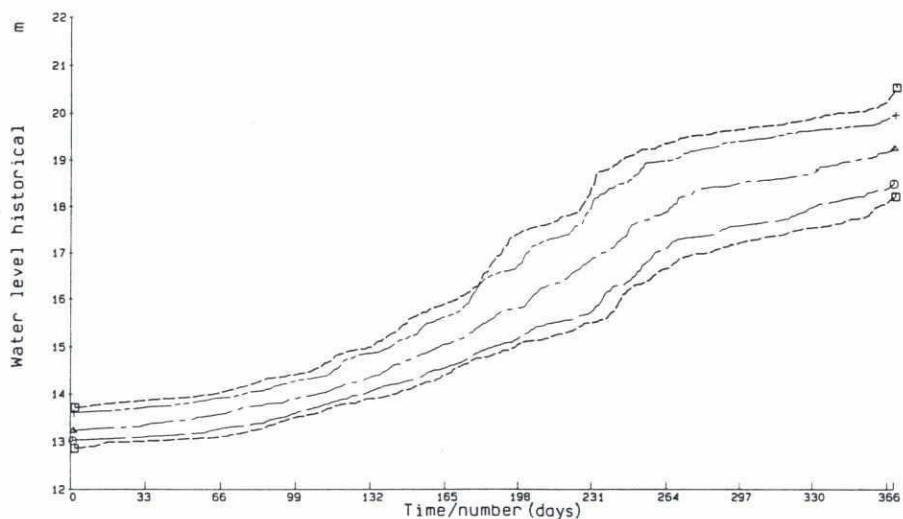
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 Period : 25 years

Legend
 — 10 %
 — 50 %
 — 90 %
 □ Minimum
 □ Maximum



Duration curves
 BWDB46_9L HH3 1
 Start date: 1965 10 1 0 1
 Period : 25 years

Legend
 — 10 %
 — 50 %
 — 90 %
 □ Minimum
 □ Maximum
 + Average



**FREQUENCY AND DURATION CURVES OF
 THE WATER LEVELS IN THE GANGE
 STATION OF BAHADURABAD**

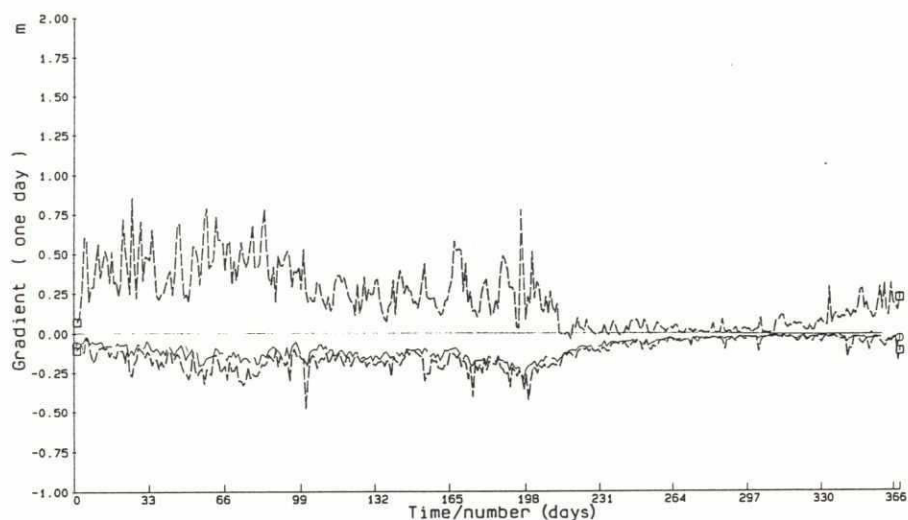
FAP - 21/22
 BANK PROTECTION AND RIVER TRAINING/
 (AFPM) PILOT PROJECT

INTERIM REPORT

FIG. A 4 - 2

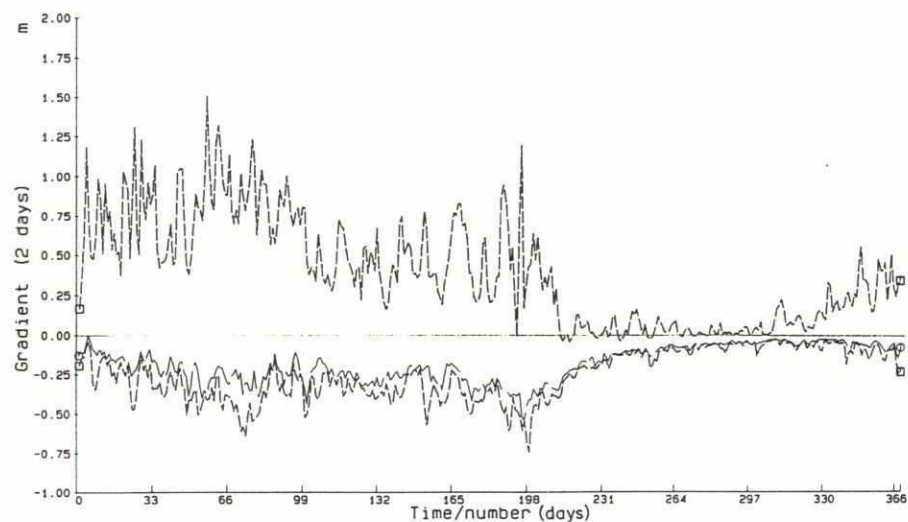
Frequency curves
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Start date: 1965 4 1 0 1
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Legend
10 %
Minimum
Maximum



Frequency curves
BWDB46_9L G23 1
Start date: 1965 4 1 0 1
Period : 25 years

Legend
10 %
Minimum
Maximum



**FREQUENCY ANALYSIS OF THE SHORT
RISES AND FALLS OF THE WATER LEVEL
AT BAHADURABAD**

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INTERIM REPORT

FIG. A4-3



ANNEX 5

SIMPLIFIED MATHEMATICAL MODEL FOR INVESTIGATION OF RECURRENT AFPM MEASURES

ANNEX 5

**SIMPLIFIED MATHEMATICAL MODEL
FOR INVESTIGATION OF
RECURRENT AFPM MEASURES**

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SIMPLIFIED HYDRAULIC MODELS

1 INTRODUCTION

The main objective of the FAP 22 - project is to indicate the viability of using recurrent (low cost !) measures for a systematical narrowing of the Jamuna river or for diverting the flow from sensitive spots (bank erosion). These measures focuss on diversion of flow and sediment discharge (see Inception Report).

As a first step the measures have to be estimated on their ability to increase sedimentation in the outer branches (near problem spots) and to divert the flow to the main channels of the Jamuna River. This process should be initiated at important bifurcations (see figure 1).

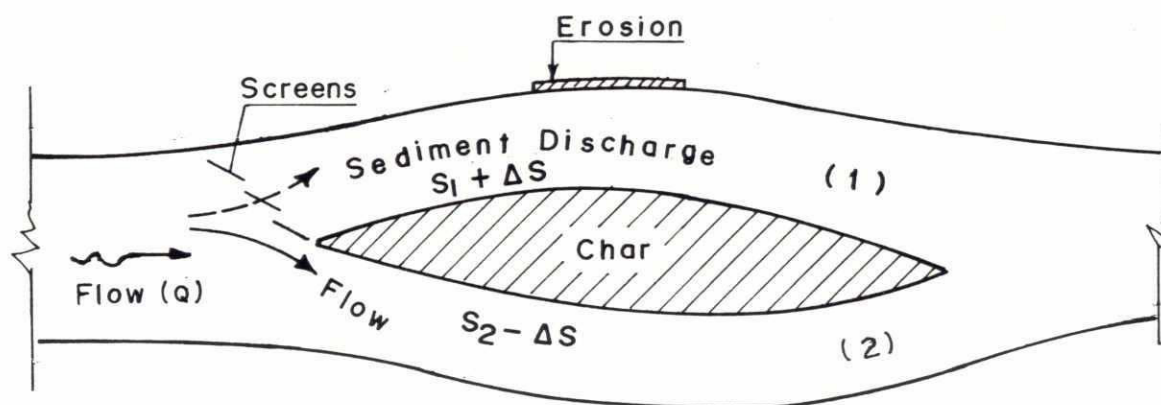


Fig. 1: Typical problem spot.

Due to the great number of bifurcations and various dimensions (length, cross-sections) of the branches the sensitivity analysis should possibly be done by using simplified hydraulic models from which the sensitivity of various parameters (such as: water depth, discharge, sediment transport, roughness, etc.) can be assessed easily.

The setup of these simplified hydraulic models will be done in steps in order to achieve some basic understanding of the physical process in the various problem channels (indicated by channel 1 in Fig. 1). The calibration and verification will be done with results of MIKE 11 computations.

The following steps are envisaged at this moment :

1. Consider channel 1, as a separate channel with constant discharge Q . The sedimentation process due to a constant sediment supply ΔS has to be predicted.

2. The same procedure as step 1 with the difference that a constant flow discharge ΔQ will be subtracted from channel 1 while no supply of ΔS will take place. The description of the sedimentation process should be the result.
3. Introduction of a time dependant $Q(t)$ in channel 1, in such a way that water levels at the entrance and the end of the channels should remain more or less constant. Calibration to MIKE 11 results are necessary.
4. Combinations of ΔS supplies and ΔQ subtractions in channel 1.

This report will describe the set up of the simplified model as described in step 1. The physical background will be presented including the calibration and verification to MIKE 11 results.

2. SINGLE CHANNEL WITH CONSTANT Δs SUPPLY

In general the hydraulic and morphological process in a channel with constant discharge of $q (= Q/B)$ can be described with motion and continuity equations for both the water and sediment discharges. Due to the relatively wide channels ($B \gg h$) of the Jamuna river it is justified to consider the hydraulic and morphological process per unit channel width. According to these considerations the following equations (see also Fig. 2) have been adopted:

$$(i) \text{ Water Motion} : u \frac{\delta u}{\delta x} + g \frac{\delta(h + z_b)}{\delta x} - gi + \frac{gu^2}{C^2 h} = 0 \quad (1)$$

$$(ii) \text{ Continuity} : q = u.h \quad (2)$$

$$(iii) \text{ Sediment Motion} : s = a_s u^5 \text{ where } (a_s = \frac{0.05}{\Delta^2 \sqrt{g} C^3 D_{s0}}) \quad (3)$$

$$(iv) \text{ Continuity of Sediment} : (1 - \epsilon) \frac{\delta z_b}{\delta t} + \frac{\delta s}{\delta x} = 0 \quad (4)$$

The used symbols are as follows :

u = flow velocity	(m/s)
h = local water depth	(m)
g = acceleration of gravity ($=9.81 \text{ m/s}^2$)	(m/s ²)
i = water level slope	(-)
C = chezy - C roughness coefficient	(m ^{1/2} /s)
s = sediment transport per unit width	(m ² /s)
Δ = specific mass = 1.65	(-)
ϵ = porosity of sand (≈ 0.4)	(-)
z_b = bottom level	(m)

Note, that sediment motion is defined according to Engelund-Hansen.

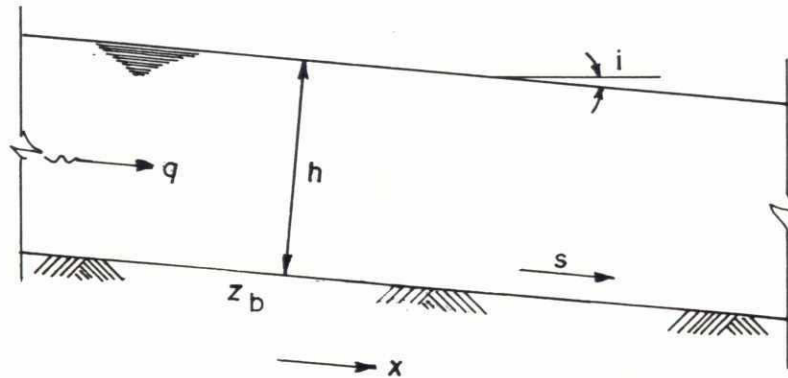


Fig. 2: Basic symbols

From the basic set of equations the following equations for bottom level changes and water level changes have been derived :

$$\frac{\delta z_b}{\delta t} + c_0 \frac{\delta z_b}{\delta x} = c_0 \cdot c_1 \quad (5)$$

$$\frac{\delta h}{\delta x} = p \left(c_1 - \frac{\delta z_b}{\delta x} \right) \quad (6)$$

$$c_0 = 5p a_s \frac{q^5}{h^6 (1 - \varepsilon)} \quad (\text{celerity term}) \quad (7)$$

$$c_1 = i - \frac{q^2}{c^2 h^3} \quad (\text{friction term}) \quad (8)$$

$$p = \frac{g}{\left(g - \frac{q^2}{h^3} \right)} \quad (\text{deceleration term}) \quad (9)$$

Under equilibrium conditions (for $h = h_{eq}$) it is clear that $c_1 = 0$, while the parameter c_0 can be considered as a specific constant propagation celerity of a bed level disturbance. From the equations it can be seen, that for bottom changes (h variable) the parameters c_0 and c_1 are varying. Within a model as MIKE 11 these parameters are computed on each location. The underlying report describes the simplification (or averaging) of these parameters for the situation of a bottom level sedimentation due to a constant input of q_s on a certain location along the river (see Fig. 3).

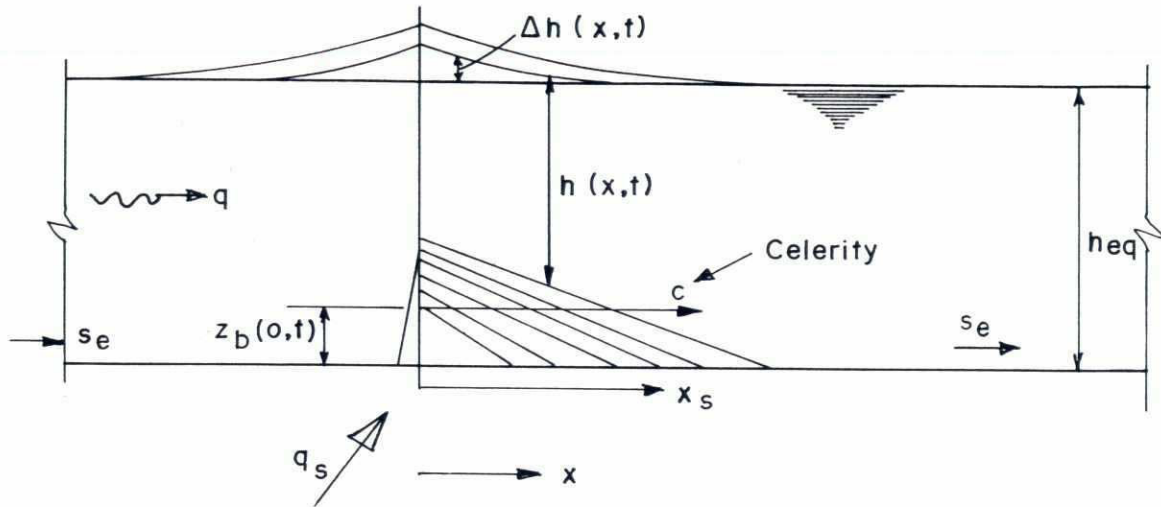


Fig. 3: Sedimentation due to a constant q_s input

At time $t = 0$ the supply of sediment on location $x = 0$ starts. This means that the total sediment load on this location instantaneously grows from s_e to $s_e + q_s$. Due to the lower sediment discharge capacity of the river (s_e) the bottom level will rise, while the front of the sedimentation propagates with celerity c .

In analogy with the basic Equation (5) the sedimentation process can mainly be described by the following equation for the bottom level height $z_b(x,t)$:

$$\frac{\delta z_b}{\delta t} + c \frac{\delta z_b}{\delta x} = R \quad (10)$$

in which R can be considered as a damping coefficient. The influence of this term will be described.

The propagation celerity c will not be constant, which clearly can be concluded from the sediment balance of the bottom disturbance. From MIKE 11 results it has been observed that the slope of the sedimentation can be assumed as a negative sloping front (see Fig. 3). Assuming a time dependent celerity $c(t)$ and bottom level height $z_b(0,t) = z$ the following balance should hold (see Fig. 4):

$$z \cdot c \cdot dt = \frac{q_s \cdot dt}{(1 - \varepsilon)} \quad (11)$$

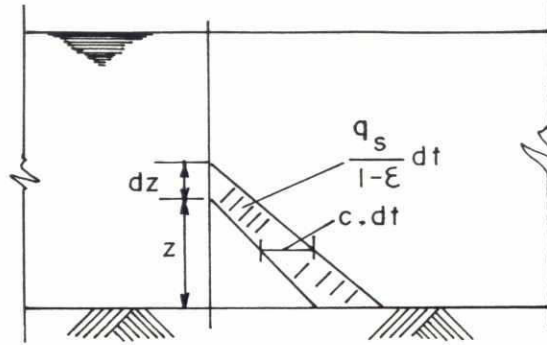


Fig. 4: Sediment Balance



A schematic presentation of Equation (11) is given in Fig. 5.

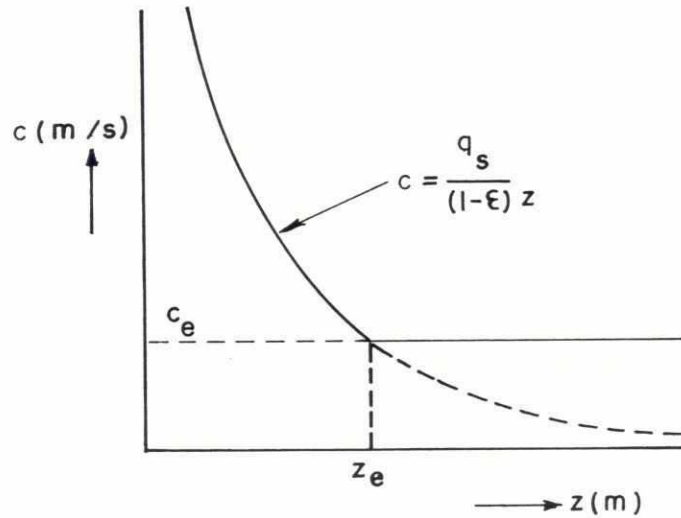


Fig. 5: Celerity as a function of the sedimentation height

From Fig. 5 it becomes clear, that the validity of Equation (11) concerns the phase, in which the bottom level height z is less than a value z_e . At this moment the celerity has been decreased to a value c_e , which can be considered as the equilibrium celerity defined by the river characteristics. Actually the value of c_e is given by (see Equation 7) :

$$c_e = \frac{q_s}{(1 - \epsilon)z_e} \quad (12)$$

$$c_e = \frac{5s_e}{h_e} \cdot \frac{1}{(1 - \epsilon)} \quad (13)$$

The symbols in equations (12) and (13) are as follows:

c_e = equilibrium celerity	(m/s)
$q_s = n s_e$ = sediment supply related to the equilibrium transport s_e	(m ² /s)
$s_e = a_s (q/h_e)^5$ = equilibrium transport	(m ² /s)
n = ratio q_s/s_e	(-)
z_e = equilibrium bottom level height	(m)

From Equations (12) and (13) the value of z_c should meet to :

$$z_c = \frac{1}{5} \cdot \frac{q_s}{s_c} \cdot h_c = \frac{n}{5} \cdot h_c \quad (14)$$

The following assumptions have been made to solve the basic Equation (11) :

(i) the general solution is given by (see also Fig. 6):

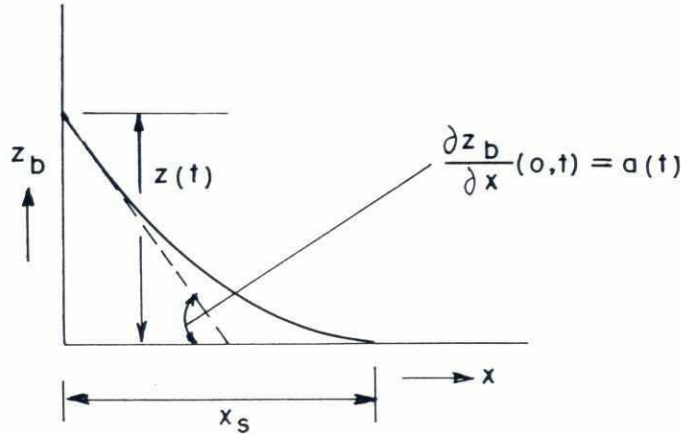


Fig. 6: Typical front of the sedimentation

$$z_b(x,t) = z(t) \cdot \left(1 - \frac{x}{x_s}\right)^\alpha \quad (15)$$

$$(ii) \quad \frac{\delta z_b}{\delta x}(0,t) = a(t) = \left(\frac{z_c}{z}\right)^\beta a_c \quad (16)$$

From these assumptions it can be seen that for $\alpha = 1$ the front of the sedimentation is linear. However, from MIKE 11 results it has been observed that for high values of the sediment supply ($n \gg 1$) the front is no longer linear. Therefore a variable α has been taken. The gradient of the front on $x = 0$ has been assumed to behave according to Equation (16) in which a_c is the gradient of the front when $z_b(0,t) = z_c$.

The analysis has been focussed on the calibration of the α and β parameters to the results of MIKE 11.

According to Equations (15) and (16) the following should hold :

$$\frac{\delta z_b}{\delta x}(0,t) = -\alpha \frac{z}{x_s} = a(t) \quad (17)$$

while using Equation (15) for defining the gradient $\delta z_b/\delta t$ and $\delta z_b/\delta x$ and substituting them into the basic equation (10) the following result has been obtained:

$$\frac{dz}{dt} - \alpha \frac{z}{x_s} \cdot c - \frac{x}{x_s} \left(\frac{dz}{dt} - \frac{z}{x_s} \frac{dx_s}{dt} \right) = R \quad (18)$$

in which (see also Fig. 6) :

c = celerity of sedimentation front (Eq. 11 and Fig. 5)	(m/s)
z = bottom level height on x = 0	(m)
x _s = sedimentation length	(m)
α = power of equation (15)	(-)
q _s = sediment input (= ns _c)	(m ² /s)
x = distance from origin	(m)
R = damping term	(m/s)

From Equation (18) the meaning of the damping term R becomes clear, which has been indicated in the following steps :

(i) Condition R = 0

This condition leads to the following equations (requirements):

$$(1) \frac{dz}{dt} - \alpha \frac{z}{x_s} \cdot c = 0$$

$$(2) \frac{dz}{dt} - \frac{z}{x_s} \frac{dx_s}{dt} = 0$$

The second Equation leads to the condition that the slope of the bottom sedimentation (z/x_s) is constant, which is not in agreement with the observations from MIKE 11 and it does not meet the requirement as specified with Equation (17). Therefore, this situation will not be solved.

(ii) Condition R ≠ 0

A solution of equation (18) could be according to the following approach:

$$\frac{dz}{dt} - \alpha \frac{z}{x_s} \cdot c - \frac{x}{x_s} \left(\frac{dz}{dt} - \frac{z}{x_s} \frac{dx_s}{dt} \right) =$$

$$\frac{dz}{dt} - p_e \alpha \frac{z}{x_s} \cdot c - (1 - p_e) \alpha \frac{z}{x_s} \cdot c - \frac{x}{x_s} \left(\frac{dz}{dt} - \frac{z}{x_s} \frac{dx_s}{dt} \right) = R \quad (19)$$

Assuming that :

$$R + (1 - p_\epsilon) \cdot \alpha \cdot \frac{z}{x_s} \cdot c \approx \frac{1}{\alpha} \cdot \frac{da}{dt} \cdot x \quad (20)$$

in which p_ϵ is a coefficient due to the damping, while $a(t)$ is the slope of the sedimentation front on $x = 0$ (see Fig. 6)

Considerations (19) and (20) lead to the following consequences for Equation (19) :

$$\frac{dz}{dt} - p_\epsilon \cdot \alpha \cdot \frac{z}{x_s} \cdot c - \frac{x}{x_s} \left(\frac{dz}{dt} - \frac{z}{x_s} \frac{dx_s}{dt} + \frac{1}{\alpha} \cdot \frac{da}{dt} \cdot x_s \right) = 0$$

leading to :

$$\frac{dz}{dt} - p_\epsilon \cdot \alpha \cdot \frac{z}{x_s} \cdot c = 0 \quad (21^a)$$

$$\frac{dz}{dt} - \frac{z}{x_s} \cdot \frac{dx_s}{dt} + \frac{1}{\alpha} \cdot \frac{da}{dt} \cdot x_s = 0 \quad (21^b)$$

Equation (21^b) meets fully the requirement as specified with Equation (17). Now the final solution for $z(t)$ is specified with Equation (21^a). Substitution of the boundary condition as given with Equations (16) and (17) into Equation (21^a) leads to the result :

$$\frac{dz}{dt} + p_\epsilon \cdot c \left(\frac{z_\epsilon}{z} \right)^\beta \cdot a_\epsilon = 0 \quad (22)$$

This differential Equation can be solved by using the expression for the celerity c as presented in Fig. 5 :

$$c = \frac{q_s}{(1 - \epsilon) \cdot z} \quad \text{for } z < z_\epsilon \quad (22^a)$$

$$c = \frac{q_s}{(1 - \epsilon) \cdot z_\epsilon} \quad \text{for } z \geq z_\epsilon \quad (22^b)$$

The solution of the differential Equation (22) is :

$$(i) \quad z = z_\epsilon \left(\frac{t}{T_\epsilon} \right)^{\frac{1}{2+\beta}} \quad (\text{for } z < z_\epsilon) \quad (23^a)$$

$$(ii) \quad z = z_\epsilon \left(1 + \frac{1+\beta}{2+\beta} \cdot (t - T_\epsilon) \right)^{\frac{1}{1+\beta}} \quad (\text{for } z \geq z_\epsilon) \quad (23^b)$$

$$(iii) \quad T_\epsilon = - \frac{z_\epsilon^2 \cdot (1 - \epsilon)}{(2 + \beta) \cdot p_\epsilon \cdot a_\epsilon \cdot q_s} \quad (23^c)$$

$$(iv) \quad z_e = \frac{n}{5} \cdot h_e \quad \left(\text{with : } n = \frac{q_s}{s_e} \right) \quad (23^d)$$

$$(v) \quad \frac{dz}{dt} (t = T_e) = \frac{z_e}{2 + \beta} \cdot \frac{1}{T_e} \quad (23^e)$$

The calibration of these simplified results has been done by using MIKE 11 runs. The calibration will be described in Section 3.

The justification of the assumption for the damping R as derived from Equation (20) will be discussed briefly. As already indicated R should be approximated by :

$$R \approx \frac{x}{\alpha} \cdot \frac{da}{dt} - (1 - p_e) \cdot \alpha \cdot c \quad (24)$$

in which :

x = distance from origin (x = 0)	(m)
α = power (to be fitted) (see Eq.15)	(-)
a = slope of front on x = 0 (see Eq.16)	(-)
p_e = coefficient (to be fitted)	(-)
c = celerity of sedimentation front	(m/s)

From the basic Equation (10) the actual sedimentation process has been described by :

$$\frac{\delta z_b}{\delta t} + c \cdot \frac{\delta z_b}{\delta x} = R \quad (25)$$

comparing this result with the original motion equations (see Eq.5, 7 and 8), the following conclusion can be drawn :

$$R \approx c \cdot c_1 \quad (26)$$

in which : c_1 = friction term (-)

For c_1 the following result during the sedimentation process (see Fig. 3) can be assumed:

$$C_1 = i - \frac{q^2}{C^2 h^3(x,t)} = i \left\{ 1 - \left(\frac{h_e}{h} \right)^3 \right\} \quad (27)$$

in which h = local water depth (m)

From Equation (26) and (27) it might be concluded, that R (= damping) is partly determined by the celerity c of the front and partly by a location (x or local h) depending term. This conclusion agrees with the assumed damping function R as presented in Equation (24).

For having some insight in the magnitude of R , Equation (24) has been used to quantify the result on $x = x_s$ and $t = T_e$ using the expression for $a(t)$ as given with Equation (16) as well as the results for $z(t)$ given with Equation (23^b). Moreover, the celerity should have reached the equilibrium celerity c_e (see Equation (13)) on $t = T_e$.

From these considerations the following results has been obtained for $R(x_s, T_e)$:

$$R(x_s, T_e) = c_e (1 - p_e - \beta) \quad (28)$$

Assuming that :

$$R(x_s, t) \approx c_e (1 - p_e - \beta) \left(\frac{t}{T_e}\right)^i \quad (29)$$

in which i = a certain power (-)

it can be concluded that $R \rightarrow 0$ under the condition that $p_e \approx 1 - \beta$, which should satisfy the situation of equilibrium for $x \geq x_s$.

At this moment it is concluded, that the assumed expression for the damping R is sufficiently justified. However, it should be kept in mind that a further elaboration of the expression R during future developments of the simplified models might be necessary.

The following section is completely devoted to the calibration of the parameters α , β (see Equations 15 and 16) and p_e .

3. CALIBRATION OF THE SIMPLIFIED MODEL

The most important characteristics for the calibration of the simplified model are :

T_e = a morphologic time (Eq. 23 ^c)	(years)
a_e = slope of the front on $t = T_e$	(-)
x_s = length of sedimentation	(km)
$b = dz/dt$ on $t = T_e$	(m/yr)

These parameters have been collected from MIKE 11 runs from output as given in the Fig. 7 and 8. The collection of these parameters could be done easily knowing that

$$z_e = \frac{n}{5} h_e$$

in which z_e = equilibrium bottom level height at $x = 0$ (m)
or $z_e = z_b(0, T_e)$ (m)

From the output as given in Fig. 7 it was easy to define b and T_e , while from the output as given in Fig. 8 a_e and X_s were approximated. The final results have been collected in Table 1.

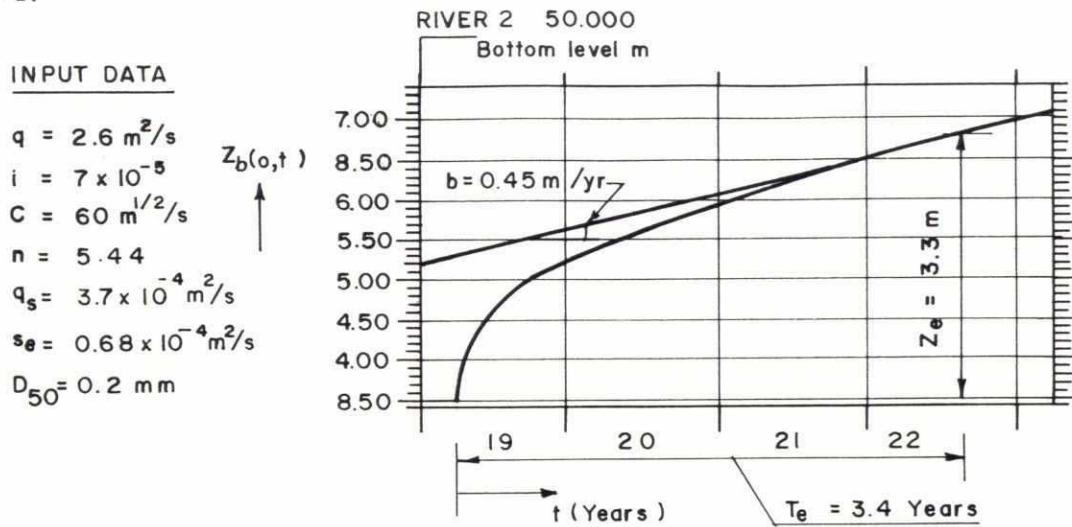


Fig. 7: Determination of b and T_e

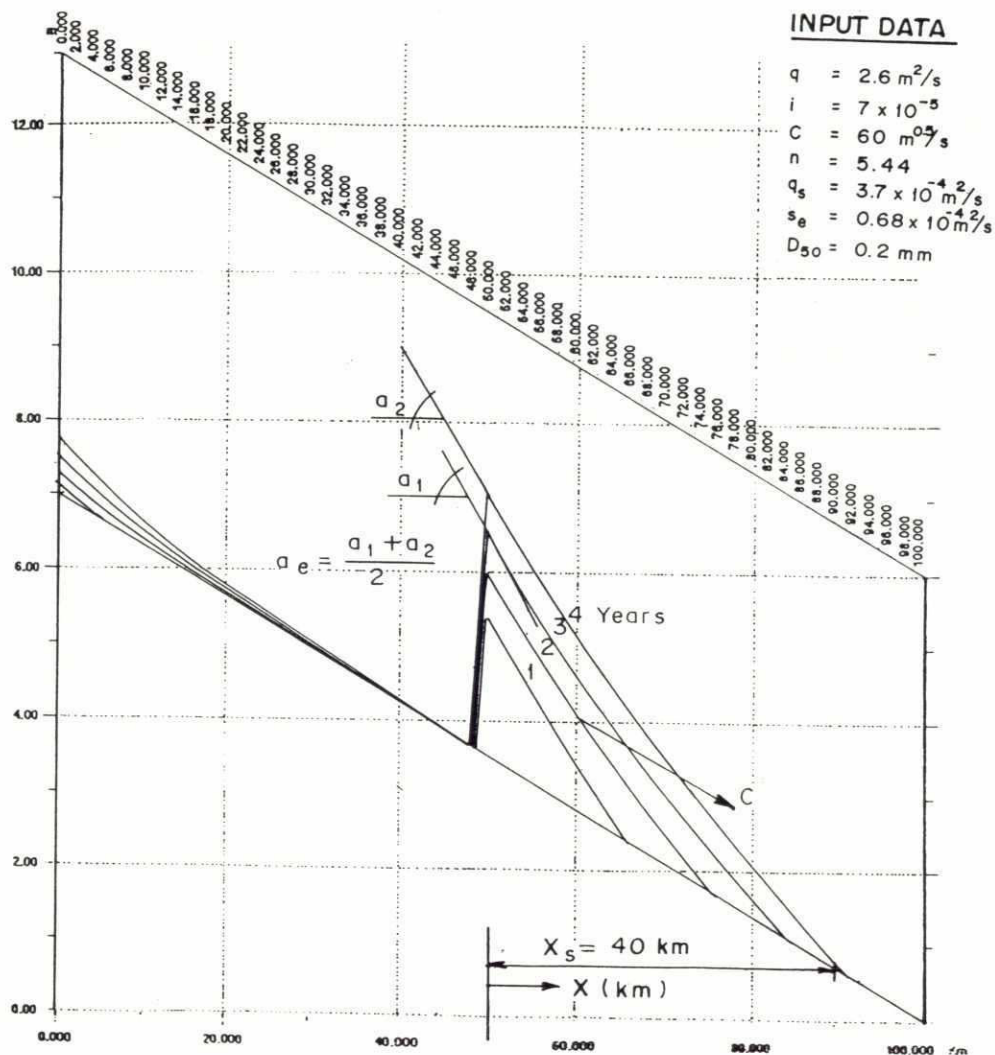


Fig. 8: Determination of a_e and X_s

INPUT: $C = 60 \text{ m}^{1/2}/\text{s}$, $i = 7 \cdot 10^{-5}$						MIKE 11 OUTPUT				CALIBRATION		
Case	$h_e(\text{m})$	$q(\text{m}^2/\text{s})$	$s_e(\text{m}^2/\text{s})^*$	$n(-)^{**}$	$Z_e(\text{m})^{***}$	$T_e(\text{yr})$	$a_e(10^{-5})$	$x_s(\text{km})$	$b(\text{m}/\text{yr})$	α	β	P_e
1	5.1	5.8	2.7	1.37	1.4	1.6	-3.7	41	0.32	1.08	0.73	0.62
2	5.1	5.8	2.7	4.11	4.2	2.1	-10.3	57	0.90	1.40	0.22	0.63
3	3.0	2.6	0.68	5.44	3.3	3.4	-13.0	40	0.45	1.58	0.16	0.58
4	3.0	2.6	0.68	7.35	4.41	3.45	-17.0	45	0.60	1.74	0.13	0.59
5	4.0	4.0	1.40	2.64	2.1	2.2	-6.3	42	0.40	1.26	0.39	0.69
6	7.0	9.3	5.63	0.66	0.92	1.45	-1.7°)	>50°)	0.18°)	- °)	1.5	0.51°)
7	8.0	11.4	8.0	0.46	0.74	1.25	-1.1°)	>50°)	0.14°)	- °)	2.2	0.50°)

- *) s_e values in $10^{-4} \text{ m}^2/\text{s}$
 **) $n = q_s/s_e$
 ***) $Z_e = (n/5) h_e$
 °) $x_s >$ boundary length of computer program. Result probably influenced by boundary conditions in MIKE 11

Table 1 : Calibration of simplified model

According to Equation (30) the results of β have been computed from the MIKE 11 runs as follows:

$$\beta = \frac{z_e}{bT_e} - 2 \quad (30)$$

The results of this exercise have been listed in Table 1 in the row of the calibration.

The results for β indicates a relation between β and n as follows:

$$\beta = \frac{z_e}{bT_e} - 2 \quad (30)$$

The fitted values of β can be compared to the computed values of β as given by Equation (31) in Table 2. The results show a very good agreement for Equation (31).

The next step in the calibration is to find a relation for a_e . The following relation has been assumed for a_e :

$$a_e = \gamma n i \quad (32)$$

in which :

γ = a co-efficient to be fitted

i = natural water level slope

The results of γ has also been listed in Table 2. From these findings it may be concluded that a good approximation for γ is given by :

$$\gamma = - 0.35 \quad (33)$$

Calibration				
Case	n (-)	β (Table 1)	$\beta = 1/n$	γ
1	1.37	0.73	0.73	- 0.38
2	4.11	0.22	0.24	- 0.36
3	5.44	0.16	0.18	- 0.34
4	7.35	0.13	0.14	- 0.33
5	2.64	0.39	0.38	- 0.34
6	0.66	1.5	1.52	- 0.37
7	0.46	2.2	2.17	- 0.34

Table 2 : Calibration of β and γ

The next step in the calibration is the determination of the co-efficient p_ϵ . From Equation (23°) it can be derived that p_ϵ should meet the following Equation :

$$p_\epsilon = - \frac{z_\epsilon^2 (1 - \epsilon)}{(2 + \beta) T_\epsilon a_\epsilon q_s} \quad (34)$$

Substitution of $n = q_s/s_\epsilon$ and the values as specified for β and a_ϵ from MIKE 11 runs (see Table 1) the following result has been obtained with substitution of $\epsilon = 0.4$, as this was used in MIKE 11 computations :

$$p_\epsilon = - \frac{0.6nh_\epsilon^2}{25s_\epsilon (2 + \beta) T_\epsilon a_\epsilon} \quad (35)$$

The computed values have been presented in Table 1. The conclusion is that p_ϵ is a rather constant value not depending on n . A good approximation is given by :

$$p_\epsilon = 0.6 \quad (36)$$

Note that, some higher deviations from this value seem to occur for small n values. These deviations might be caused by the fact that, the sedimentation length as computed with MIKE 11 has passed the downstream boundary. The boundary conditions are supposed to influence the final results. So, the results of the cases 6 and 7 (see Table 1) should be treated with some care.

The last parameter, that should be fitted is the power α , which is determining for the shape of the sedimentation (see Equation 15). The value of α has been determined by using Equation (17), from which it can be derived that:

$$\alpha = - a_e \cdot \frac{x_s}{z_e} \quad (37)$$

Using the results of a_e and x_s as determined from the MIKE 11 runs (see Table 1) the α value could be calculated for each case. The results for α have also been collected in Table 1. Clearly it can be observed that α depends on n . Figure 9 presents the values of α plotted against n . A good result for α should be:

$$\alpha = + (1 + 0.1n) \quad (38)$$

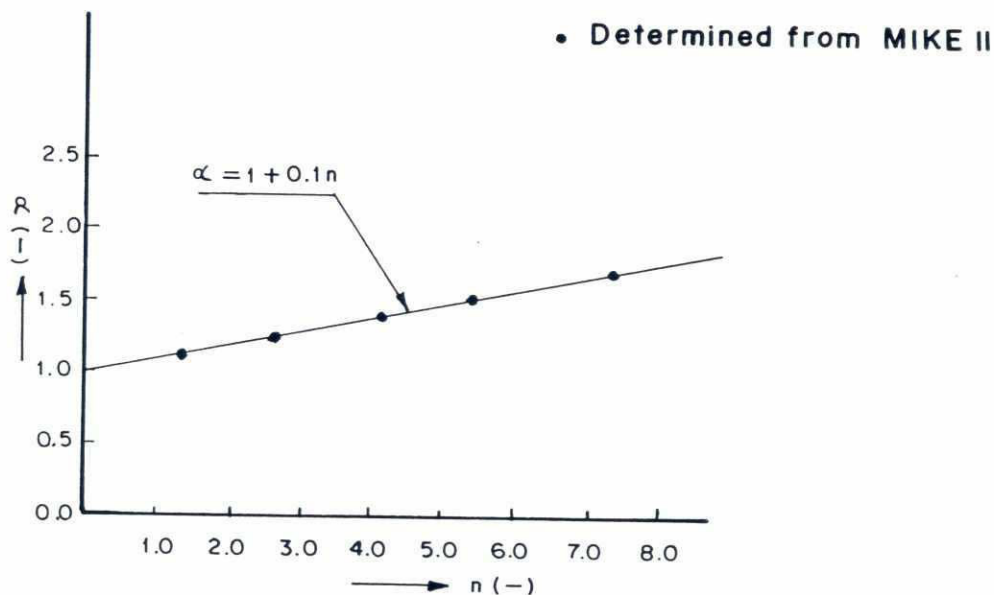


Fig. 9: α versus n

However, the α power seems also to depend on the porosity ε . A first indication has been obtained from one calculation. The calculated case is comparable to the case 4 as given in Table 1 with the only difference that $\varepsilon = 0$. The results of MIKE 11 showed that $z_e = (n/5) h = 4.41$ m has been reached at :

$T_e = 3.3$ year, which totally agrees to Equation (23°)

and

$x_s = 28$ km

From the result for x_s for this specific case it is concluded that the effect of ε should be included in the relation for α as follows :

$$\alpha = \frac{p_\varepsilon}{1 - \varepsilon} (1 + 0.1n) = \frac{0.6}{1 - \varepsilon} (1 + 0.1n) \quad (39)$$

Now all the parameters have been calibrated resulting in the following set of Equations to compute the sedimentation in a river with constant discharge q as a result of a sediment supply q_s :

$$q = C h_e^{\frac{3}{2}} i^{\frac{1}{2}} \quad (m^2 / s) \quad (40)$$

$$s_e = \frac{0.05}{\Delta^2 \sqrt{g} C^3 D_{50}} \cdot \left(\frac{q}{h_e} \right)^5 \quad (m^2 / s) \quad (41)$$

$$q_s = n s_e \quad (m^2 / s) \quad (42)$$

$$z_e = \frac{n}{5} h_e \quad (m) \quad (43)$$

$$T_e = + \frac{1}{5.25} \cdot \frac{h_e^2}{s_e \cdot i} \cdot \frac{(1 - \varepsilon)}{2 + \frac{1}{n}} \quad (s) \quad (44)$$

i = water level slope in equilibrium situation (-)

$$z = z_b(o, t) = z_e \left(\frac{t}{T_e} \right)^{\frac{1}{2 + \frac{1}{n}}} \quad (for\ z < z_e) \quad (m) \quad (45)$$

$$z = z_b(o, t) = z_e \left\{ 1 + \frac{n+1}{2n+1} \left(\frac{t}{T_e} - 1 \right) \right\}^{\frac{1}{1 + \frac{1}{n}}} \quad (for\ z \geq z_e) \quad (m) \quad (46)$$

$$z_b(x, t) = z \left(1 - \frac{x}{x_s} \right)^\alpha \quad (m) \quad (47)$$

$$\alpha = (1 + 0.1n) \cdot \frac{0.6}{1 - \varepsilon} \quad (-) \quad (48)$$

$$x_s = \frac{\alpha}{0.35n \cdot i} \left(\frac{5}{n} \right)^{\frac{1}{n}} \left(\frac{z}{h_e} \right)^{1 + \frac{1}{n}} \cdot h_e \quad (m) \quad (49)$$

The Figures 10 and 11 present the dimensionless results of z/h_e and x_s/h_e as a function of t/T_e .

These figures will be the final key figures, from which a sensitivity analysis can be done easily.

Fig. 12 presents the results of $z_b(0,t)$ from the 7 cases as computed with MIKE 11 compared to the results of the simplified model. The agreement between MIKE 11 results and the simplified model results are evident due to the calibration to these cases.

Therefore a verification has been made for a total different situation (flood condition !). This situation can be described with the following input parameters:

$$q = 34 \text{ m}^2/\text{s}$$

$$h_e = 15 \text{ m}$$

$$C = 70 \text{ m}^{1/2}/\text{s}$$

$$s_e = 5.14 \cdot 10^{-3} \text{ m}^2/\text{s}$$

$$n = q_s/s_e = 2$$

The results for $z_b(0,t)$ have been presented in Figure 13, from which the prediction value of the simplified model is evident. The small differences can possibly be caused by some instabilities in MIKE II due to the sudden step (s_e to $q_s + s_e$) in the sediment transport.

From the simplified model the following for T_e was determined :

$$T_e = 0.906 \text{ year}$$

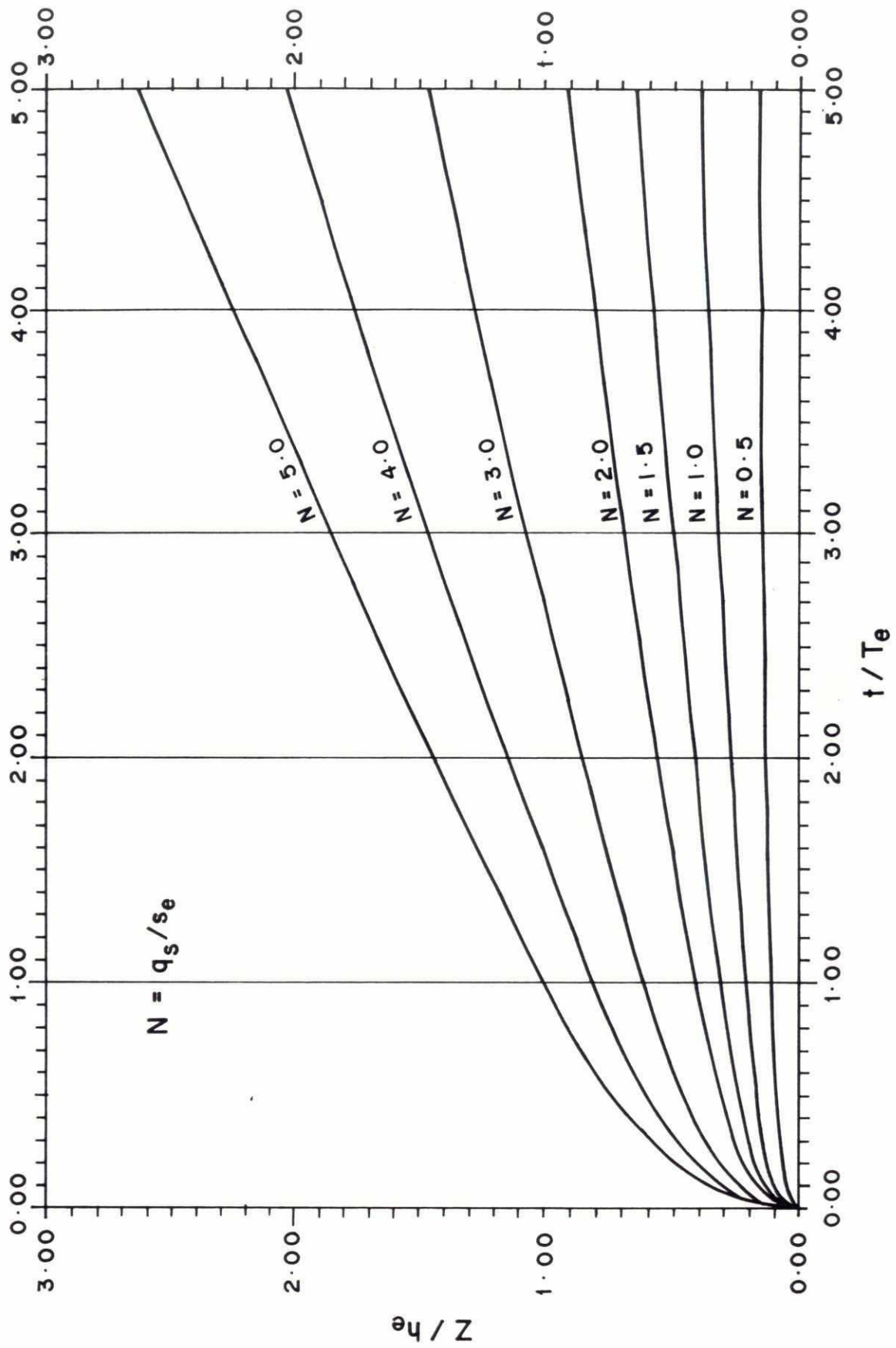
consequently $x_s = 147 \text{ km}$

Which is also in good agreement with the results of MIKE 11. This can be observed from Figure 14 showing the sedimentation after 1 year. This Figure shows a sedimentation length $x_s = 140 \text{ km}$.

The final conclusion is, that the simplified model is a good first step to provide graphs, from which a sensitivity analysis for closing branches of the Jamuna river could be done easily.

Looking forward to the changing water level conditions in the Jamuna river during a year a first trial for the sedimentation process is given as follows.

Assuming that the water level changes are faster than the sedimentation height changes $\{z_b(0,t)\}$, the results of the present simplified model can possibly already serve as a tool.

Fig. 10: z/h_e versus t/T_e

$$\frac{x_s}{h_e} = \frac{1 + 0.1n}{1.75i} \left(\frac{t}{T_e} \right)^{\frac{n+1}{2n+1}} \quad \frac{t}{T_e} < 1$$

$$\frac{x_s}{h_e} = \frac{1 + 0.1n}{1.75i} \left\{ 1 + \frac{n+1}{2n+1} \left(\frac{t}{T_e} - 1 \right) \right\} \quad \frac{t}{T_e} \geq 1$$

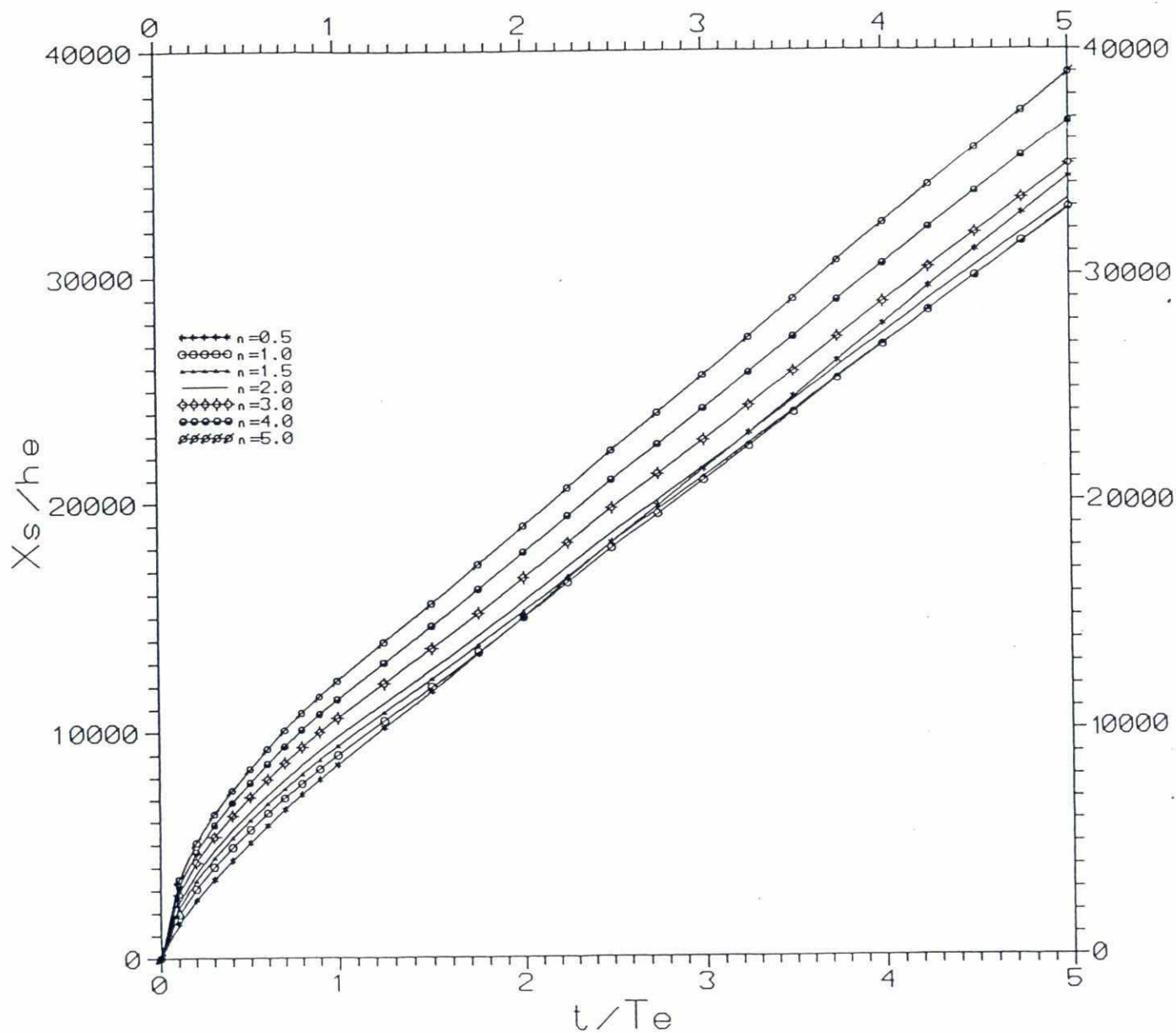
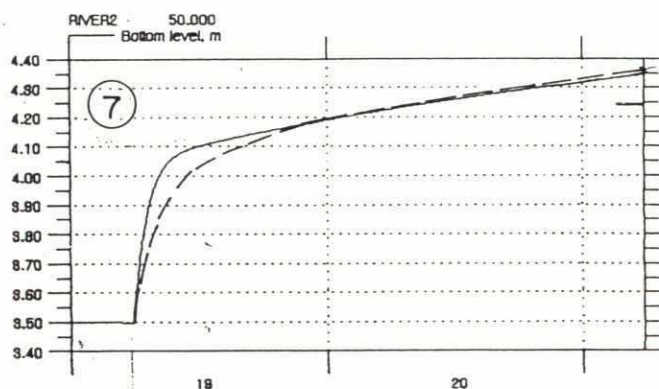
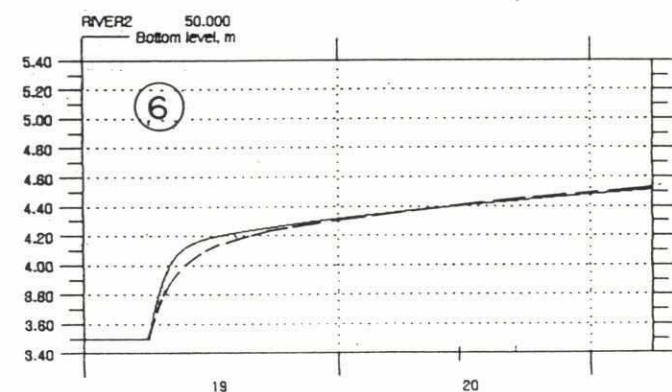
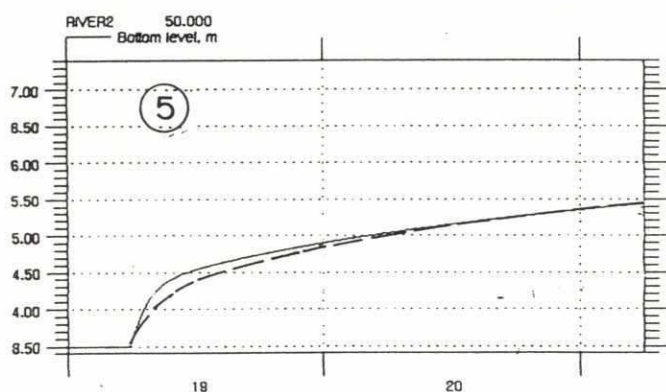
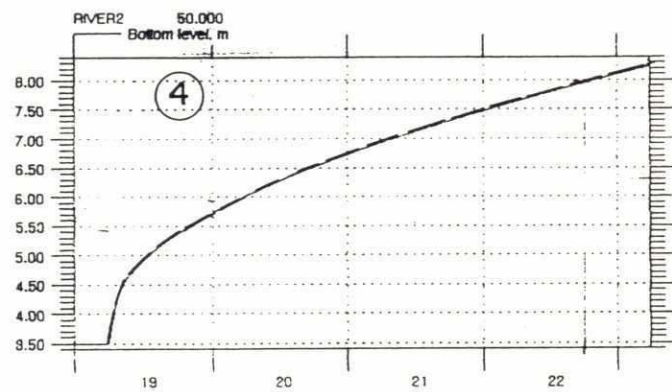
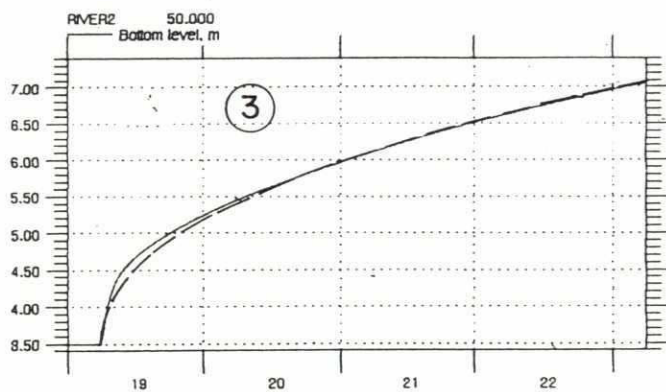
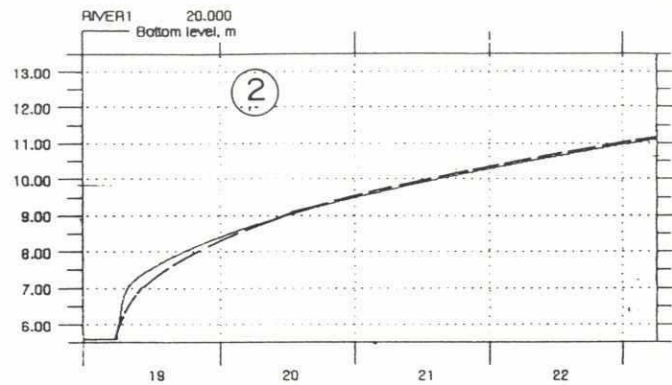
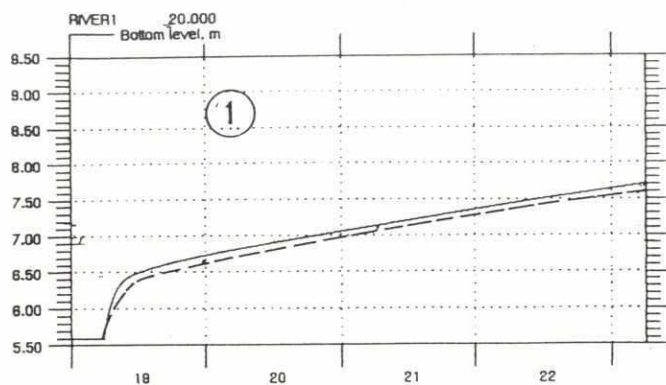


Fig. 11: x_s/h_e versus t/T_e



— Mike II
 - - - Simplified model

Fig. 12: Calibration results for $z_b(o,t)$

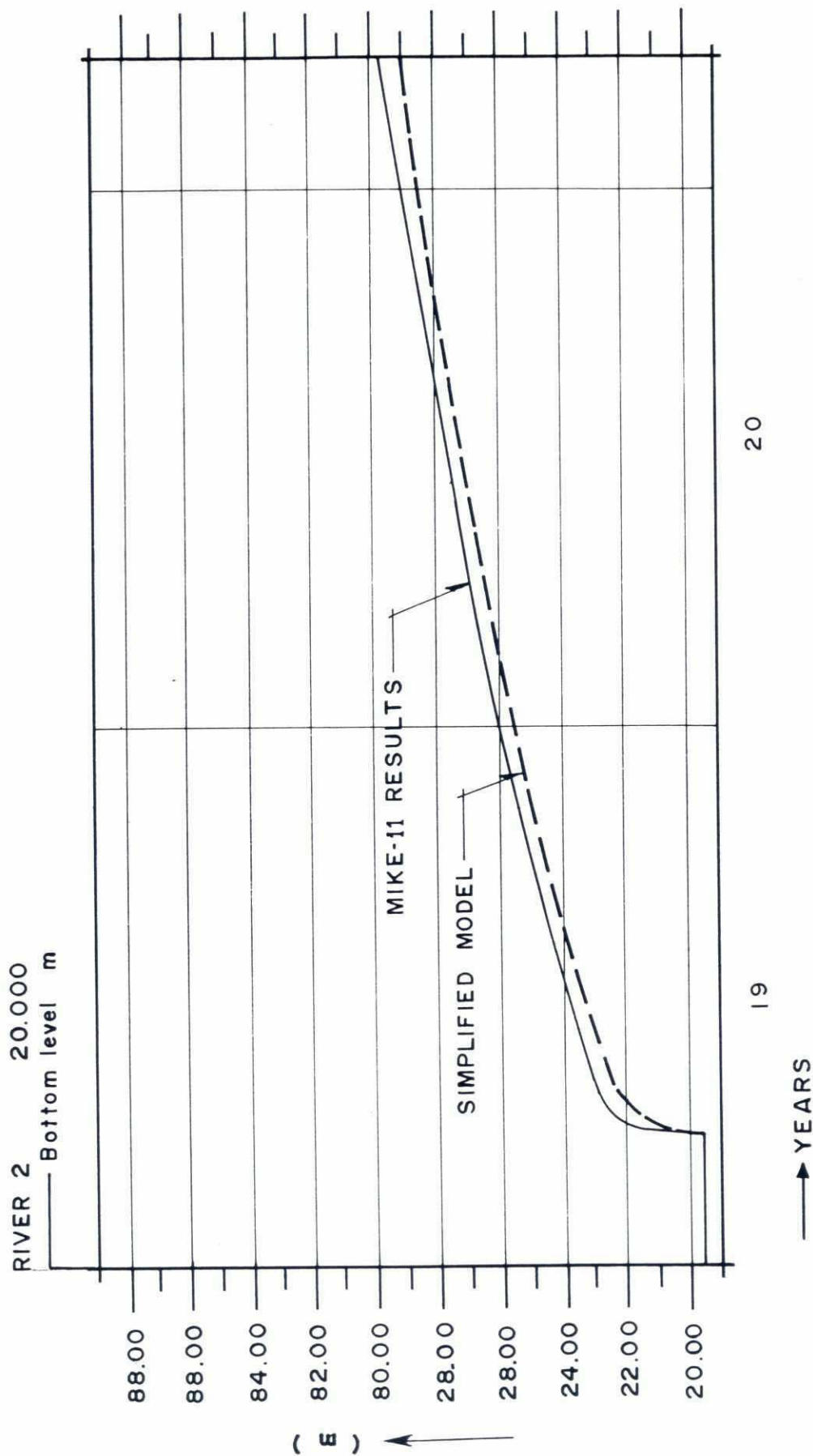


Fig. 13: Verification of simplified model

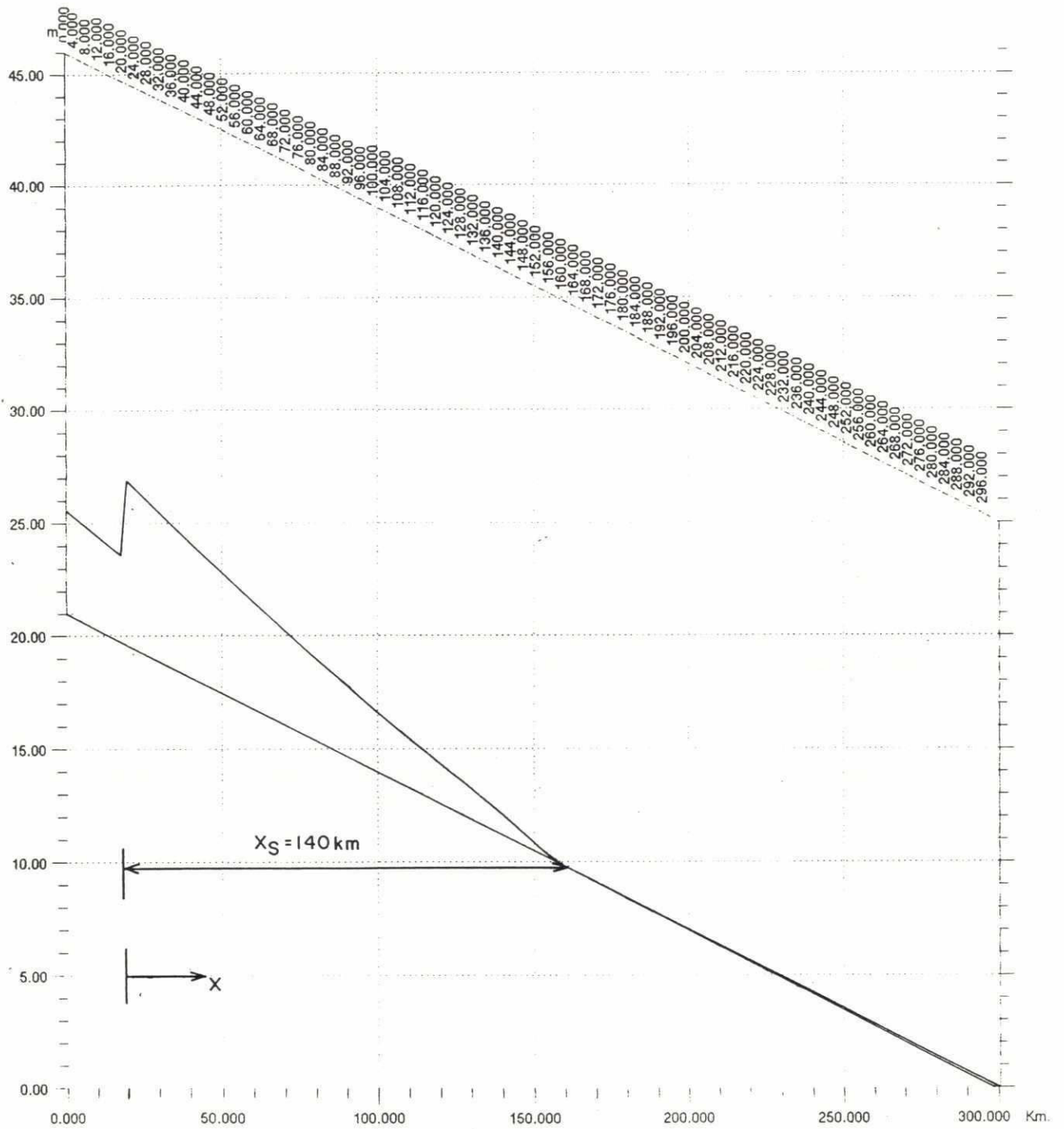


Fig. 14: Sedimentation length for verification case

From Equations (40), (41) & (44) the following result can be derived :

$$T_e = A \cdot \frac{1}{\sqrt{h_e}} \quad (s) \quad (50)$$

$$\text{With } A = \frac{3.18 \Delta^2 \sqrt{g} D_{50} (1 - \varepsilon)}{C^2 i^{3.5} (2 + \frac{1}{n})} \quad (m^{2.5}) \quad (51)$$

According to the Equations (45) and (46) the following results for the sedimentation height can be derived :

$$z_b(o, t) = 0.2 n h_e \left(\frac{\sqrt{h_e}}{A} t \right)^{\frac{1}{2 + \frac{1}{n}}} \quad \text{for } \frac{\sqrt{h_e}}{A} t \leq 1 \quad (52)$$

$$z_b(o, t) = 0.2 n h_e \left\{ 1 + \frac{n+1}{2n+1} \left(\frac{\sqrt{h_e}}{A} t - 1 \right) \right\}^{\frac{1}{1 + \frac{1}{n}}} \quad \text{for } \frac{\sqrt{h_e}}{A} t > 1 \quad (53)$$

Note, that Equations (52) and (53) only can hold under the condition that the effectivity of measures (bandals, vanes, etc.) is given by a constant n value. This actually indicates that the effectivity of a measure is directly related to the sediment load as transported by the river, which might be supposed but verified !

When h_e is supposed to be time dependent according to the hydrographs from the Jamuna river, expressions (52) and (53) might give a first insight in the sedimentation process. The validation of Equations (52) and (53) will be done at a later stage of the project.

Note that, with expressions (50) and (51) an easy calculation of T_e is possible for Jamuna conditions ($C \approx 60 m^{1/2}/s$, $i \approx 7 \cdot 10^{-5}$, $D_{50} = 2 mm$):

$$T_e \approx \frac{12n}{2n+1} \cdot \frac{1}{\sqrt{h_e}} \quad (years) \quad (54)$$

4. WATER LEVEL CHANGES AND UPSTREAM BOUNDARIES

As described in Section 2 the bottom level changes (see Equation 10) is a rather independent process and mainly based on the conservation of mass consideration for the sediment. This especially holds for the present situation of a constant discharge during the sedimentation. It might be supposed that the influence of a rather stable water level at bifurcation points might influence the process as described in Section 2.

However, at this moment it should be important to understand the relation between bottom level changes and water level changes in order to assume a varying (decreasing) discharge in the river branches to be closed. As a first step to assess the influence of a varying (decreasing) discharge reference is made to Fig. 7. Supposing that the specific morphological time scale T_e is directly related to the discharge $q(t)$, it might be concluded that T_e will decrease with decreasing discharge $q(t)$ resulting in a faster process for the sedimentation height $z_b(0,t)$. These considerations will be checked and verified at a later stage of the project.

From the basic Equation (6) it might be concluded that the gradient of the water level can be calculated according to :

$$\frac{\delta (h + z_b)}{\delta x} \approx c_1 \quad (55)$$

in which c_1 is a friction term depending on the slope i of the river and local water depth h as follows :

$$c_1 = i \left\{ 1 - \left(\frac{h_e}{h(x,t)} \right)^3 \right\} \quad (56)$$

Assuming a mean value of c_1 between $x = 0$ and $x = x_s$ (length of sedimentation), the result of Equation (55) can be approximated by :

$$h + z_b \approx \bar{c}_1 x + K \quad (57)$$

with the boundary conditions on $x = x_s$ (see Fig. 3), that :

$$h = h_e \text{ and } z_b = 0$$

and thus :

$$h_e \approx \bar{c}_1 x_s + K \quad (58)$$

resulting in :

$$K = h_e - \bar{c}_1 x_s \quad (59)$$

Substitution of this result in Equation (57) leads to :

$$\Delta h(x,t) = h(x,t) + z_b(x,t) - h_e = \bar{c}_1 (x - x_s) \quad (60)$$

With the result for x_s as given with Equation (17) this results in :

$$\Delta h(x,t) = \bar{c}_1 \left(x + \frac{\alpha}{a} z \right) \quad (61)$$

Which results on $x = 0$ in :

$$\Delta h (0, t) = \alpha \cdot \frac{\bar{c}_1}{a} \cdot z_b (0, t) \quad (62)$$

The description of the parameters has already been described extensively in Section 3. With the derivation of Equation (62) it is only indicated that a direct relation between $\Delta h(0, t)$ and $z_b(0, t)$ holds. Knowing this, the following consideration has been made.

When the sedimentation process starts, the water level is supposed to be stable until sufficient water depth $h(0, t)$ is present to transport the total sediment load $s_e + q_s$. From that moment the water level is supposed to rise according to the bottom level rise, resulting in :

$$\Delta h (0, t) = \Delta h_o = z + h_o - h_e = z - z_a \quad (63)$$

in which h_o is computed from the sediment transport Equation (3) in such a way that the following result for h_o should hold:

$$s_e + q_s = a_s \left(\frac{q}{h_o} \right)^5 \quad (64)$$

or, with $q_s = ns_e$

$$(n + 1) s_e = a_s \left(\frac{q}{h_e} \right)^5 \left(\frac{h_e}{h_o} \right)^5 \quad (65)$$

$$\text{or } (n + 1) s_e = s_e \left(\frac{h_e}{h_o} \right)^5 \quad (66)$$

from which finally the following result has been obtained:

$$\frac{h_o}{h_e} = \frac{1}{(n + 1)^{0.2}} \quad (67)$$

$$z_a = h_e - h_o = \left(1 - \frac{1}{(n + 1)^{0.2}} \right) h_e \quad (68)$$

The time T_a at which z_a will occur can consequently be determined from Equation (45), according to :

$$\frac{z_a}{z_e} = \left(\frac{T_a}{T_e} \right)^{\frac{1}{2 + \frac{1}{n}}} \quad (69)$$

The result for Δh_o should now be :

$$\Delta h_o = z - z_a \quad \text{for } t > T_a \quad (70)$$

$$\Delta h_o \approx 0 \quad \text{for } t \leq T_a \quad (71)$$

The verification of Equation (70) has been made by collecting Δh_o values and $z_b(o,t)$ from the MIKE 11 runs for $t > T_a$.

The results have been presented in Table 3. Also the values of z_a as specified with Equation (68) have been taken up in table 3. The computed values of Δh_o according to Equation (70) show very good agreement with the observed values from the MIKE 11 runs. This result justifies the process as described in this Section and especially the Δh_o Equation as presented with Equations (70) and (71). As a final remark reference is made to Figure 15 presenting a MIKE 11 result for $\Delta h(x,t)$ as being done for case 4 (see also Table 1).

INPUT		MIKE-11 RESULT		CALIBRATION	
Case	n	Δh_o *)	z (m) *)	z_a (m)	Δh_o (m)
1	1.37	0.8	1.6	0.81	0.79
2	4.11	2.8	4.3	1.42	2.88
3	5.44	1.74	2.6	0.93	1.67
4	7.35	2.45	3.50	1.04	2.46
5	2.64	1.16	2.00	0.91	1.09
6	0.66	0.42	1.02	0.67	0.35**
7	0.46	0.47	0.85	0.58	0.27**

*) Data collected for $t = 2$ years

**) Sedimentation length x_s passed downstream boundary.

Table 3 : Calibration on water levels

As a final conclusion it can be assumed, that the effects of the distance between the upstream boundary and sediment input location are neglectible regarding the sedimentation process and water level elevations. This is clearly shown in the Figures 16 - 19 presenting the MIKE 11 results of case 1 (see Table 1) on water levels and bottom levels for two situations:

1. The sediment input takes place at a distance of 50 km downstream of the upper boundary.
2. The sediment inputs takes place at a distance of 20 km downstream of the upper boundary.

Comparing Fig. 16 with Fig. 17 for water levels and Fig. 18 with Fig. 19 for bottom levels, the following conclusion might be drawn :

The water level changes and shape of the sedimentation downstream the location of sediment input q_s are not depending on the location of the upstream boundary. Note that the upstream boundary condition in MIKE 11 should meet the condition, that the sediment transport equals the equilibrium sediment discharge s_e as carried by the river. This condition leads also to the more pronounced water level rises at the upstream boundary in the second situation.

This conclusion might be very helpful to describe the sedimentation process, when a sediment input takes place at a bifurcation point.

From the conclusion that the sedimentation is not depending on the location of the upstream boundary, it seems to be justified to suppose that, the upstream slope of the sedimentation is according to the natural angle of repose (ca. 1:3).

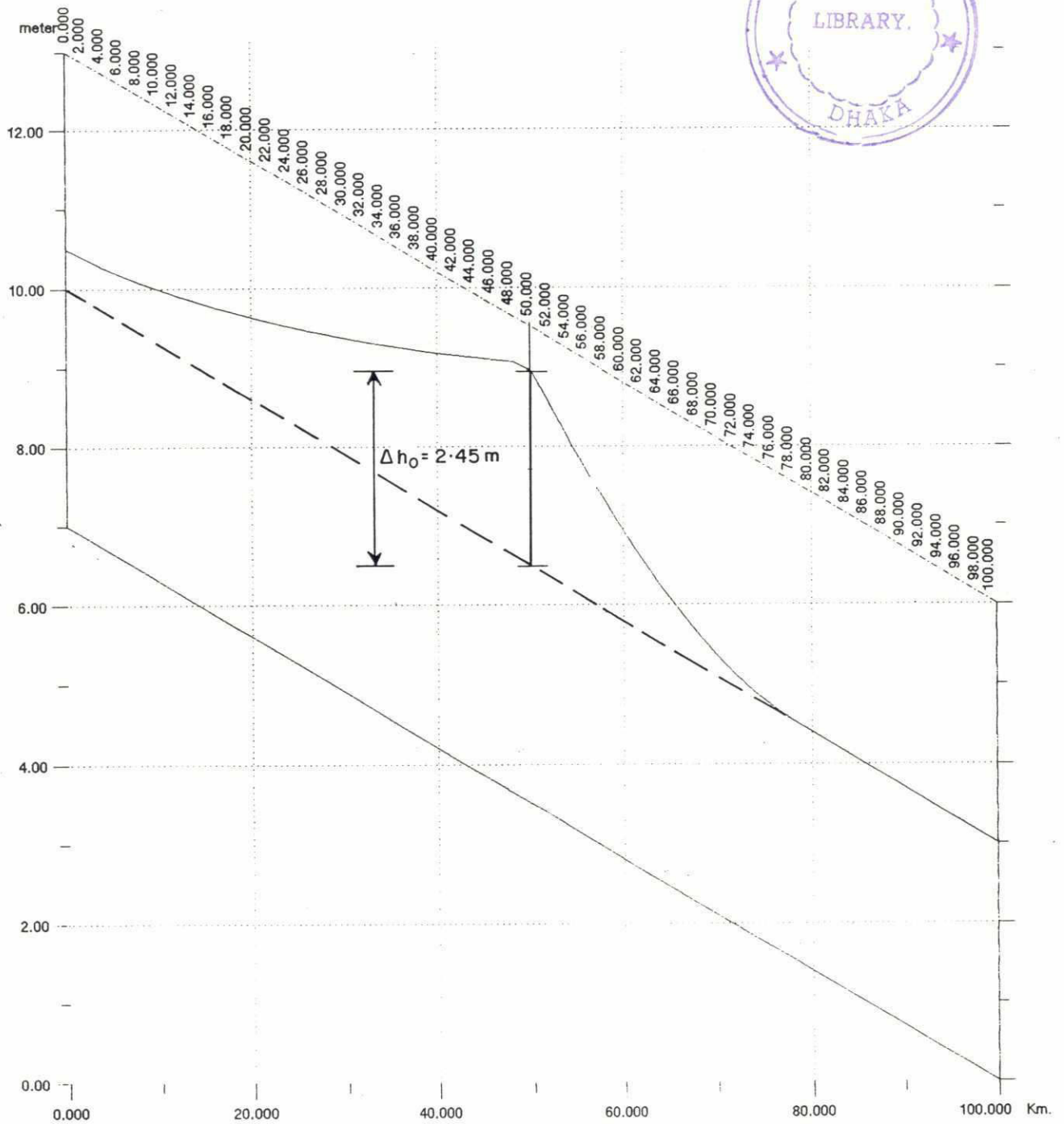
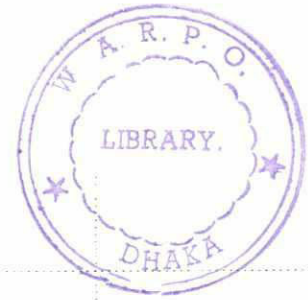


Fig. 15: Water level computation with MIKE-11

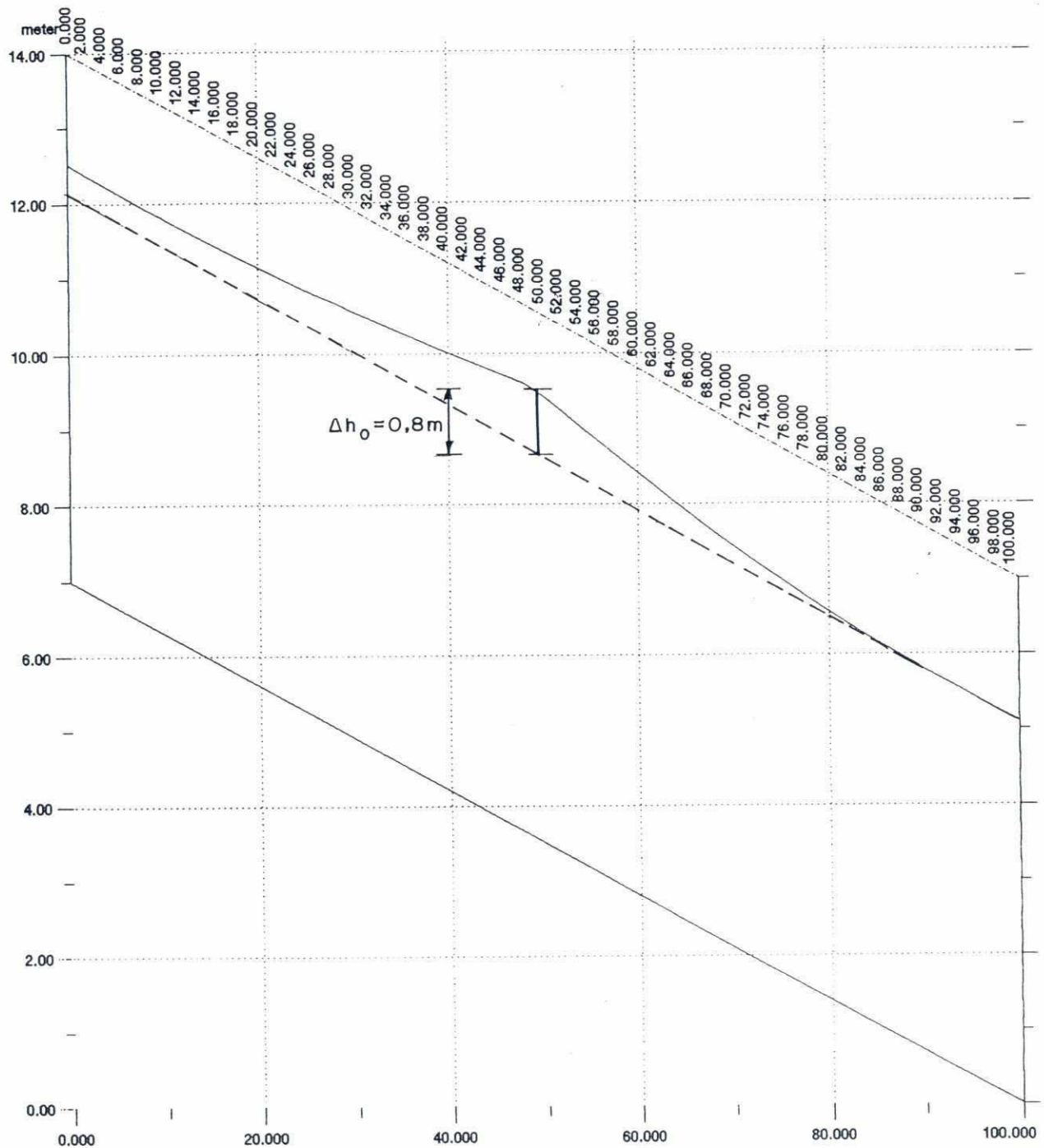


Fig. 16: Water levels due to sediment input on 50 km from upper boundary (Case 1)

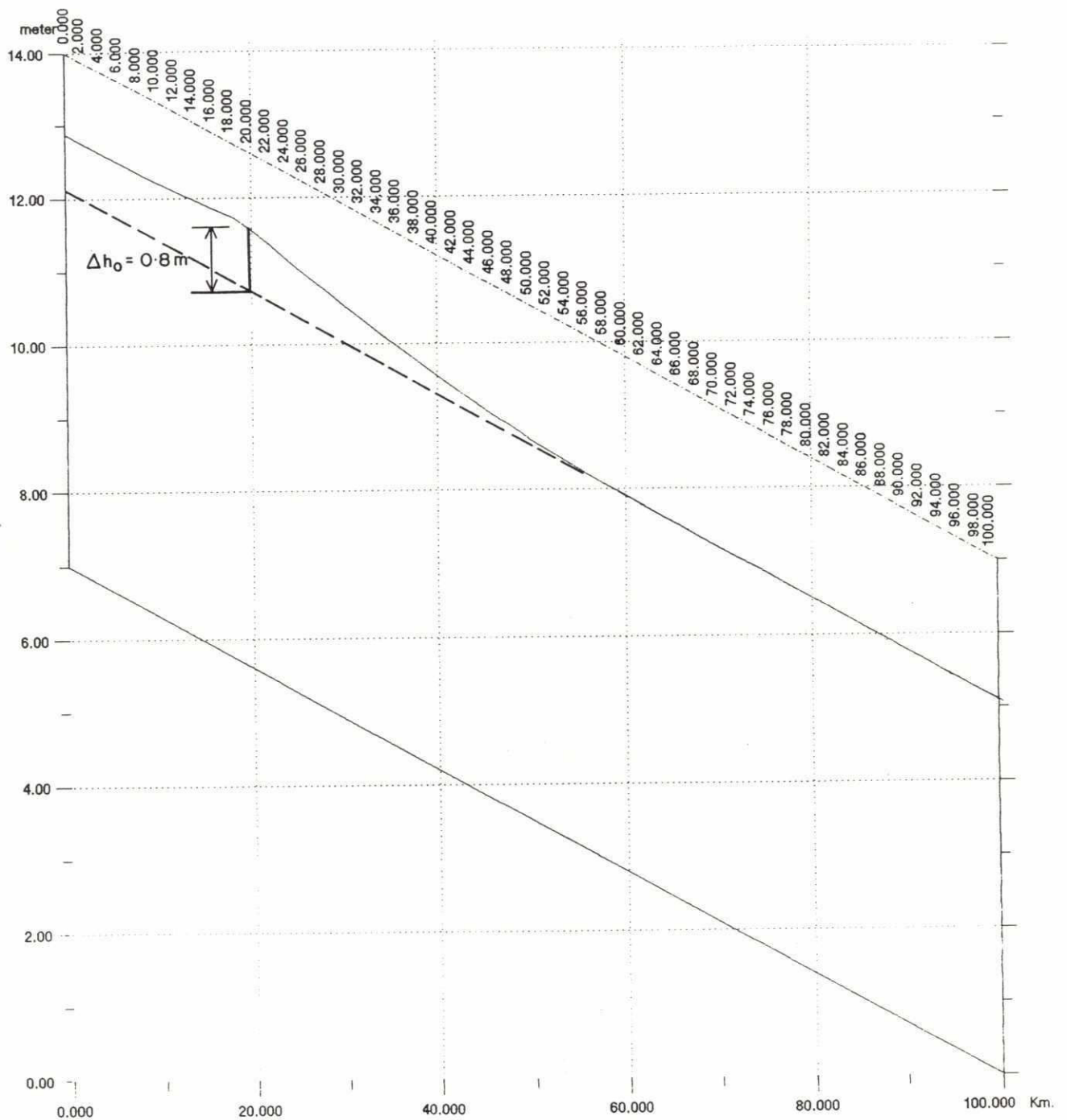


Fig. 17: Water levels due to sediment input on 20 km from upper boundary (Case 1)

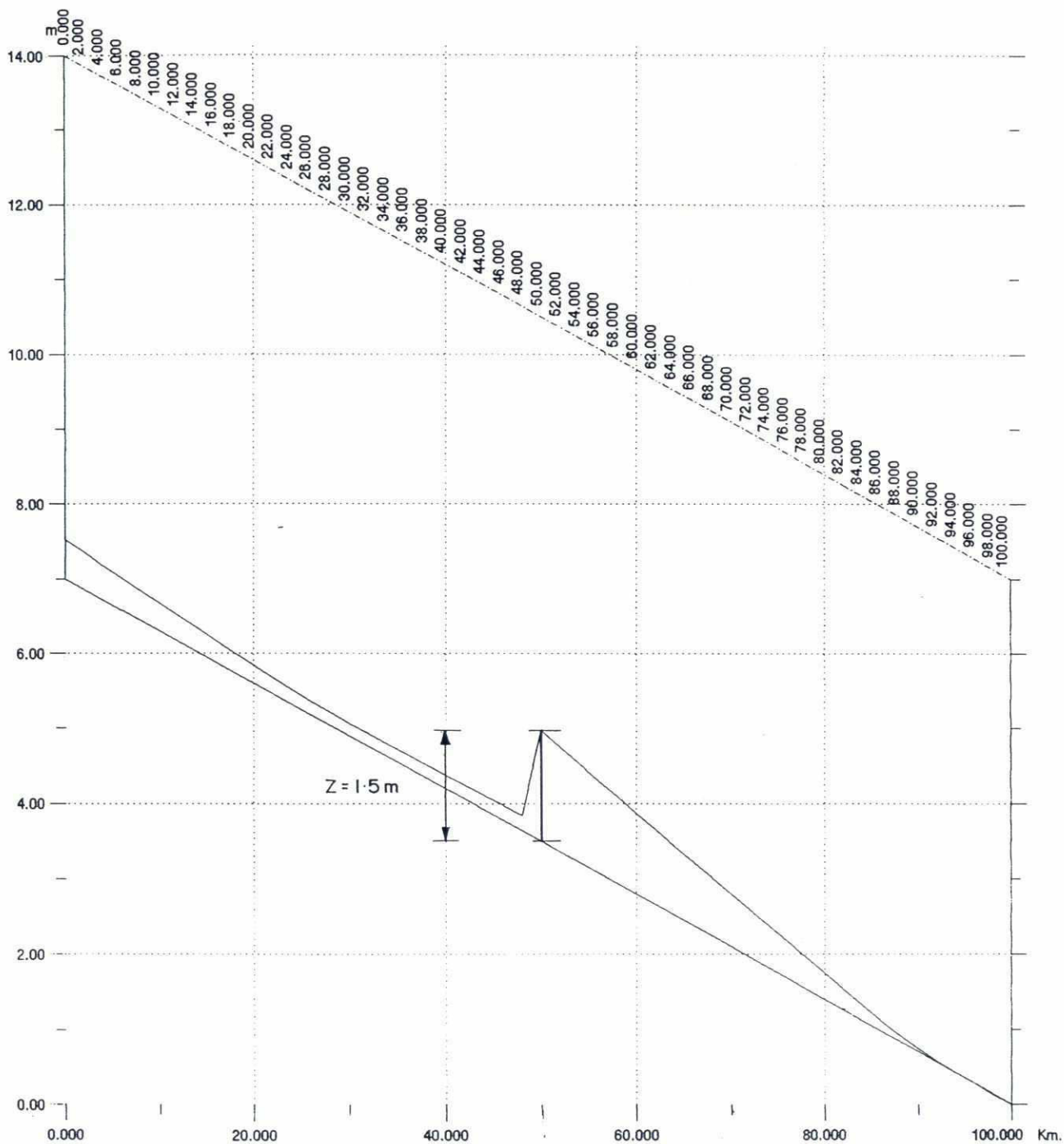


Fig. 18: Sedimentation due to sediment input on 50 km from upper boundary (Case 1)

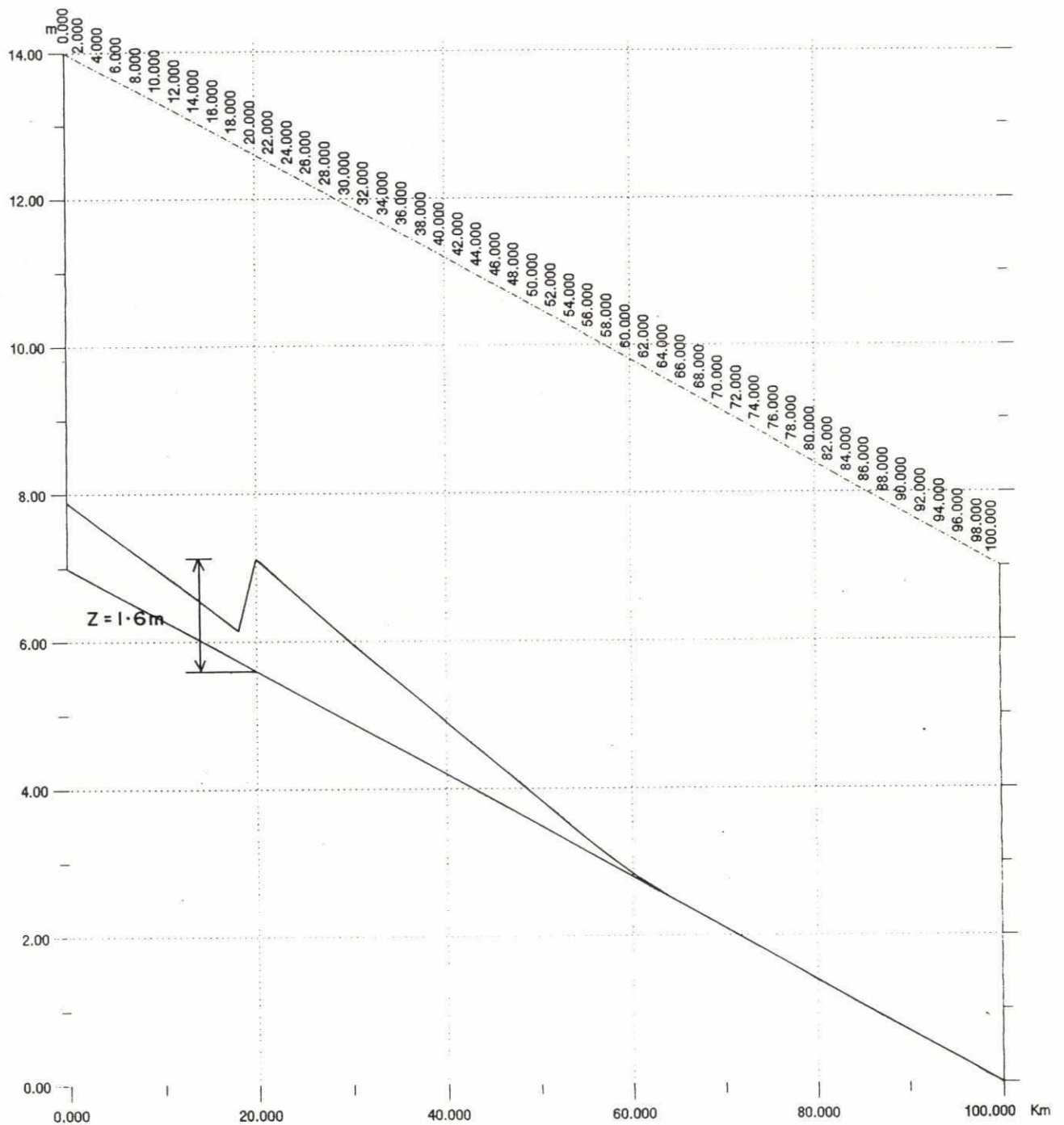


Fig. 19: Sedimentation due to sediment input on 20 km from upper boundary (Case 1)

5. SOME FUTURE DEVELOPMENTS

At this moment of the development of the simplified model, the starting position is the sediment process in a river with constant discharge. At bifurcation points this will not be an actual situation. Therefore the influence of a varying discharge (actually decreasing in the branch to be closed) has to be considered.

Looking forward to solve this problem the following will be considered :

1. the sedimentation process due to a sediment input q_s in a branch to be closed can be described with the Equations as presented for a constant discharge. This process can only hold for the time $t < T_a$ (see Eq. 69)
2. after this time $t = T_a$ the water level on $x = 0$, (location of sediment input) leads to the following condition on $x = 0$, related to the continuity requirement:

$$\frac{dq}{dt} = (h_e - z) \frac{du}{dt} - u \frac{dz}{dt} \quad (72)$$

Neglecting the acceleration term and assuming that, u remains constant after $t = T_a$ and that on $t = T_a$ the local water depth is equal to h_o (see Eq. 67), the following result has been obtained :

$$\frac{dq}{dt} = - \frac{q_e}{h_o} \cdot \frac{dz}{dt} = - \frac{q_e}{h_e} (n + 1)^{0.2} \cdot \frac{dz}{dt} \quad (t = T_a) \quad (73)$$

3. The last assumption will be that dq/dt remains constant right from the beginning of the process (indicated by a first calculation with MIKE 11 on a bifurcation point)

These conditions enable as a first step to describe the sedimentation in a branch to be closed.

The used symbols are :

q_e = initial equilibrium discharge	(m ³ /s)
h_e = initial equilibrium water depth	(m)
z = bottom level on $x = 0$	(m)
u = water velocity on $x = 0$	(m/s)
$n = q_s/q_e$ = ratio between sediment input and initial bottom transport	(-)

To solve this problem the important time T_a has to be determined, which has been done with Equation (43), (68) and (69) resulting in :

$$\frac{T_a}{T_e} = \left\{ \frac{5}{n} \left(1 - \frac{1}{(n+1)^{0.2}} \right) \right\}^{2 + \frac{1}{n}} \quad (74)$$

To have some impression in the ratio $m = T_a/T_e$ reference is made to Fig. 20 presenting $m = f(n)$ according to :

$$m = \left\{ \frac{5}{n} \left(1 - \frac{1}{(n+1)^{0.2}} \right) \right\}^{2 + \frac{1}{n}} \quad (75)$$

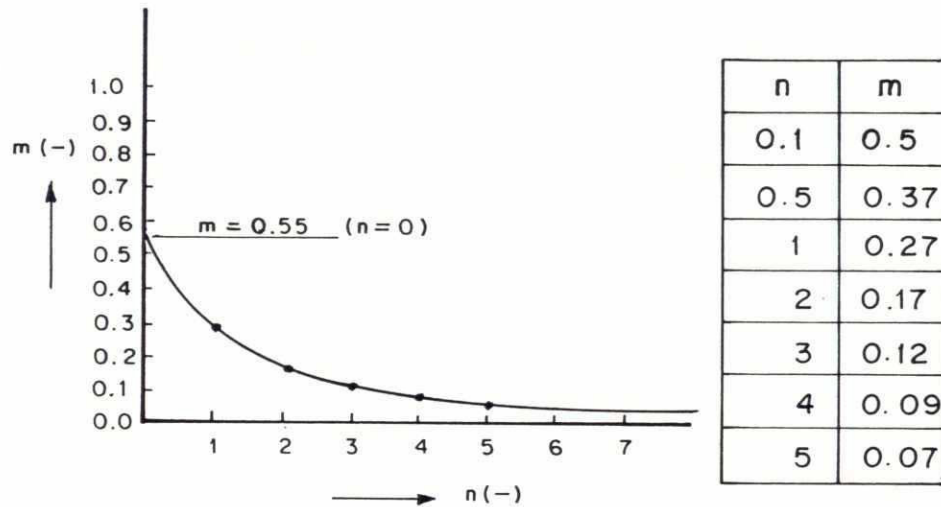


Fig. 20: m versus n

From Equation (45) it is possible to determine the gradient dz/dt on $t = T_a$ resulting in :

$$\frac{dz}{dt} = \frac{z_e}{2 + \frac{1}{n}} \left(\frac{T_a}{T_e} \right)^{-\frac{n+1}{2n+1}} \cdot \frac{1}{T_e} \quad (76)$$

or

$$\frac{dz}{dt} = \frac{n^2}{5(2n+1)} \cdot m^{-\frac{n+1}{2n+1}} \cdot \frac{h_e}{T_e} = \dot{z}_a \quad \text{at } (t = T_a) \quad (77)$$

According to the third consideration the bottom level at $x = 0$ should rise linearly according to :

$$z = \dot{z}_a (t - T_a) + z_a \quad (78)$$

While the discharge in the branch to be closed should decrease according to (see Eq. 73):

$$q = q_e - \frac{q_e}{h_e} (n+1)^{0.2} \cdot \dot{z}_a \cdot t \quad (79)$$

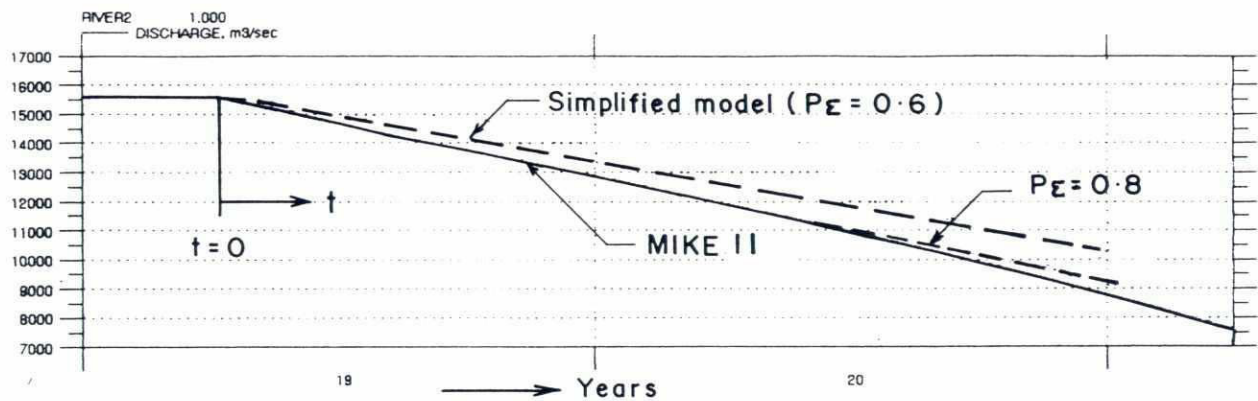


Fig. 21: Discharge as a function of time

The results of Equations (78) and (79) should be verified. A first impression, however, can already be presented from a MIKE 11 run. This run considered a situation of a bifurcation point in such a way that, a certain amount of sediment ($q_s = \Delta s/B$) is subtracted from channel 1 just downstream the bifurcation. This q_s has been added to the channel to be closed. The two channels had dimensions in such a way that, the situation in the channel with the sediment input q_s is totally comparable to case 1 as presented in Table 1. The most important parameters for the channel to be closed, are :

$$s_e = 2.7 \cdot 10^{-4} \text{ m}^2/\text{s}$$

$$q_e = 5.8 \text{ m}^2/\text{s}$$

$$h_e = 5.1 \text{ m}$$

$$C = 60 \text{ m}^{1/2}/\text{s}$$

$$i = 7 \cdot 10^{-5}$$

$$n = q_s/s_e = 1.37 \text{ and thus : } m = 0.224$$

According to Equation (44) the following result for T_e has been obtained :

$$T_e = 1.82 \text{ year}$$

Using Equation (77) the following result for $\dot{z}_a$ has been obtained:

$$\dot{z}_a = 0.26 \text{ h}_e/T_e = 0.73 \text{ m/year}$$

Substitution of this result in Equation (73) leads to :

$$\frac{dq}{dt} = -1 \text{ m}^2 / \text{s} / \text{year}$$

Knowing that the actual width of the branch is $B = 2700 \text{ m}$ and $Q_e = 15625 \text{ m}^3/\text{s}$, the process for $Q(t)$ is as follows :

$$Q = 15625 - 2700 T \quad (m^3 / s) \quad (80)$$

with t in years.

The result of Equation (80) has been compared to the result of the MIKE 11 calculation as presented in Figure 20. As a first conclusion the result seems to be in good agreement with the process described. The predicted value of dq/dt seems to be too low according to the assumptions. Three reasons can be given for this smaller gradient :

1. The MIKE 11 situation represents a situation with a subtraction of q_s in the other branch, which might affect a bigger decrease of q in the branch to be observed.
2. In the considerations for describing the sedimentation process with decreasing $q(t)$ it has been assumed, that the process equals the situation with constant discharge q_e between $t = 0$ and $t = T_a$. This time should be probably smaller than assumed. When for the considered case a time $t = 0.7 T_a$ was assumed, the decrease of $q(t)$ is exactly the same as computed with MIKE 11.
3. A third reason can be the reduction of the damping R (see Section 2, Equation 24), which might result in an increased p_e co-efficient and consequently in a reduction of T_e (see Eq. 23°). When $p_e = 0.8$ (instead of 0.6 for constant discharge), the result of $q(t)$ is comparable to the MIKE 11 result (see Figure 21).

In the next phase of the project the process describing the situation with varying discharges will be verified and, if necessary, calibrated.

Another step in the future developments is a further elaboration of the considerations for describing the sedimentation under varying water levels (hydrographs!) as indicated at the end of Section 3. However, when a water level is falling and a sedimentation height is already present the effect of flow diversion to the other branch (or part) of the Jamuna river as has been indicated in this Section should be included.

Finally, the whole procedure, as presented in this report to describe the sedimentation as result of a sediment input q_s due to the measures on the bifurcations, should be repeated for the consequences of subtracting a water discharge Δq from the branches to be closed. However, it is expected that this process is very similar to the process of sedimentation input.

Depending on time, a combination of sediment input and water subtraction will be treated aiming at achieving an impression, whether (low cost) alternative measures (as bandals, vanes, etc.) are able to change the character of the Jamuna river.

As a final conclusion it is noticed that the first result of the simplified methodology to tackle the physical processes of the Jamuna river are very encouraging!

ANNEX 6

FILTER TEST RIG

ANNEX 6**FILTER TEST RIG****Table of Contents**

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1 INTRODUCTION

Within the framework of the Bank Protection and River Training (AFPM) Pilot project, FAP 21/22, which is a component of the Bangladesh Flood Action Plan, a test rig will be constructed as an universal test facility to study the stability of filter and toplayers constructions.

In general a bank protection consists of a tippler on a filter, which is placed on a subsoil of erodible material. A filter can be made of granular layers of different materials and/or of a geotextile. Nowadays many different types of geotextiles have been developed and these can be used as an alternative for a granular filter.

An adequately designed filter should fulfill several conditions, for example:

- A filter should be almost impermeable for the penetration of material of one layer into another layer.
- A filter should have sufficient permeability for groundwater flow to prevent too high lift forces on an almost impermeable top layer.
- The granular layers of a filter should have a minimum thickness and this granular material should fulfill the conditions for internal stability, to guarantee the layer will function at every place of the bank protection.

Whether a proposed filter fulfills these conditions, can be tested in a specially designed test rig.

2 FUNCTIONAL DESIGN OF THE TEST RIG

A functional design of a test rig in the hydraulic laboratory of BUET has been made. For the selection of this location the following criteria have been considered:

The test rig will be placed in a hall, in which an area of 10 m * 4 m with a free height of 3.5 m is available. Preferably this hall has a strong concrete floor, to save the construction of a special foundation for the test rig.

At a short distance from the test rig some supporting facilities, as some office rooms and a simple mechanical workshop, will facilitate the operation of the test rig much.

In the hall an electricity connection is available to run a pump, for light and electric tools. Of cause normal water supply is available.

It can be an advantage if a truck can park near the test rig, to unload building materials and equipment. Near the selected location only a small car can park, however this is not a severe limitation. The selected location fulfills all other conditions.

A test rig is composed of the following main elements:

- test section with connections for piezometric tubes,
- outflow section with a weir to measure the discharge, and with a facility to collect sediment,
- inflow section with an overflow gate,
- water reservoir,
- pump and connecting pipes.

Contrary to the preliminary design of the test rig, this functional design the test rig in BUET does not have a closed water circulation. That means the test rig is connected to the existing underground water reservoir, which is shared with the existing test facilities in the hydraulic laboratory.

Some details of this functional design are elaborated in the following:

Test section

The width of the test section is 1.1 m, because of:

- the minimum required working space to build in a filter with tippler in the test section,
- the required reduction of the disturbing influence by the boundaries of the build in filter with tippler, for example leakage along the glass wall or the unequal settlement along the glass wall.

The height of the test section is about 1.5 m. Some examples of built in test constructions can be found in figure 5.

The minimum length of the test section is 1 m, selected is 2 m. The test section will be made of steel plates reinforced by a steel frame of bars.

The inflow and outflow side of the test section can be made of a removable steel frame with a permeable gauze or a geotextile to allow the groundwater flow and to prevent transport of particles from the subsoil, filter layers and tippler.

In the back wall and in the bottom of the test section a big number of connections for the plastic pivot tubes are designed. The free water level in plastic tubes will be read to determine the pressures in the subsoil or in the filterlayers. In general about 30 tubes will be sufficient for one test. At the back side of the outflow section a wooden board is designed to which these tubes can be attached with an interval of about 0.05 m.

The maximum discharge Q through the test section is based on a maximum flow velocity of 0.20 m/s, a width of the test section = 1.1 m and a water depth = 1 m:

$$Q_{\max} = 0.20 \cdot 0.4 \cdot 1.1 \cdot 1 = 0.088 \text{ m}^3/\text{s}$$

In this estimation a porosity of 0.4 is assumed. The pump should have a capacity of 0.100 m³/s.

Outflow section

A water level range of about 1.0 m is selected with a maximum water level between 1.7 and 2 m and a minimum water level at 0.7 to 1.0 m above the bottom of the test section. The water level can be varied by an overflow weir.

The minimum water depth of 0.7 m in the test section and the outflow section should be emptied by gravity in case of a power failure. Therefore an outflow pipe with a valve will be placed in a side wall of the outflow section and also in the inflow section. The diameter of the steel or plastic pipe should be about 0.1 m.

The outflow section should be made of steel plates reinforced by a frame of steel bars. This structure is placed on a 0.4 m high foundation of brick work, which is built on the existing concrete floor. Upstream of the weir some brick work is designed to facilitate entering the outflow section. Therefore also a steel step is designed at the inside of the outflow section.

The discharge can be measured with a standardized V-shaped weir, which is part of the gate. The maximum discharge measured by the weir is 0.090 m³/s.

During the preparation of a test the outflow section will be filled with water through a perforated filling pipe with a diameter of about 0.1 m. In the outflow section this steel or plastic pipe needs one or two supports.

During a test the sediment, which is transported from the subsoil or filter layers in the test section, can be observed through a window and the transported sediment can be collected in a simple sediment trap. Generally only small amounts of sediment will be trapped and the sediment can be collected by syphoning. Therefore a connection will be made through the front wall. The sediment will be collected in a basket. A more sophisticated method is to use an electric suction pump to guarantee that the suction velocity is equal to the flow velocity.

It is mentioned that only in some special test types this sediment transport has to be measured. Therefore this sediment trap can be considered as an optional facility, that can be built in later if required.

A moveable bridge or ladder, which crosses the test section and / or the outflow section, will be useful for taking photographs during a test.

Inflow section

In the inflow section the pressure of the water, which enters the test section, depends on the water level in the inflow section. This water level is regulated by a free overflow gate. The vertical gate has a range of about 1 m at least and the width of the moveable overflow section can be reduced from the total width of 1.1 m to 0.6 m. If a maximum

overflow discharge of $0.070 \text{ m}^3/\text{s}$ is assumed the height of the water level above the crest is about 0.15 to 0.20 m. The aeration of the overflow will be no problem.

The length is 1 m and the width is the same as the width of the test section, 1.1 m. The height of the water level is maximum 2 m above bottom of the test section. This allows maximum gradients in the test section of about 1.

The inflow section will be made of steel plates reinforced by a steel frame of bars. This steel structure is placed on the existing concrete floor. The details of the gate have to be determined. Some leakage can be accepted.

The separation between the inflow section and the test section can be made by a gauze, geotextile or impermeable parts, depending on the test. These elements are placed in a frame, which can be removed from the filter box.

The filling pipe from the pump enters the inflow section through a back wall. The end of this steel or plastic pipe with an diameter of about 0.1 m, is perforated to limit the turbulence in the inflow section. The excess pump discharge returns as a free overall discharge back to the reservoir. To allow some fluctuations in the pump discharge the free water level area should not be too small: $1 * 1.1 \text{ m}^2$. The water level in the inflow section is read from the scale, which is behind or attached to a separate, vertical glass tube, in which these turbulent fluctuations will be damped strongly. A ladder of the indicated steel steps is required to make the readings, if the water level is high. As an additional facility a lamp should light this scale.

Reservoir

The reservoir should have an effective storage capacity equal to the volume of water in the inflow section, test section and the outflow section: $4 * 1.1 * 1.1 = 4.9 \text{ m}^3$.

A condition is that all the water in the test section can be drained by gravity, that means the reservoir should be below the test section.

This reservoir has two connections with the outflow section (e.g. the overflow from the weir, and the drain pipe in the sediment trap) and also two connections with inflow section; (e.g. the excess pump discharge returns via the free overall gate to the reservoir, and a drain pipe). And through the suction pipe of the pump water is pumped from the reservoir.

The maximum level is 1.5 m above the bottom of the reservoir and the minimum level is estimated at 0.75 m above the bottom level, since no separate stump has been designed for the suction pipe of the pump to prevent air suction. This means that the effective height of the reservoir is 0.75 m and the effective storage capacity is 14 to 15 m^3 , which is much more than the minimum required storage capacity of around 5 m^3 .

The end of the suction pipe has a special shape to prevent the suction of air.

Pump

The maximum discharge of the pump is $0.100 \text{ m}^3/\text{s}$ and the head difference is only 3 to 4 m. A first estimate of the required capacity of the pump is 8 kW.

The type of the pump will be determined later and it is possible to select two smaller pumps in parallel instead of one bigger pump. The pump should be located near to the reservoir, because the length of the suction pipe should be short. In the reservoir possibly a support of the suction pipe will be necessary to prevent vibrations. The selected location of the pump in the drawings can be shifted over a small distance, if required.

The diameter of the suction pipe and the pressure pipe is about 0.1 m.

A generator to guarantee an uninterrupted power supply to the pump, is considered as optional.

3 CONSTRUCTION MATERIALS AND EQUIPMENT

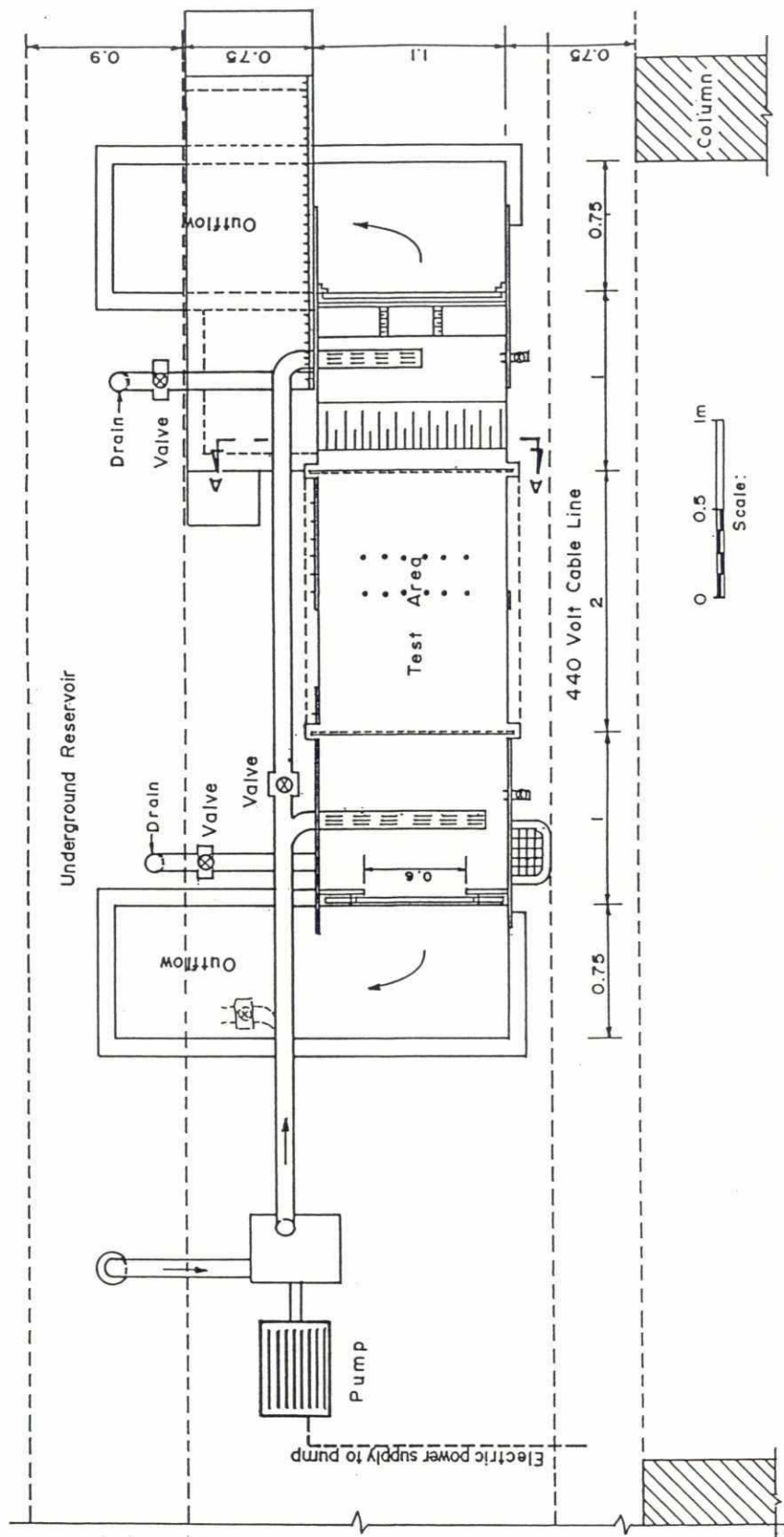
In the preliminary design the following materials have been used:

- reinforced concrete, bricks and cement,
- steel plates, steel frames, steel bars,
- plastic tubes, glas tubes,
- steel or plastic pipes, special end of suction pipe,
- glas windows, which are about 1.5 cm thick,
- different types of gauze, geotextiles,
- wooden bars, plate,
- silicone kit to prevent leakages,
- brick and mortar for the foundation and outflow channels,
- concrete slab for the apron to make the readings,
- miscellaneous small materials.

The following equipment will be needed:

- a standard V-shaped weir,
- a electric pump, electric connection line,
- three valves,
- two scales, to be attached near the glas tubes,
- two ladders,
- some lights, electric wire and some baskets,
- a balance,
- optionally a generator.

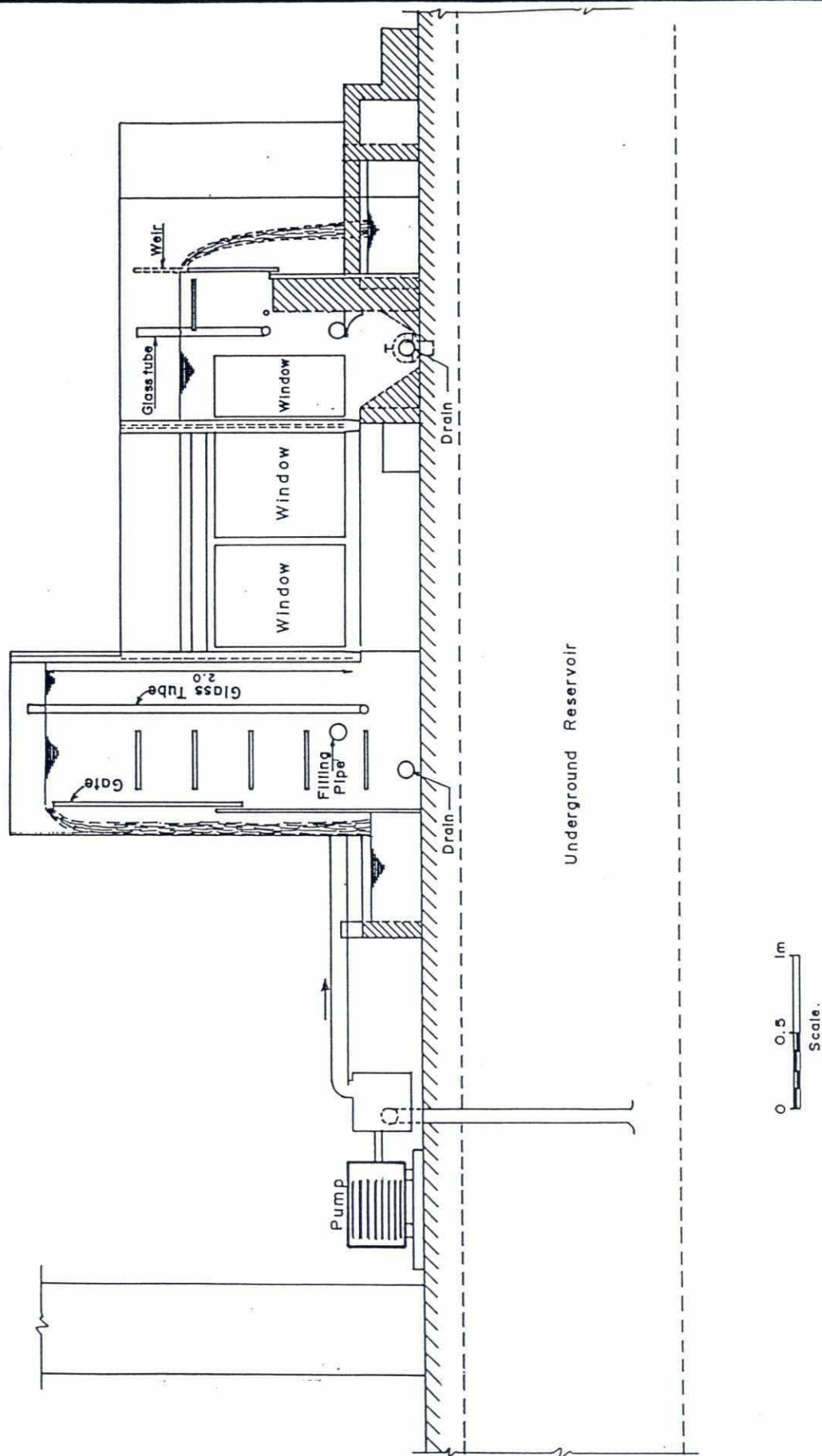
It seems to be that all these materials and the majority of the equipment can be purchased in Bangladesh, therefore no major delay due to the importation of goods from abroad is to be expected.



FUNCTIONAL DESIGN TEST RIG, PLAN

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INTERIM REPORT FIG. A6-1



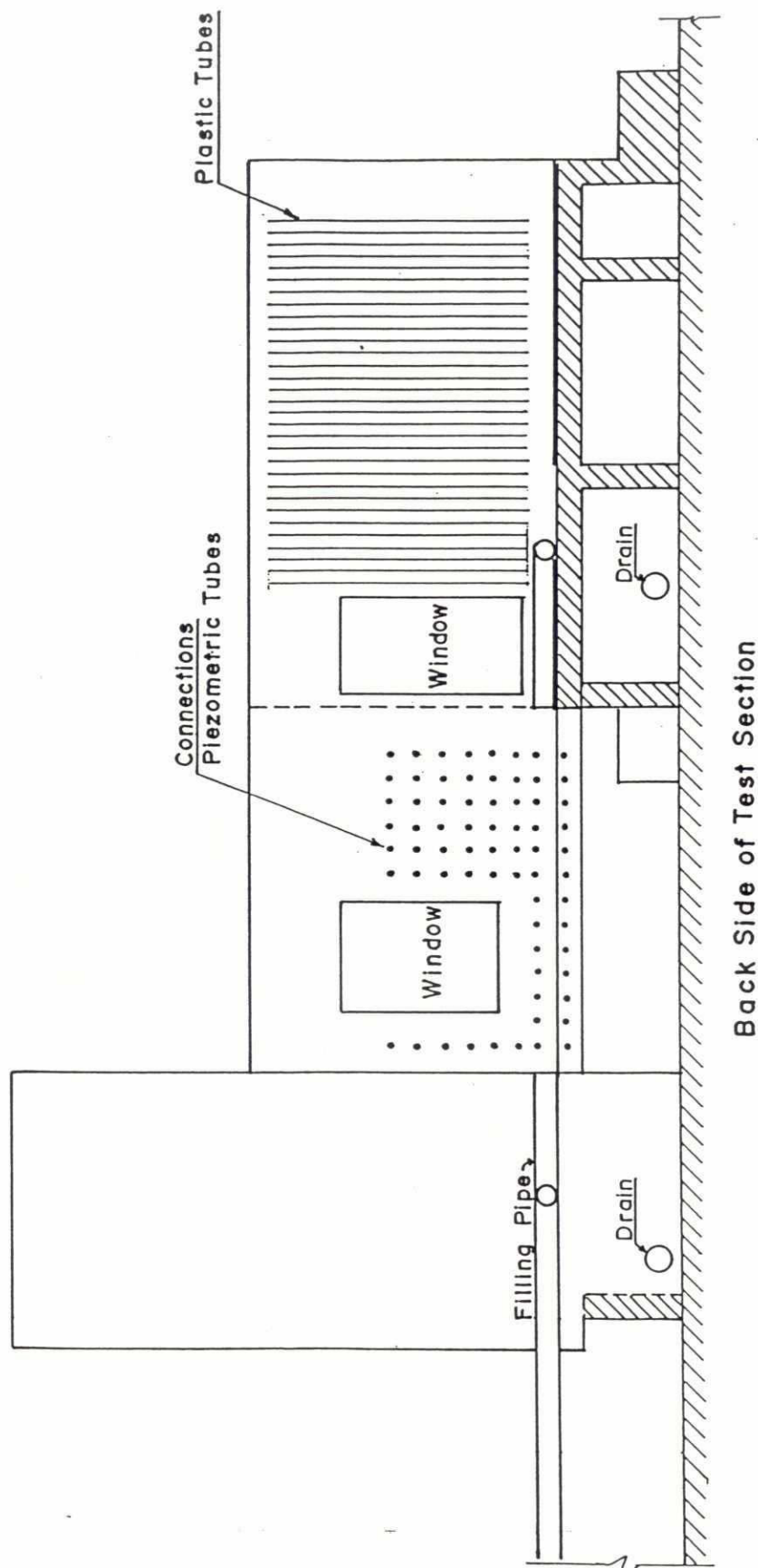
FUNCTIONAL DESIGN TEST RIG,
LONGITUDINAL SECTION

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(AFPM) PILOT PROJECT

INTERIM REPORT

FIG. A6 - 2



FUNCTIONAL DESIGN TEST RIG,
LONGITUDINAL SECTION

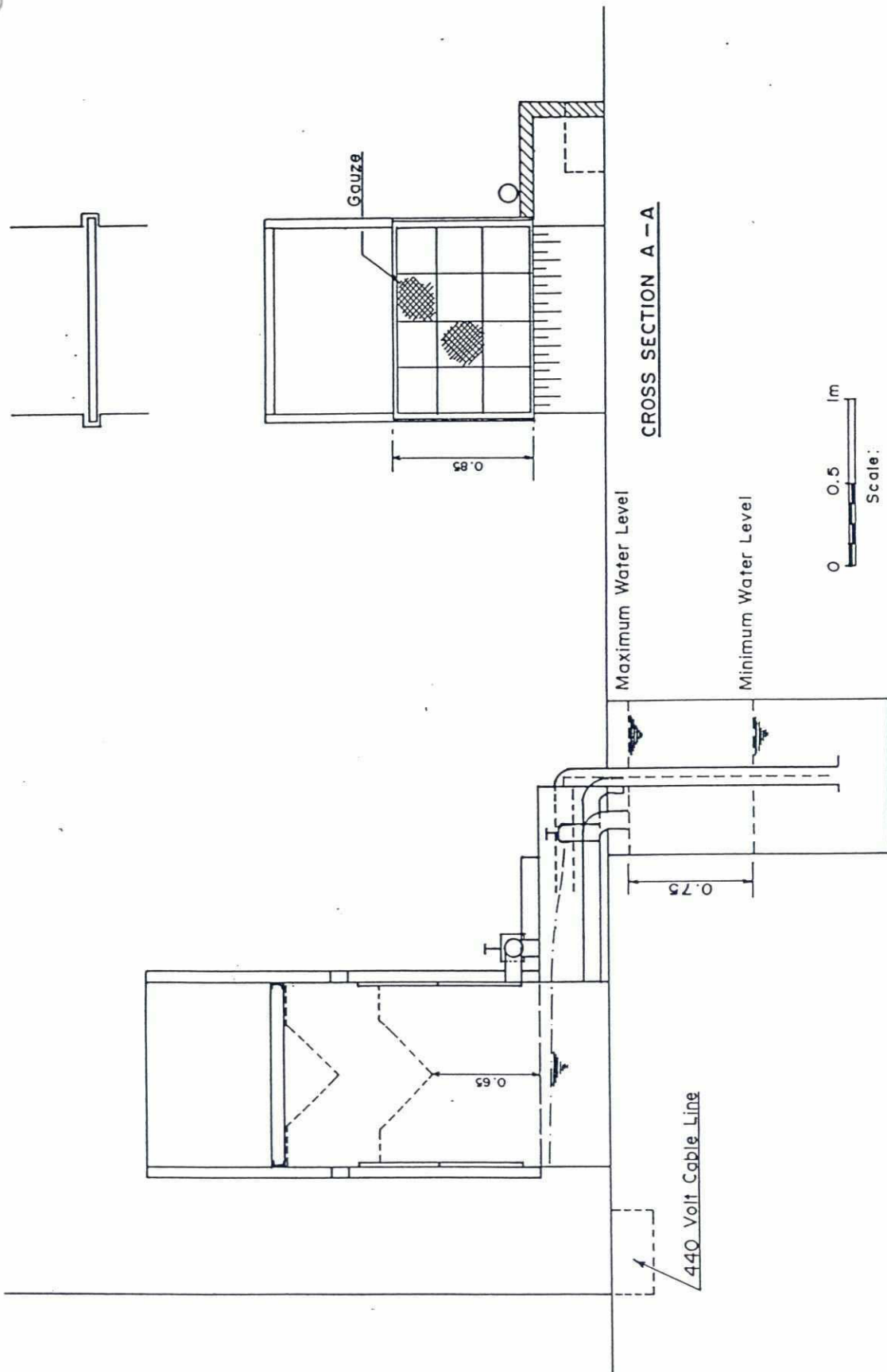
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FIG. A6 - 3

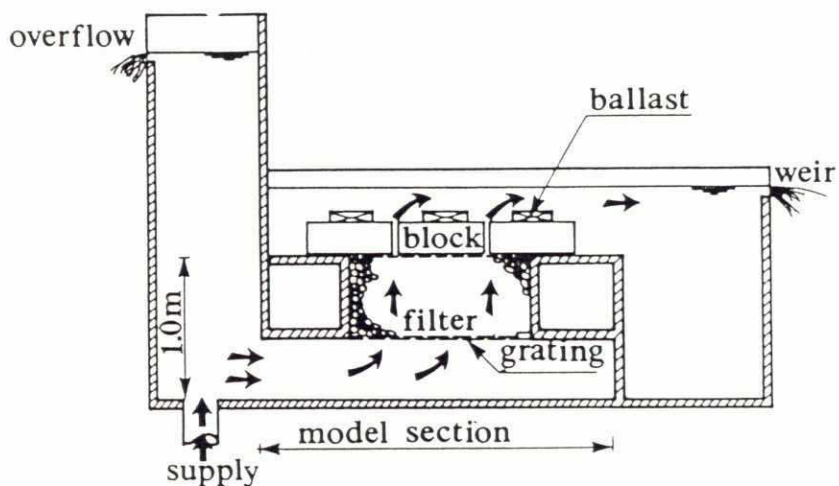
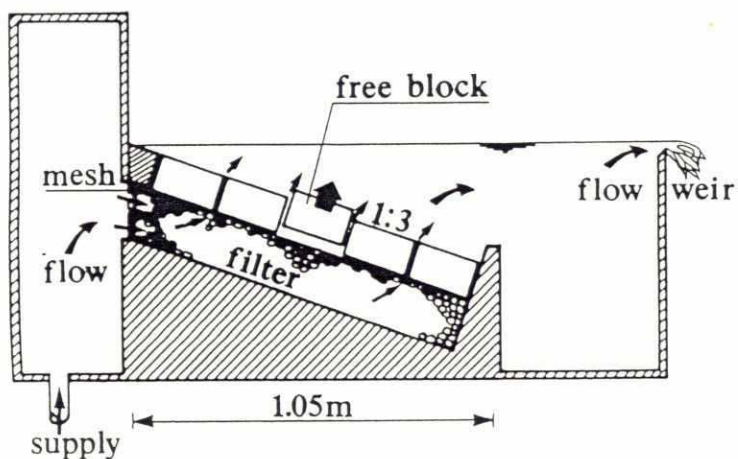
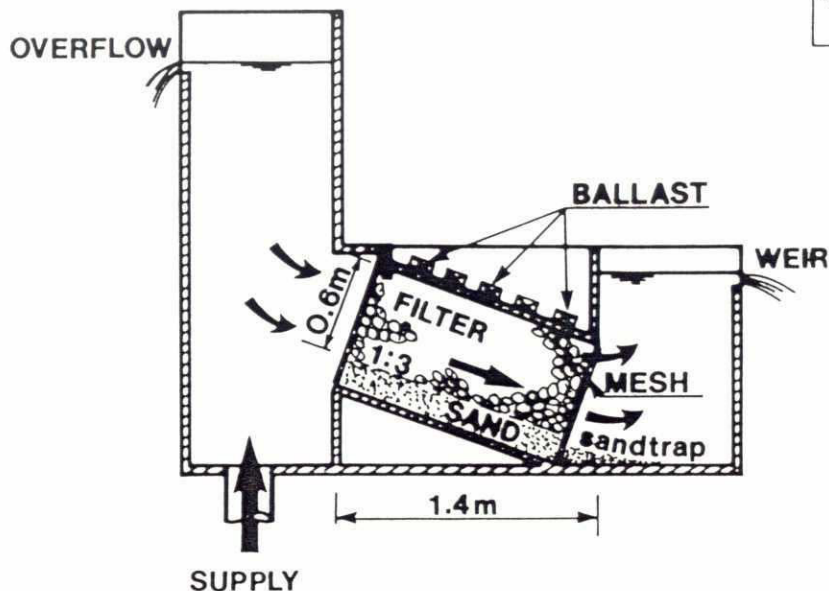
130



FUNCTIONAL DESIGN TEST RIG,
CROSS SECTIONS

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BANK PROTECTION AND RIVER TRAINING /
(A FPM) PILOT PROJECT

INTERIM REPORT FIG. A6- 4



EXAMPLES OF TESTED FILTER AND TOPLAYERS

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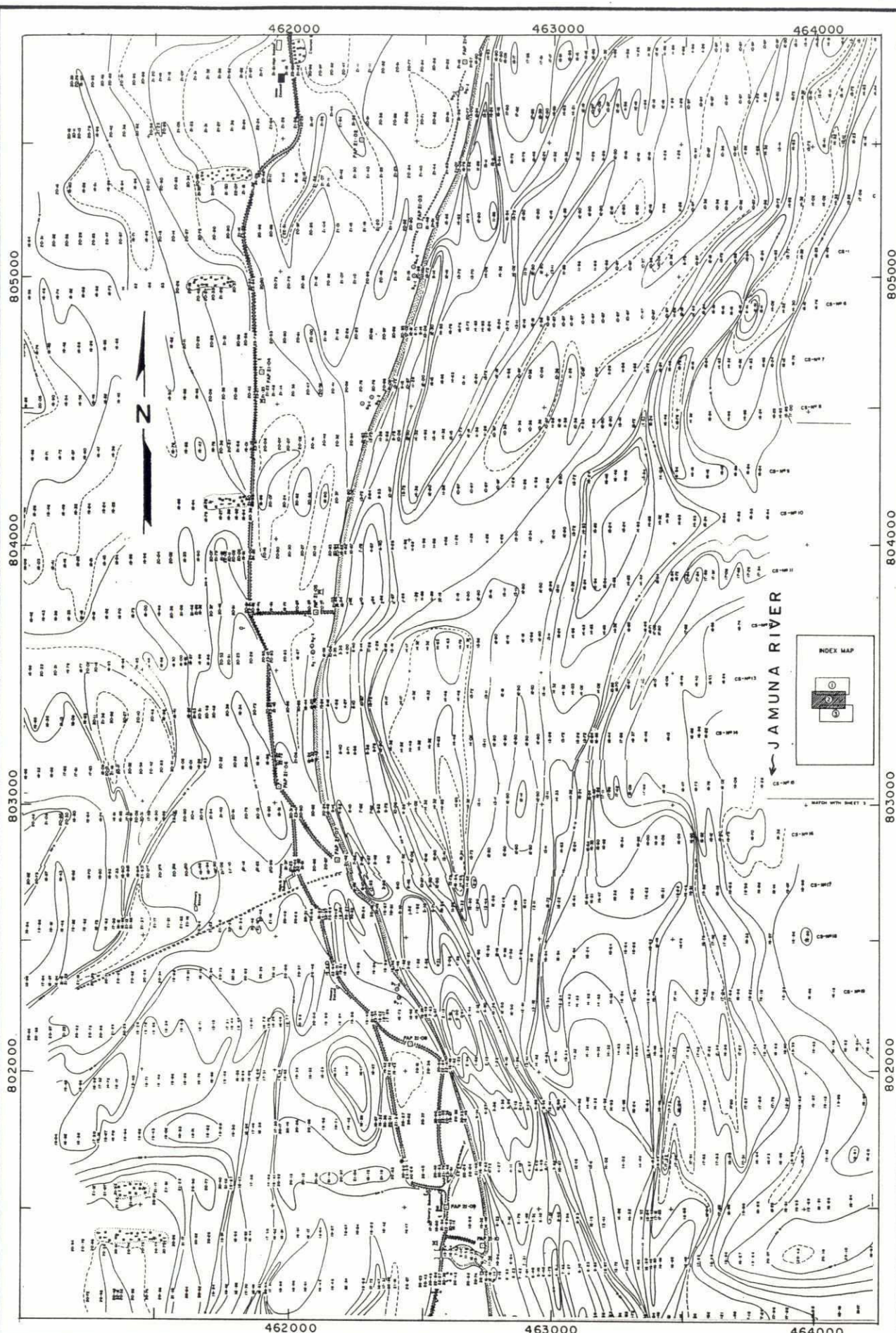
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INTERIM REPORT

FIG. A6-5

ANNEX 7

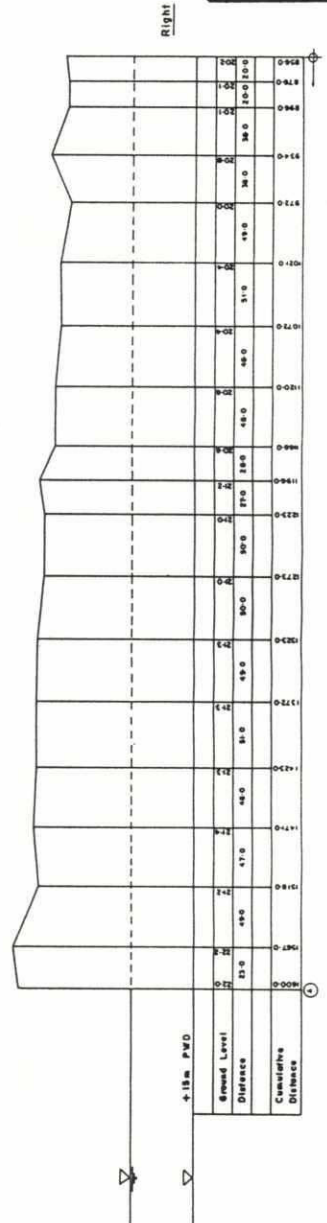
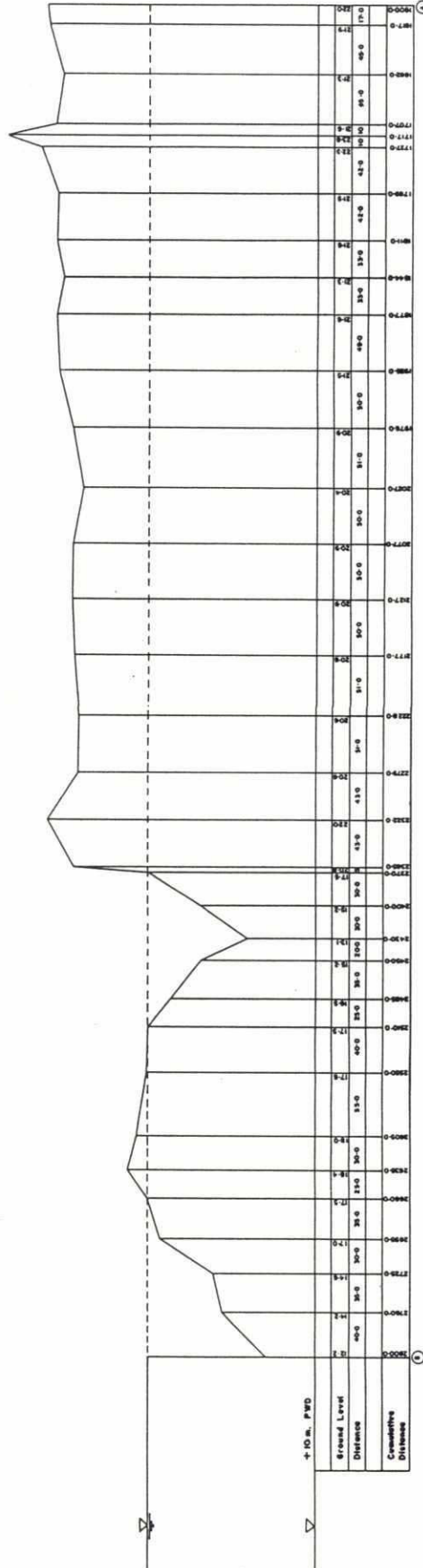
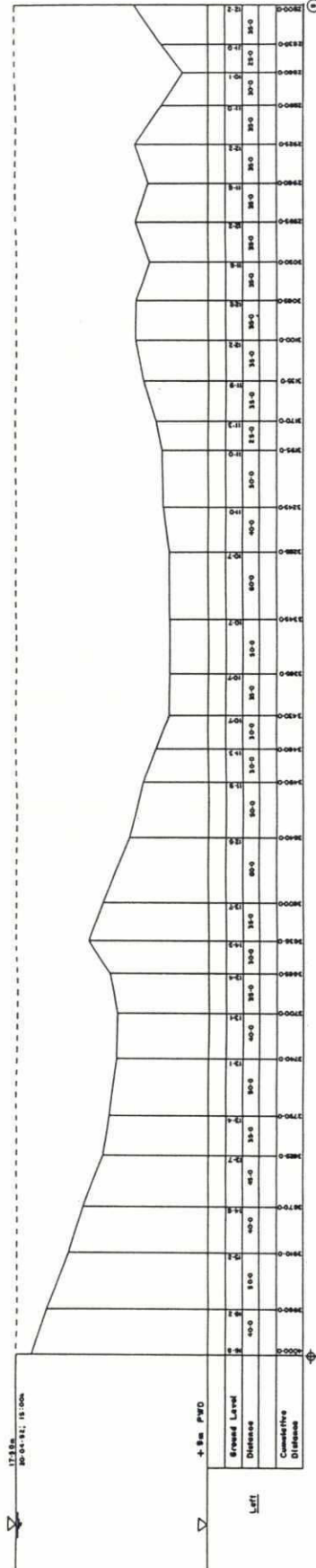
EXAMPLES OF SURVEY DRAWINGS PREPARED



KAMARJANI
TOPOGRAPHIC AND HYDROGRAPHIC SURVEY
OF APRIL 1992
SECTION OF SURVEY MAP
SCALE 1:20000

FAP - 21/22
BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT
INTERIM REPORT FIG. A7-1

CS - No 2/Km 0 + 400

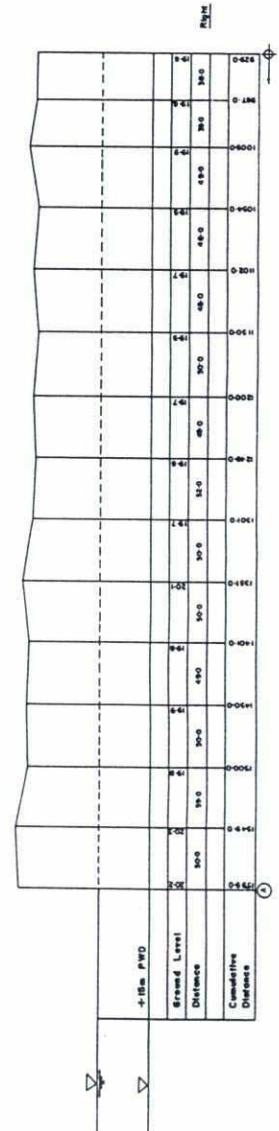
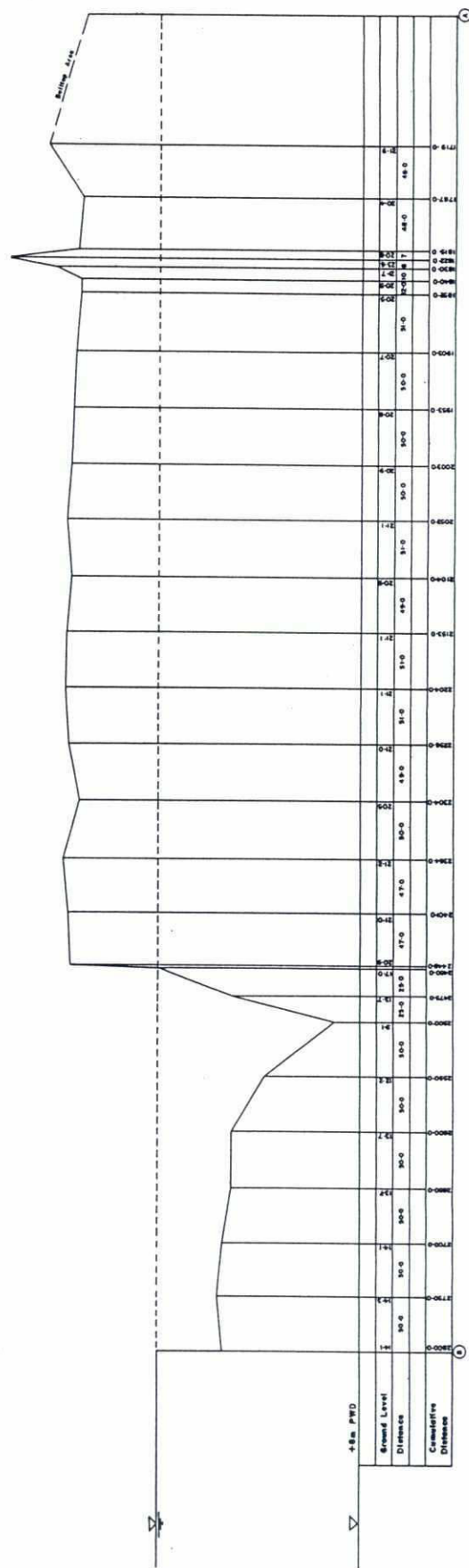
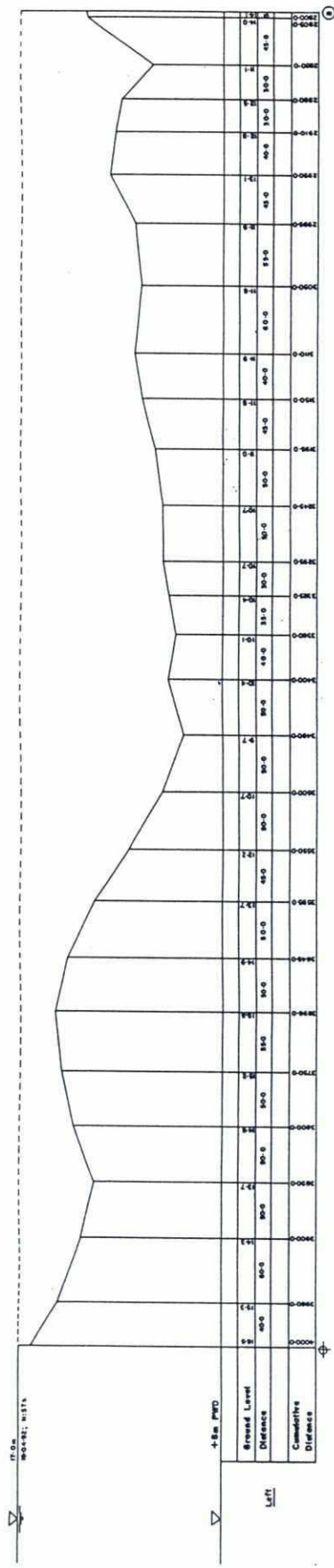


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KAMARJANI
TOPOGRAPHIC AND
HYDROGRAPHIC SURVEY
TYPICAL CROSS SECTION

INTERIM REPORT FIG. A7 - 2

CS-No 5/Km 1+0

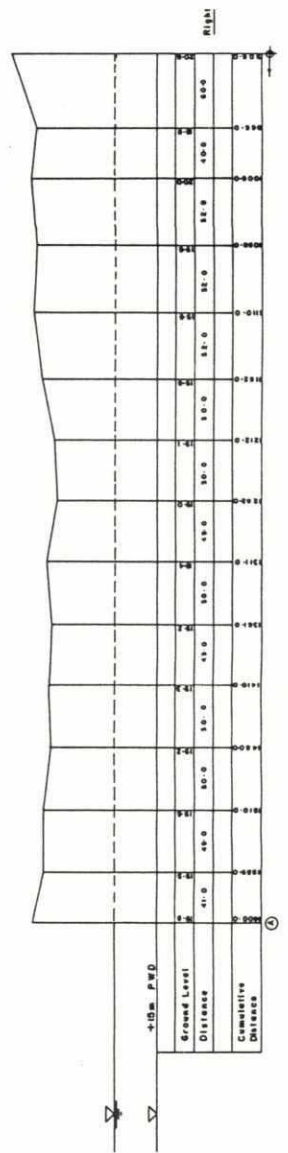
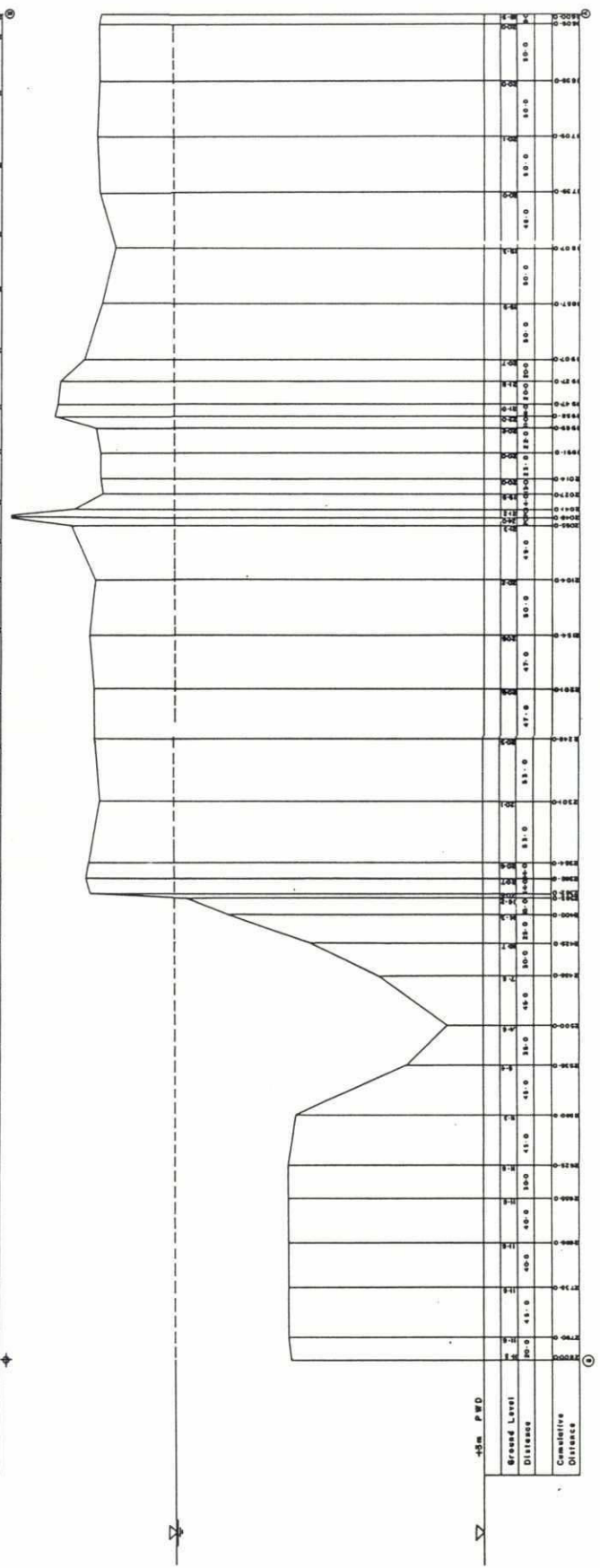
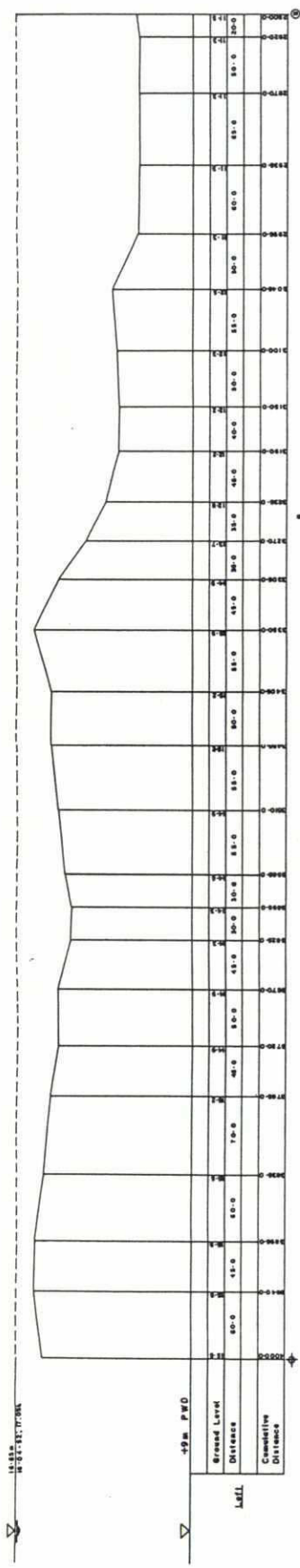


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TOPOGRAPHIC AND
HYDROGRAPHIC SURVEY

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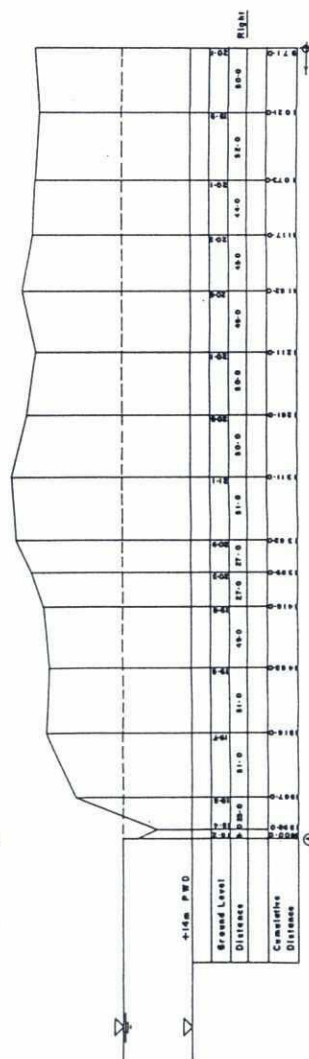
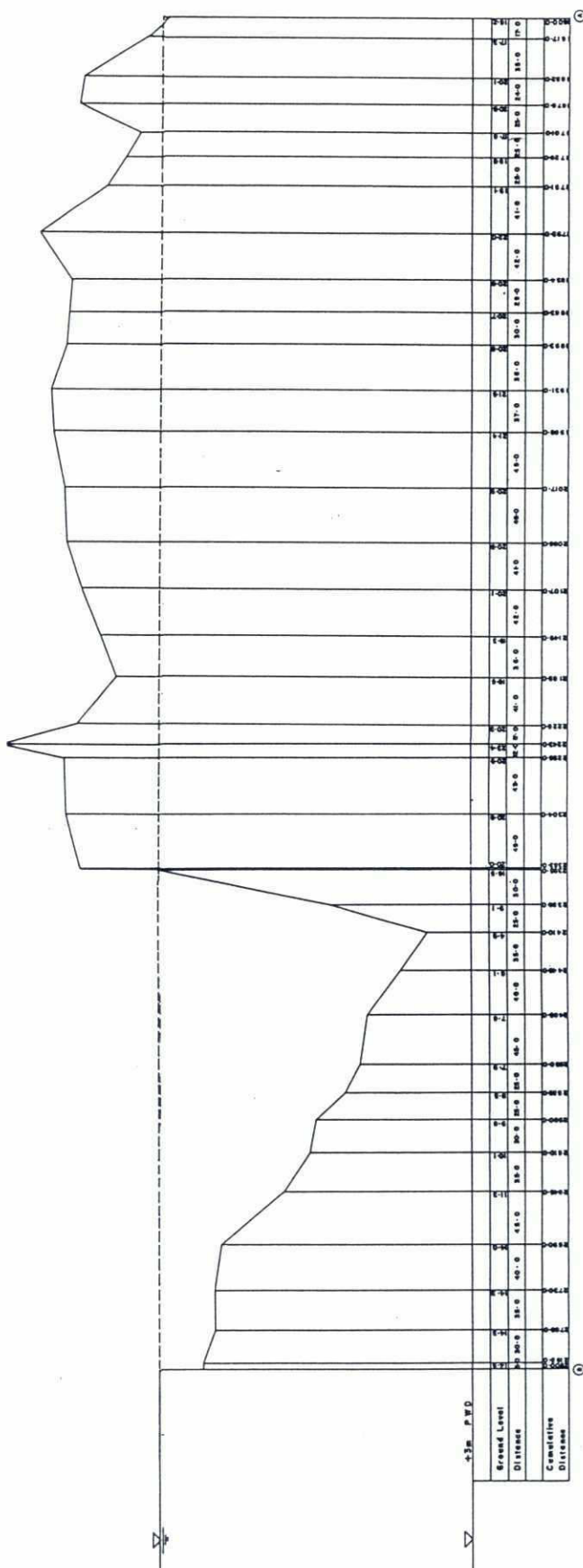
INTERIM REPORT FIG. A 7 - 3

CS-No 10/Km 2+0



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TOPOGRAPHIC AND
HYDROGRAPHIC SURVEY
TYPICAL CROSS SECTION

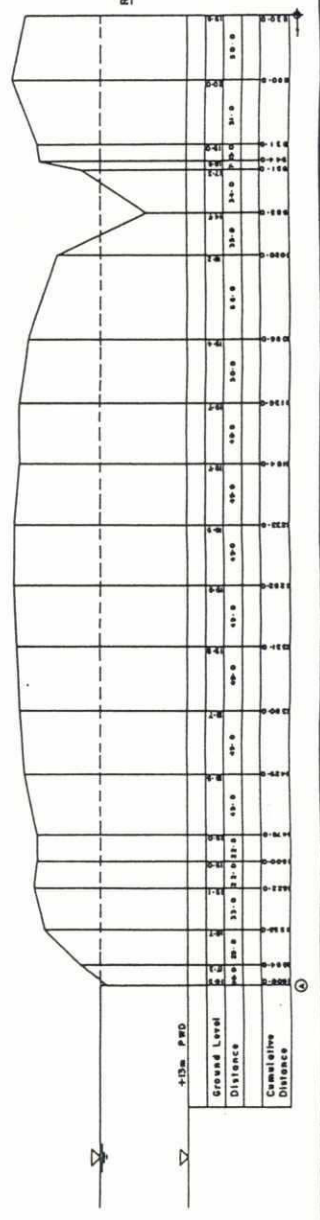
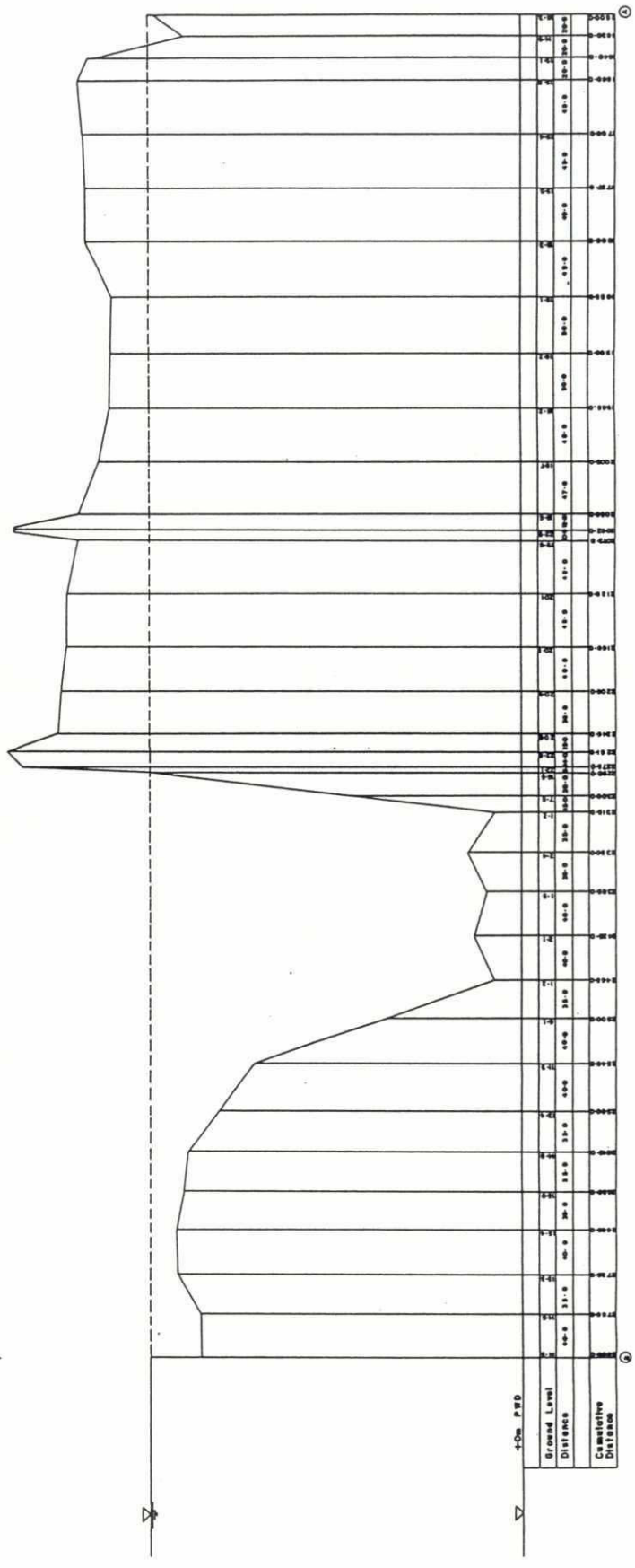
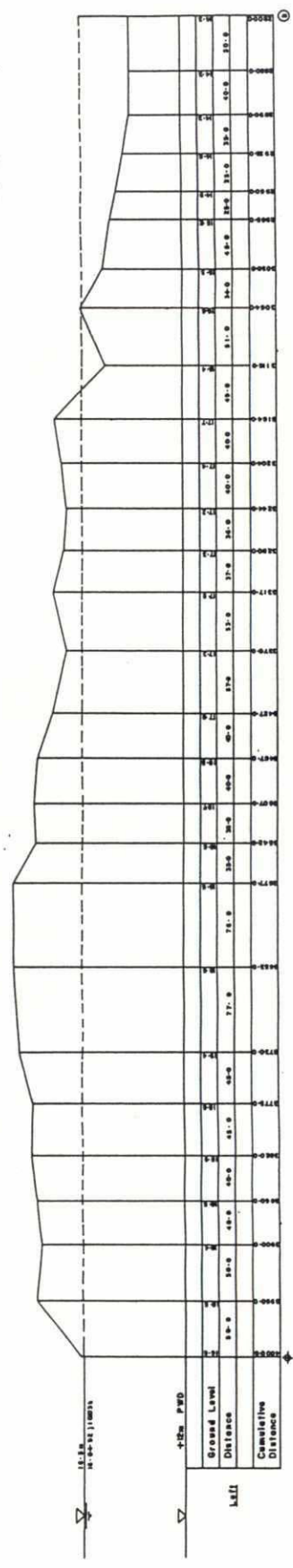
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BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT
INTERIM REPORT FIG. A7 - 4



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BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT

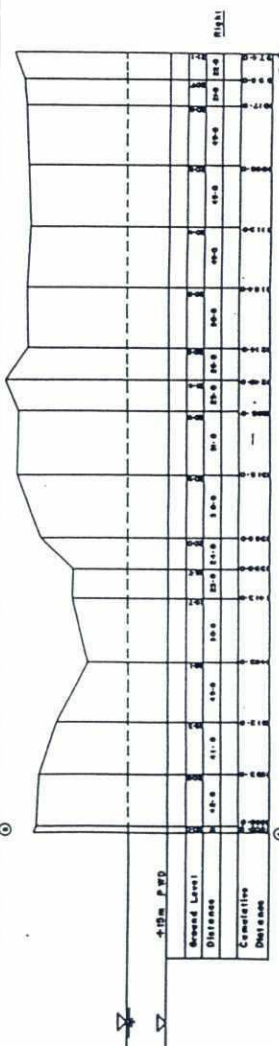
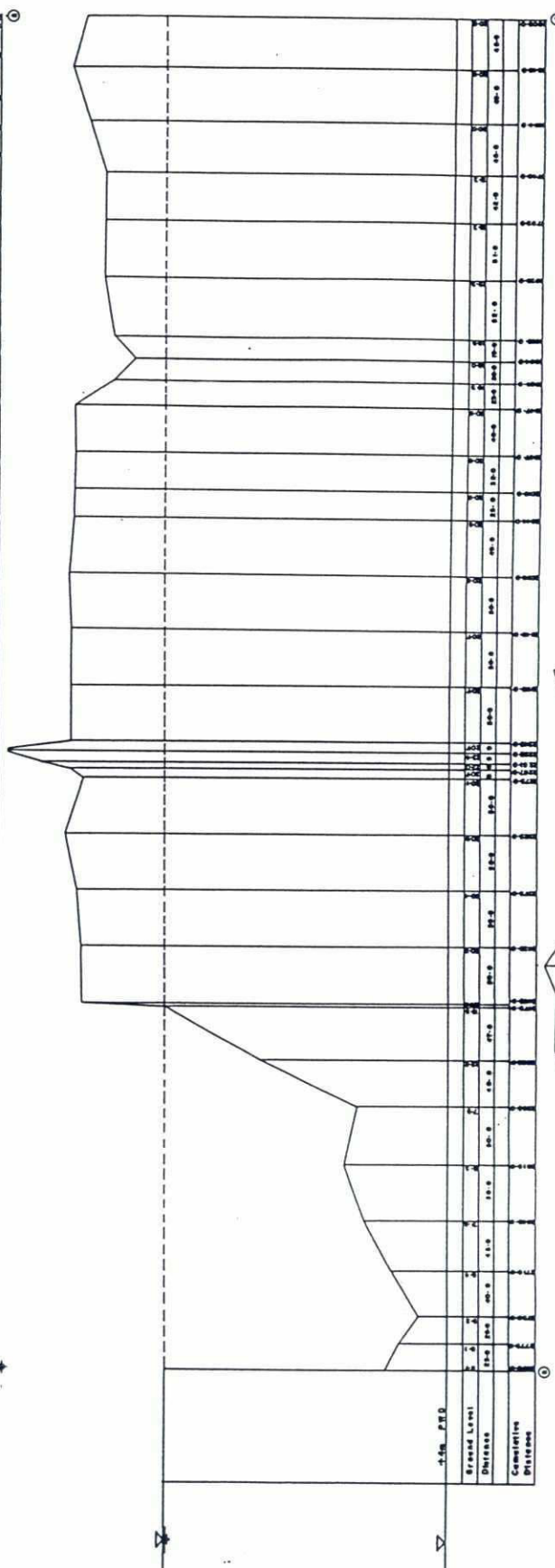
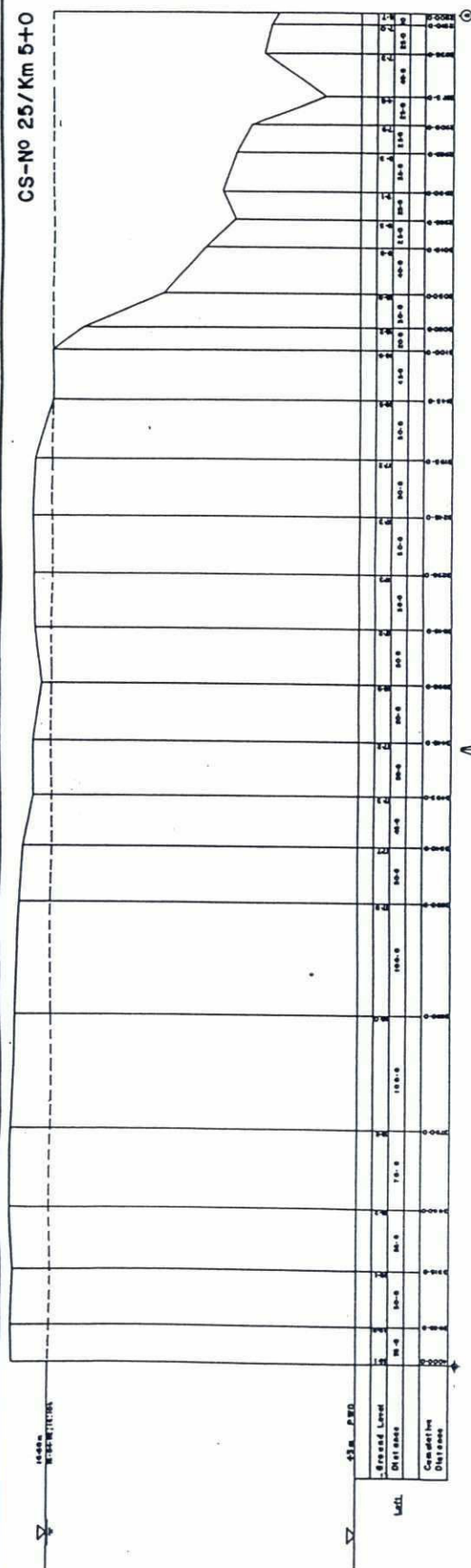
INTERIM REPORT | FIG. A7 - 5

CS-No 20/Km 4+0



KAMARJANI
TOPOGRAPHIC AND
HYDROGRAPHIC SURVEY
TYPICAL CROSS SECTION

FAP - 21/22
BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT
INTERIM REPORT FIG. A7 - 6

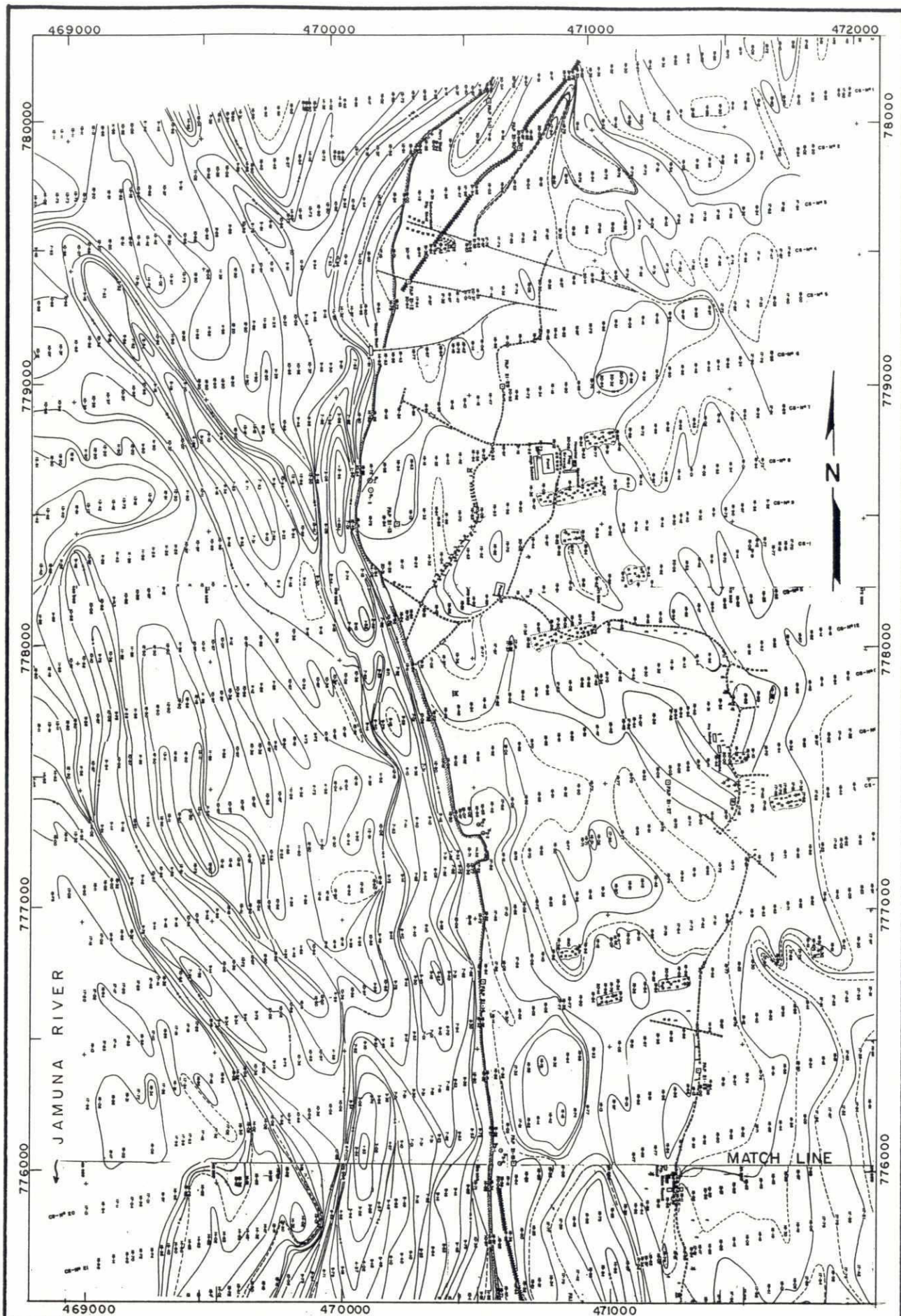


KAMARJANI TOPOGRAPHIC AND HYDROGRAPHIC SURVEY TYPICAL CROSS SECTION

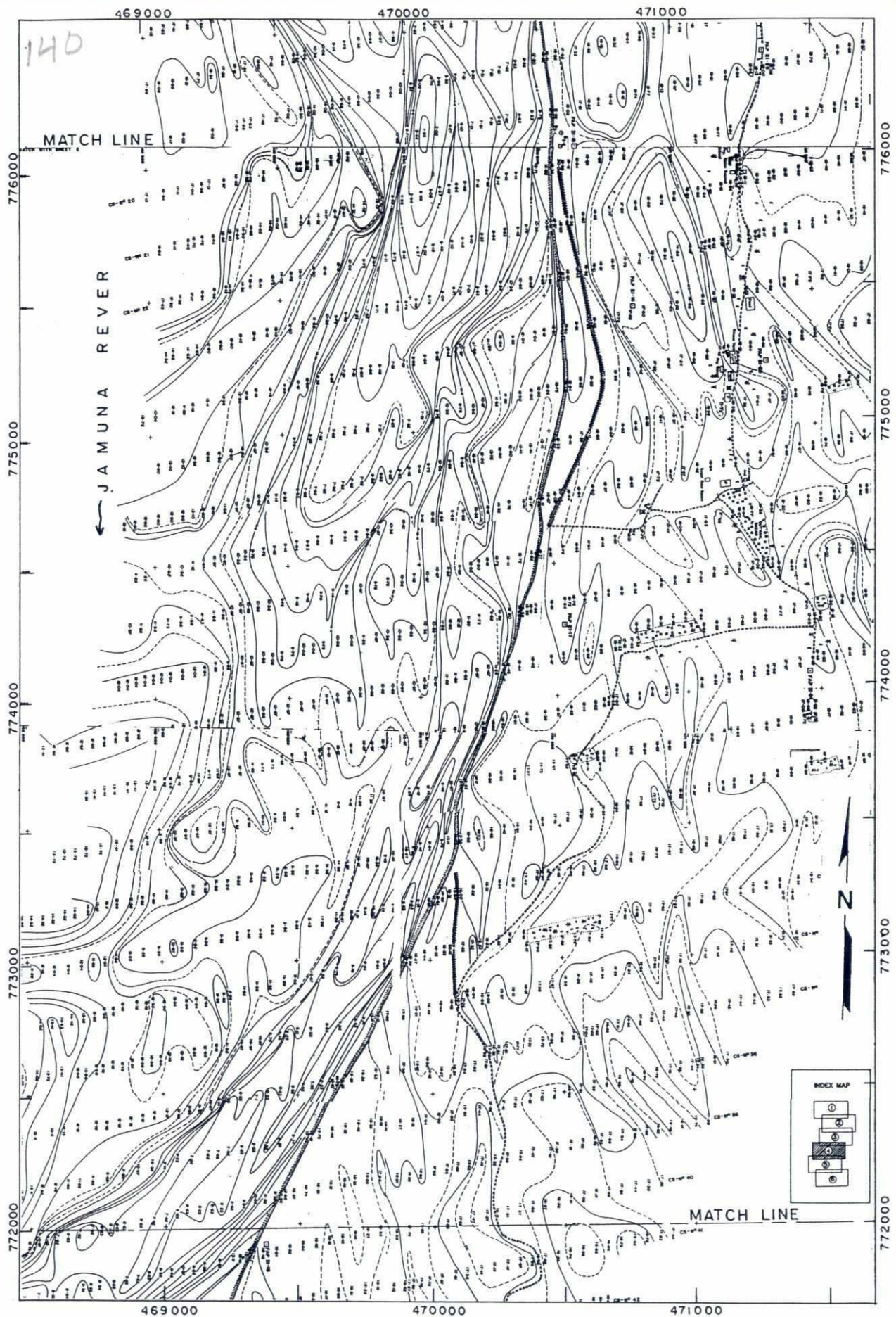
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BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT

INTERIM REPORT FIG. A7-7



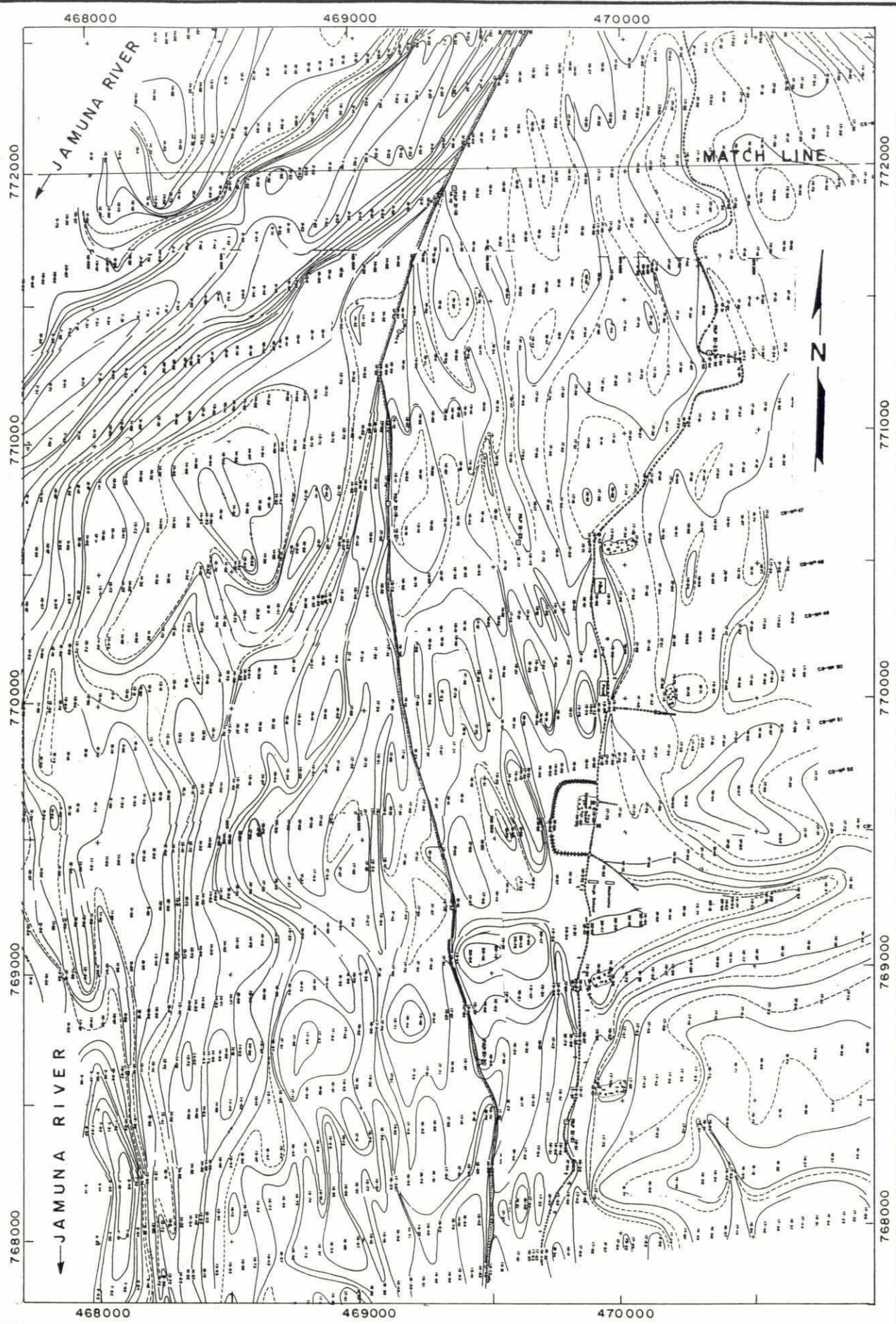
BAHADURABAD TOPOGRAPHIC AND HYDROGRAPHIC SURVEY OF APRIL / MAY 1992 SECTION OF GROUND PLAN SCALE 1: 20000	FAP - 21/22 BANK PROTECTION AND RIVER TRAINING/ (AFPM) PILOT PROJECT INTERIM REPORT FIG. A7- 8
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BAHADURABAD
TOPOGRAPHIC AND HYDROGRAPHIC SURVEY
OF APRIL / MAY 1992
SECTION OF GROUND PLAN
SCALE 1: 20000

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

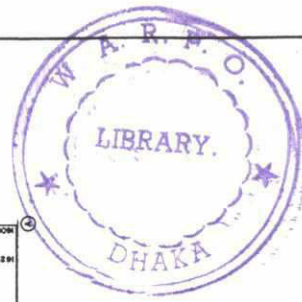
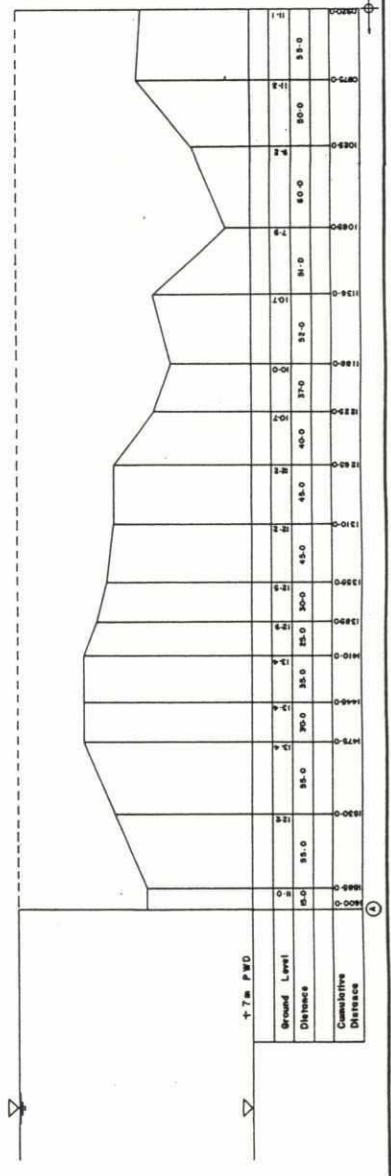
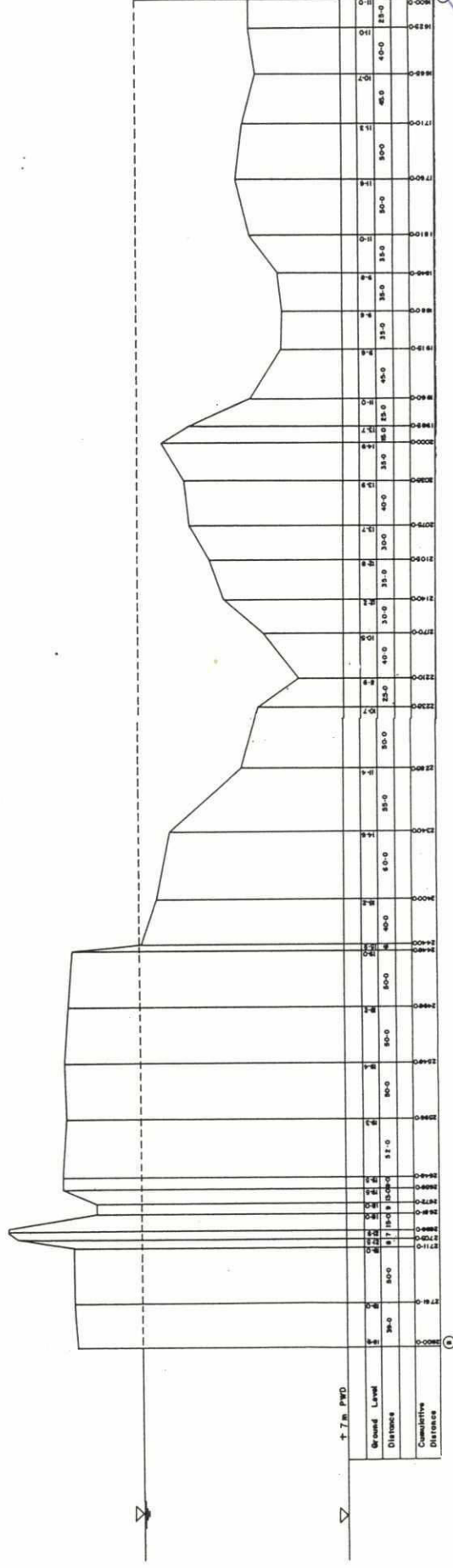
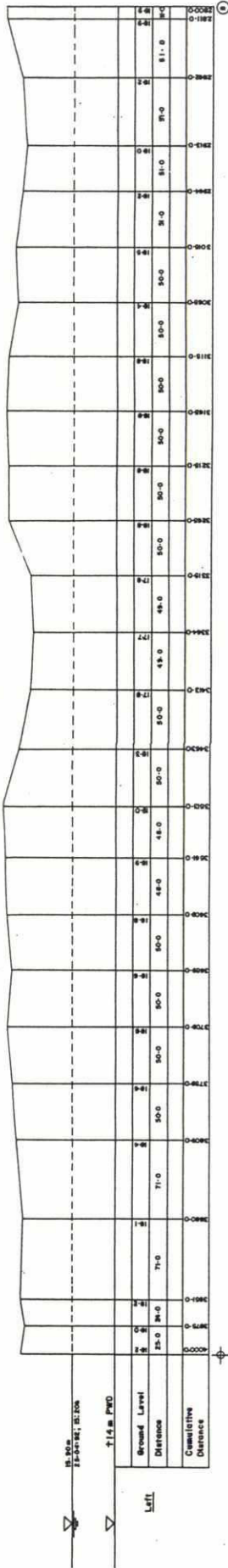
INTERIM REPORT FIG. A7 - 9



BAHADURABAD
TOPOGRAPHIC AND HYDROGRAPHIC SURVEY
OF APRIL / MAY 1992
SECTION OF GROUND PLAN
SCALE 1: 20000

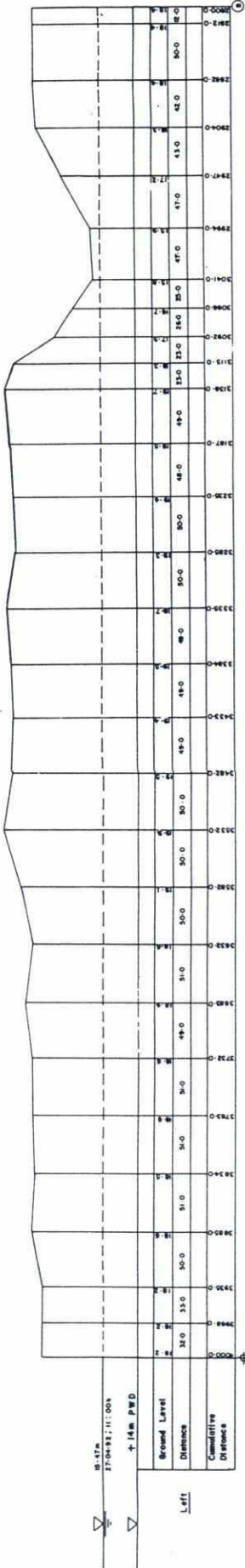
FAP - 21/22
BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT
INTERIM REPORT FIG. A7 - 10

CS - No 2 / Km 0 + 400

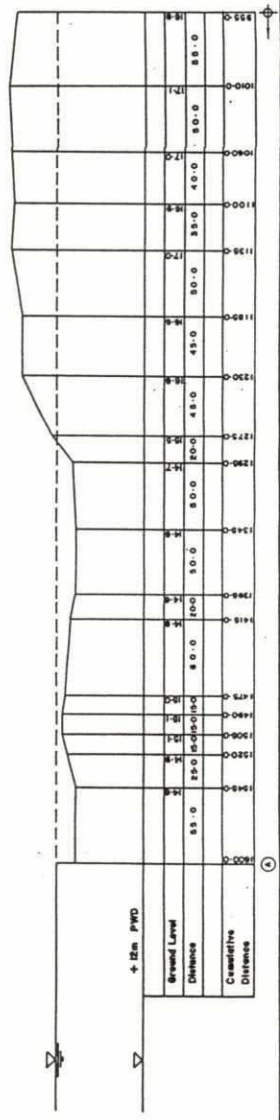
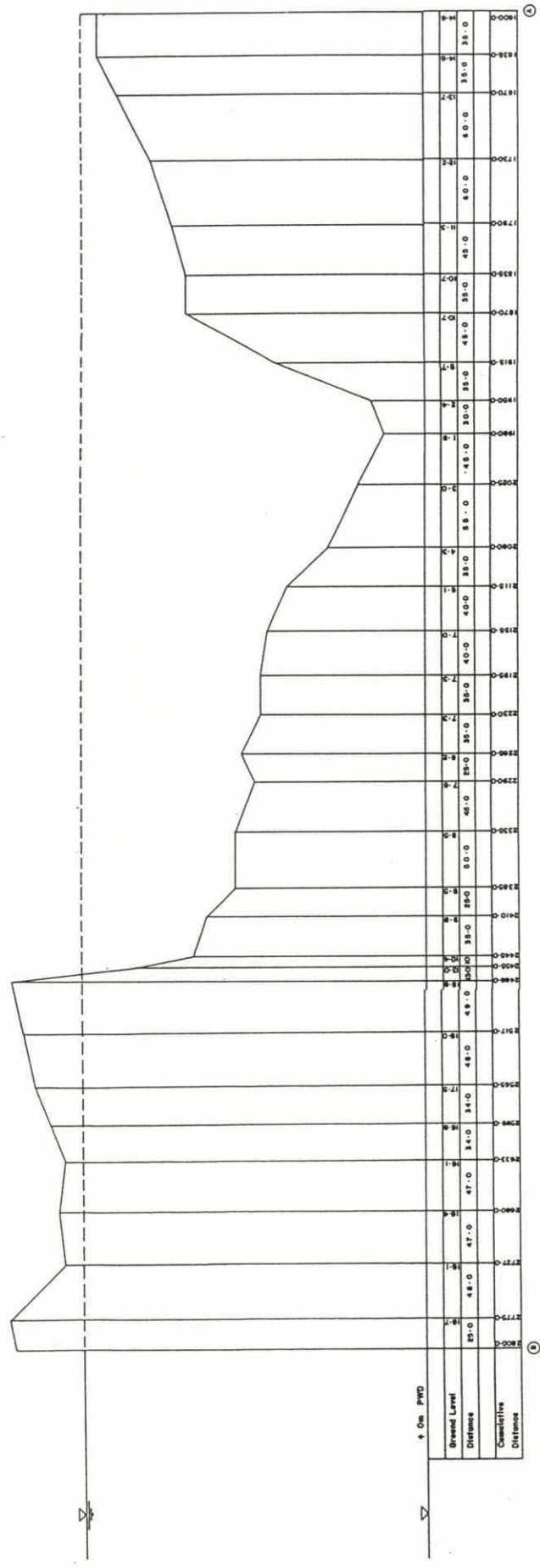
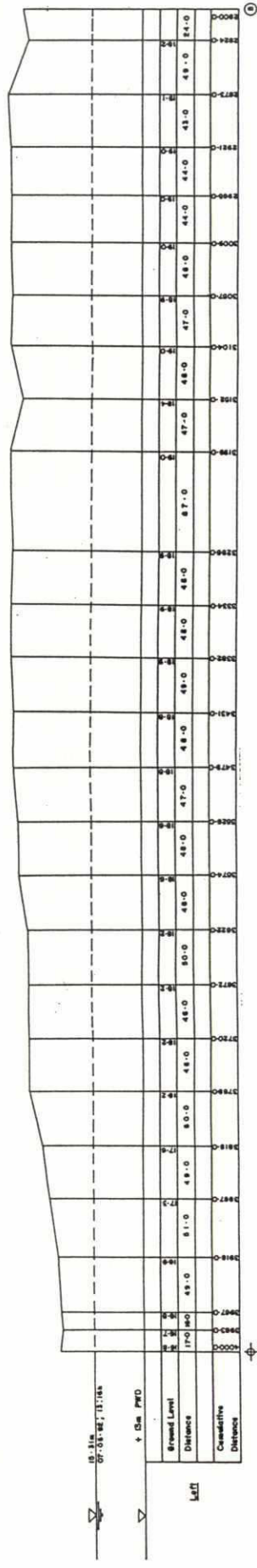


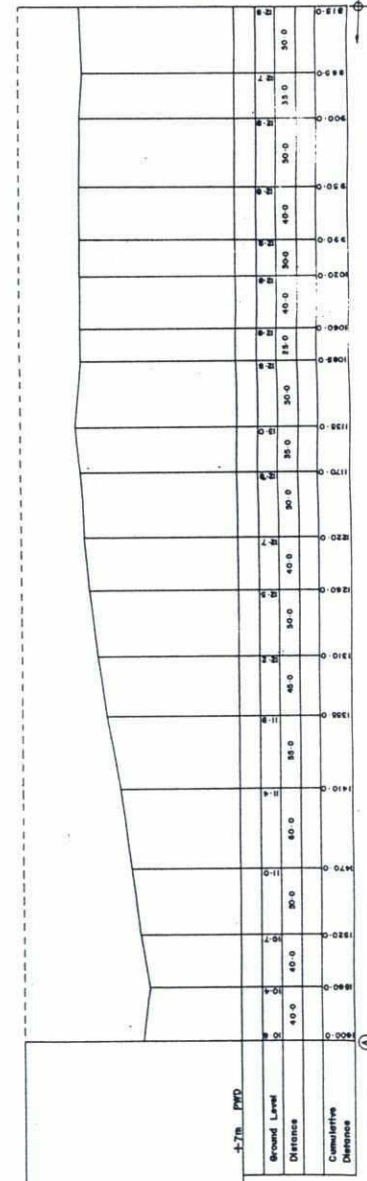
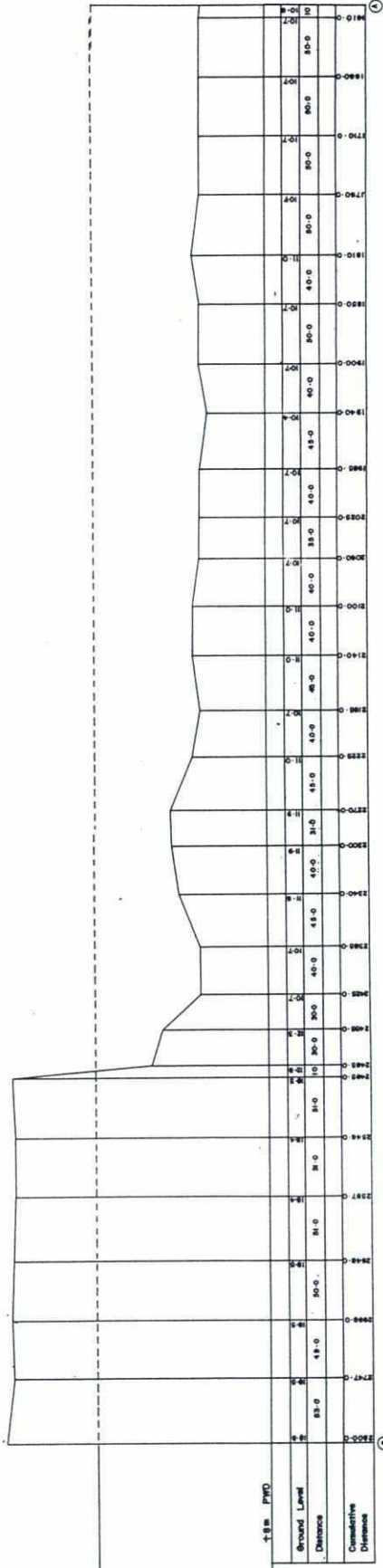
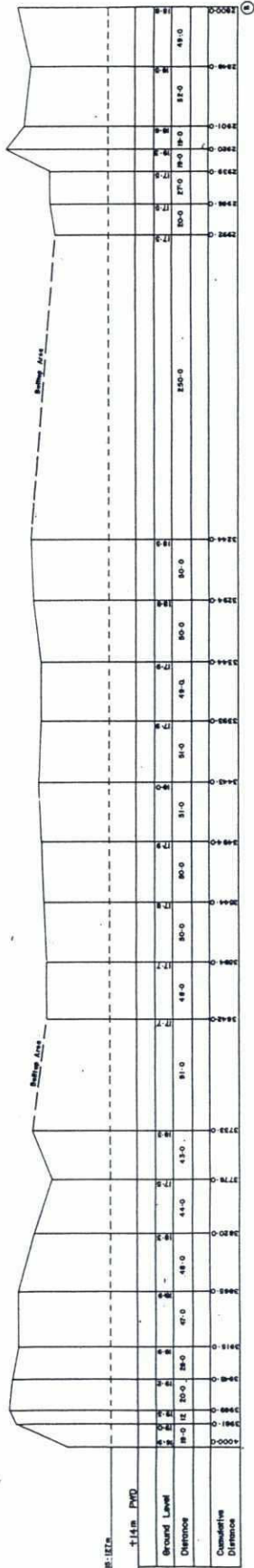
BAHADURABAD
TOPOGRAPHIC AND
HYDROGRAPHIC SURVEY
TYPICAL CROSS SECTION

FAP - 21/22
BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT
INTERIM REPORT FIG. A7 - 11



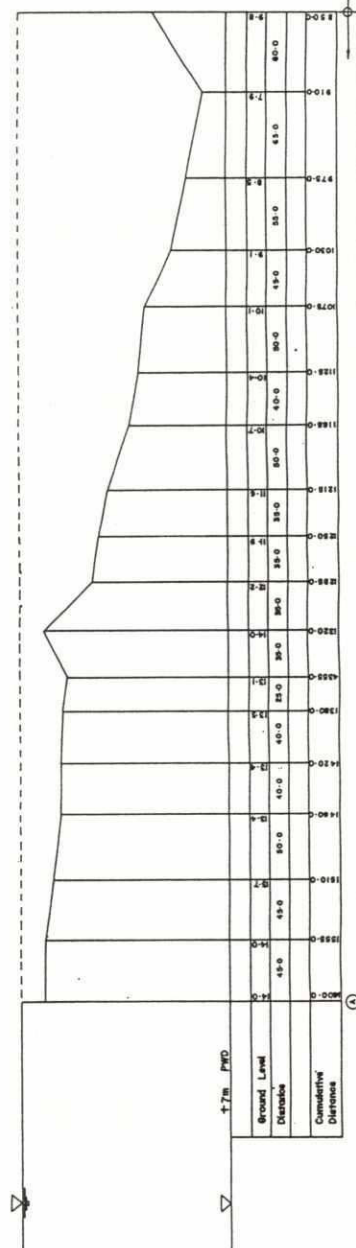
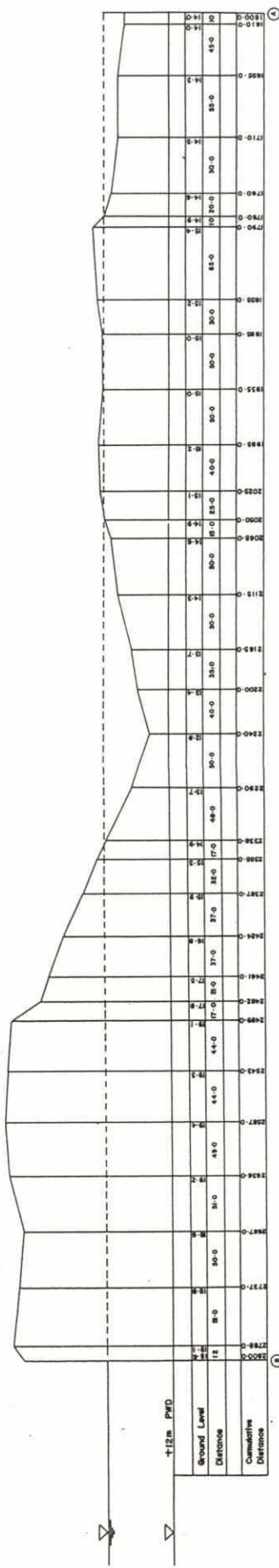
CS-Nº 20/Km 4+0





BAHADURABAD
TOPOGRAPHIC AND
HYDROGRAPHIC SURVEY
TYPICAL CROSS SECTION

FAP - 21/22
BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT
INTERIM REPORT FIG. A7 - 14

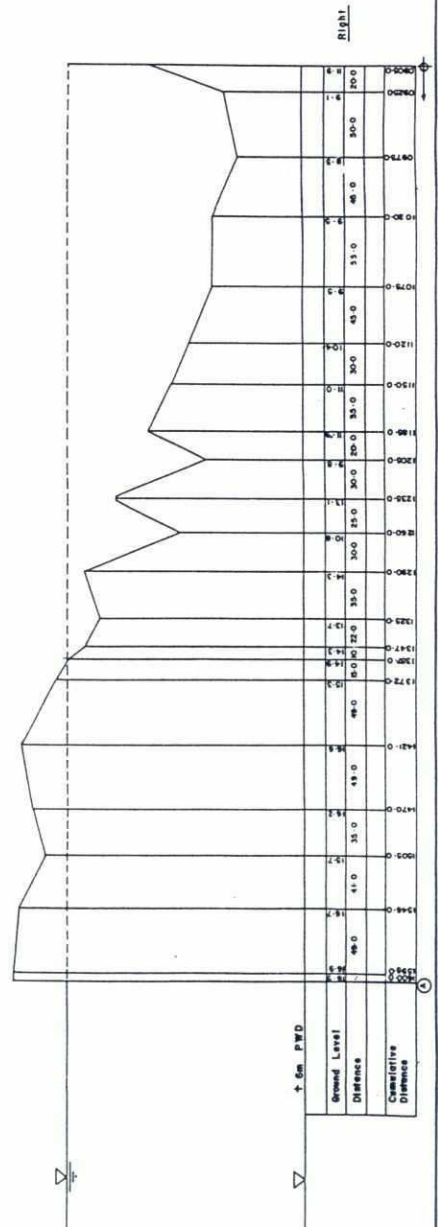
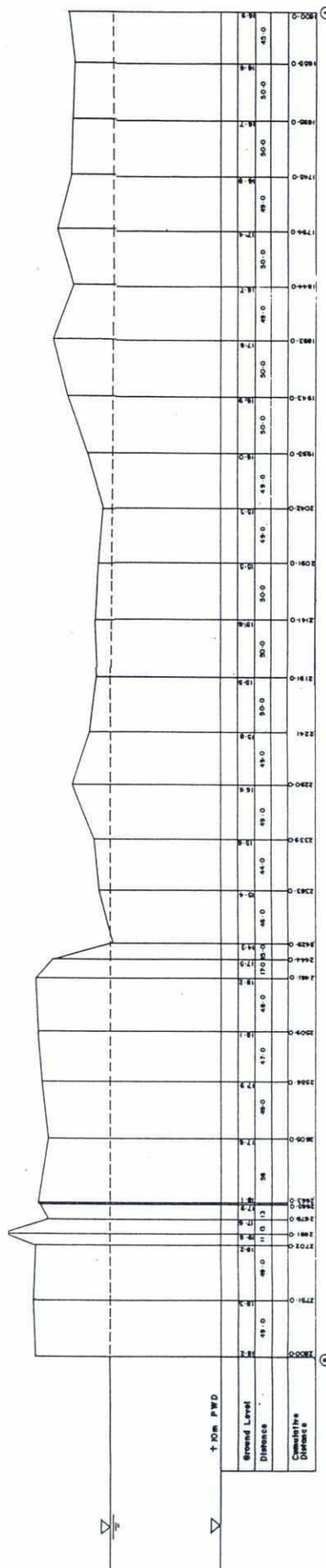
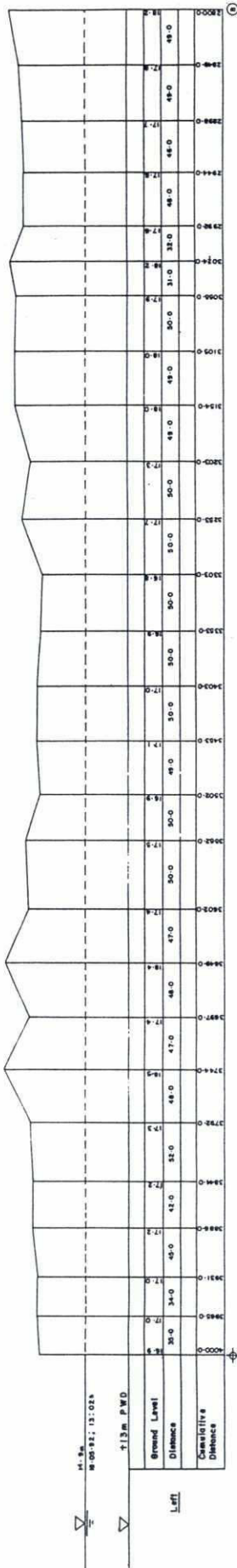


BAHADURABAD
TOPOGRAPHIC AND
HYDROGRAPHIC SURVEY
TYPICAL CROSS SECTION

BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT

INTERIM REPORT FIG. A7-15

147

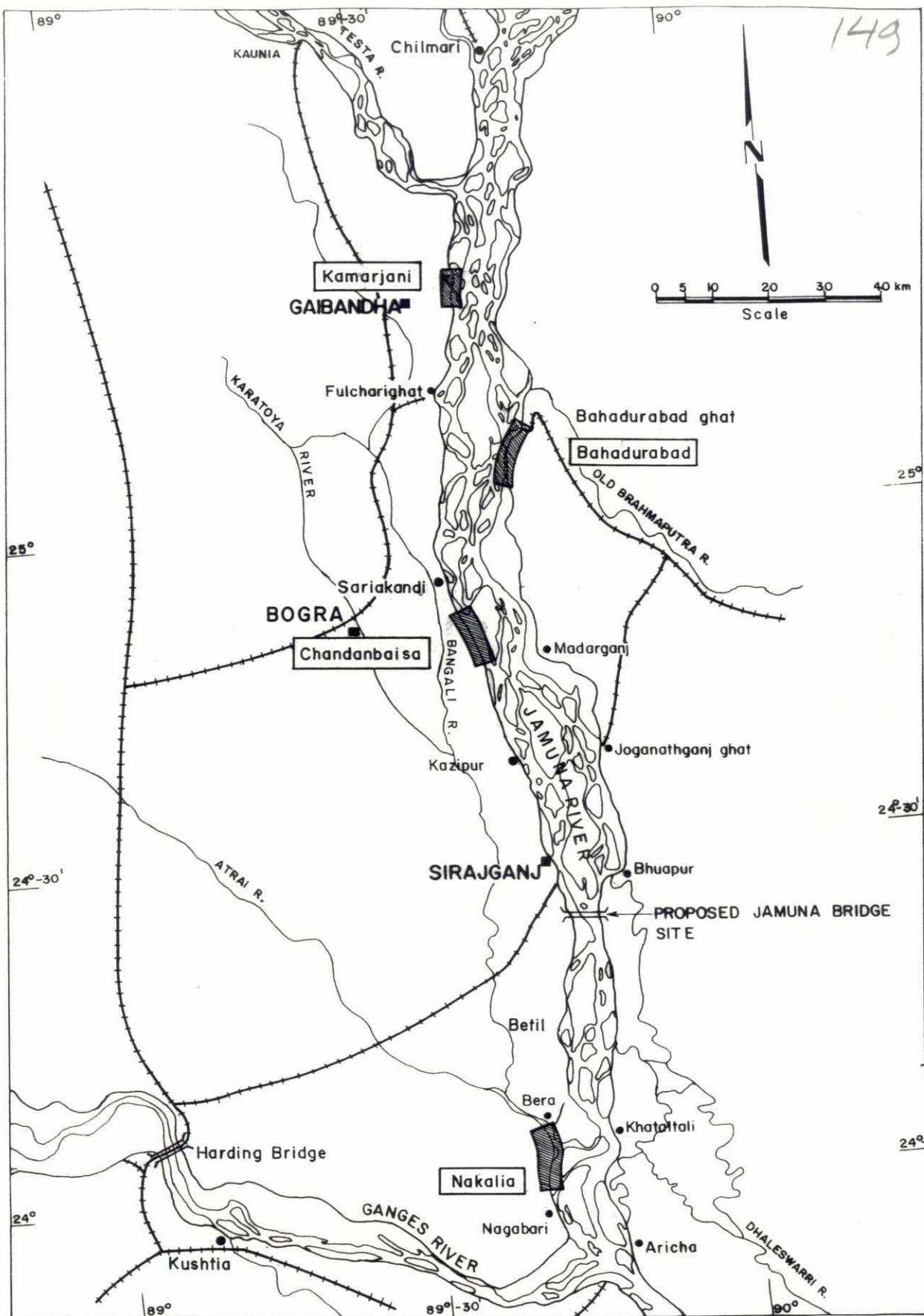


BAHADURABAD
TOPOGRAPHIC AND
HYDROGRAPHIC SURVEY
TYPICAL CROSS SECTION

FAP - 21/22
 BANK PROTECTION AND RIVER TRAINING/
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 INTERIM REPORT FIG. A 7 - 16

ANNEX 8

SUBSOIL INVESTIGATION RESULTS (INCOMPETE)



PRESELECTED TEST AREAS

FAP - 21/22

**BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT**

INTERIM REPORT

FIG. A8 - 0

LABORATORY TESTS PERFORMED (Nos.)

LOCATION	BH	S	MC	G	AL	UW	Dr	LI	TA	UC	C
CHANDANBAISA (CH)	1	13									
	2	13									
	3	13									
	4	9	(1)		1	1			(1)	1	
KAMARJANI (GA)	1	4	(1)		(1)	(1)			(1)		(1)
	2	5	(1)		(1)	(1)			(1)		(1)
	4	5									
	5	7	1+(1)		(1)	1+(1)			(1)	1	(1)
BAHADURABAD (BA)	1	8	1+(1)		(1)	1+(1)			(1)	1	1
	3	5									
	4	5									
	6	9	(1)			(1)			(1)		
NAKALIA (NK)	1	9	1+(1)		2	1+(1)			(1)	1	1
	3	7	1+(1)		2	1			(1)	1	1
	5	6	(1)		1	(1)			(1)		1
	7	4	(1)		1	(1)			(1)		1
TOTAL TESTS EXECUTED		122	4	No test done	7	5	No test done			5	5
STILL TO BE DONE			(10)		(4)	(8)			(10)		(3)
TESTS ASKED FOR	101	14	7	15	13	8	11	6	9		

LEGEND

S — Grain size distribution
 MC — Moisture content
 G — Specific gravity
 AL — Atterberg Limit
 UW — Unit weight
 Dr — Relative density
 LI — Loss on ignition
 TA — Triaxial shear test
 (Consolidated drained)
 UC — Unconfined Compression
 C — Consolidation test
 BH — Bore Hole

() No. of tests still to be executed

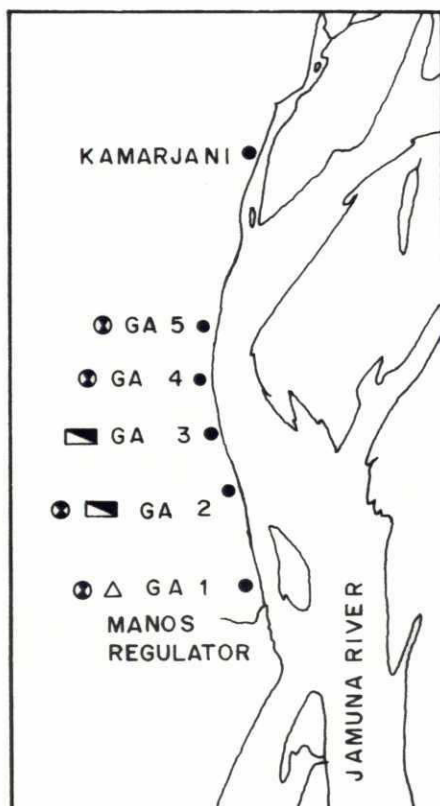
TYPE OF SOILS LABORATORY
TESTS EXECUTED

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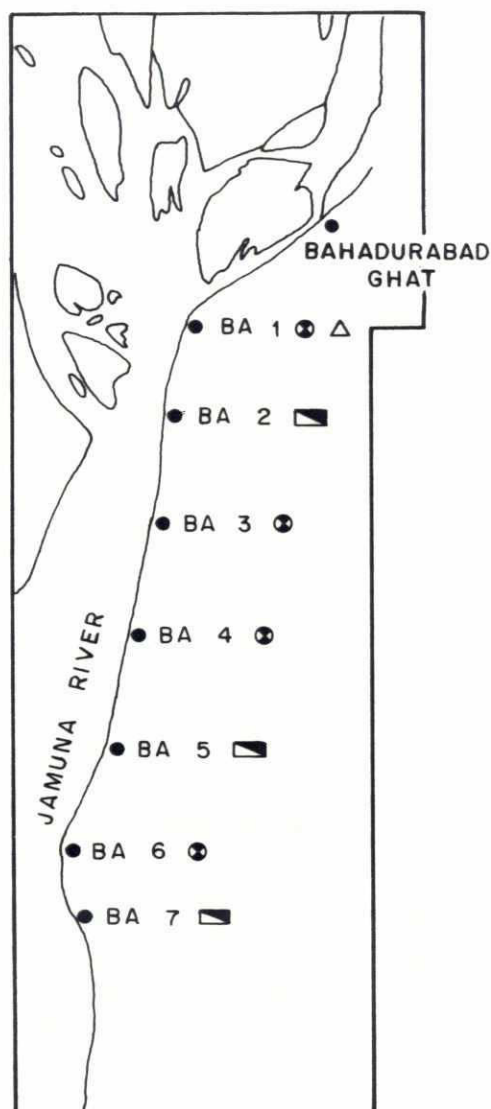
BANK PROTECTION AND RIVER TRAINING/
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INTERIM REPORT

TABLE A8-1



KAMARJANI
(NORTH OF MANOS REGULATOR)



BAHADURABAD

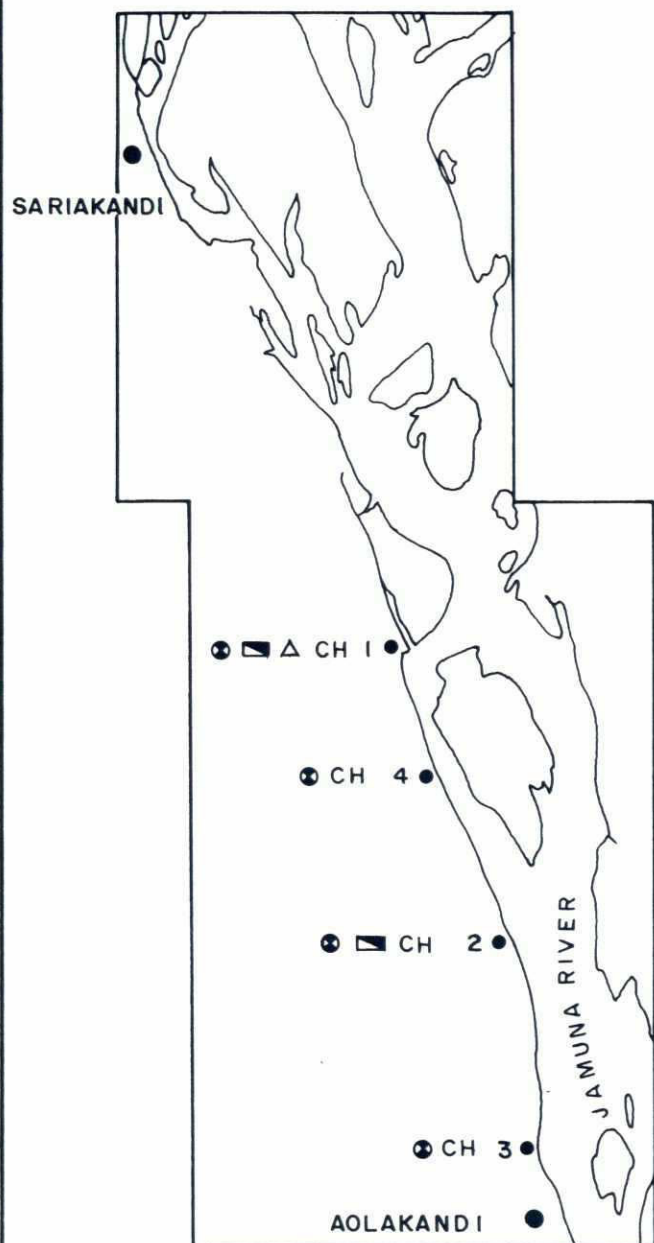
LEGEND

- BA 1 - Bahadurabad Location 1
 GA 1 - Kamarjani Location 1 (Gaibanda area)
 ⊗ - Standard Penetration test
 ⊞ - Cone Penetrometer test
 △ - Piezometer installation

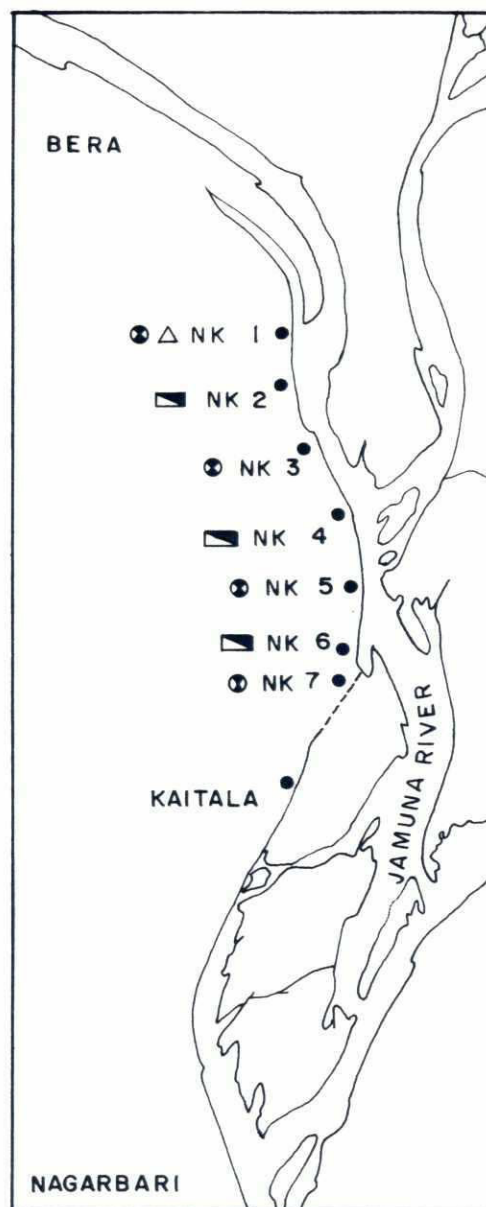
**LOCATION MAP OF SOIL
EXPLORATORY WORK**
(BASED ON SPOT IMAGE OF 1989)

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INTERIM REPORT **FIG. A8 - I**



CHANDANBAISA



NAKALIA

LEGEND

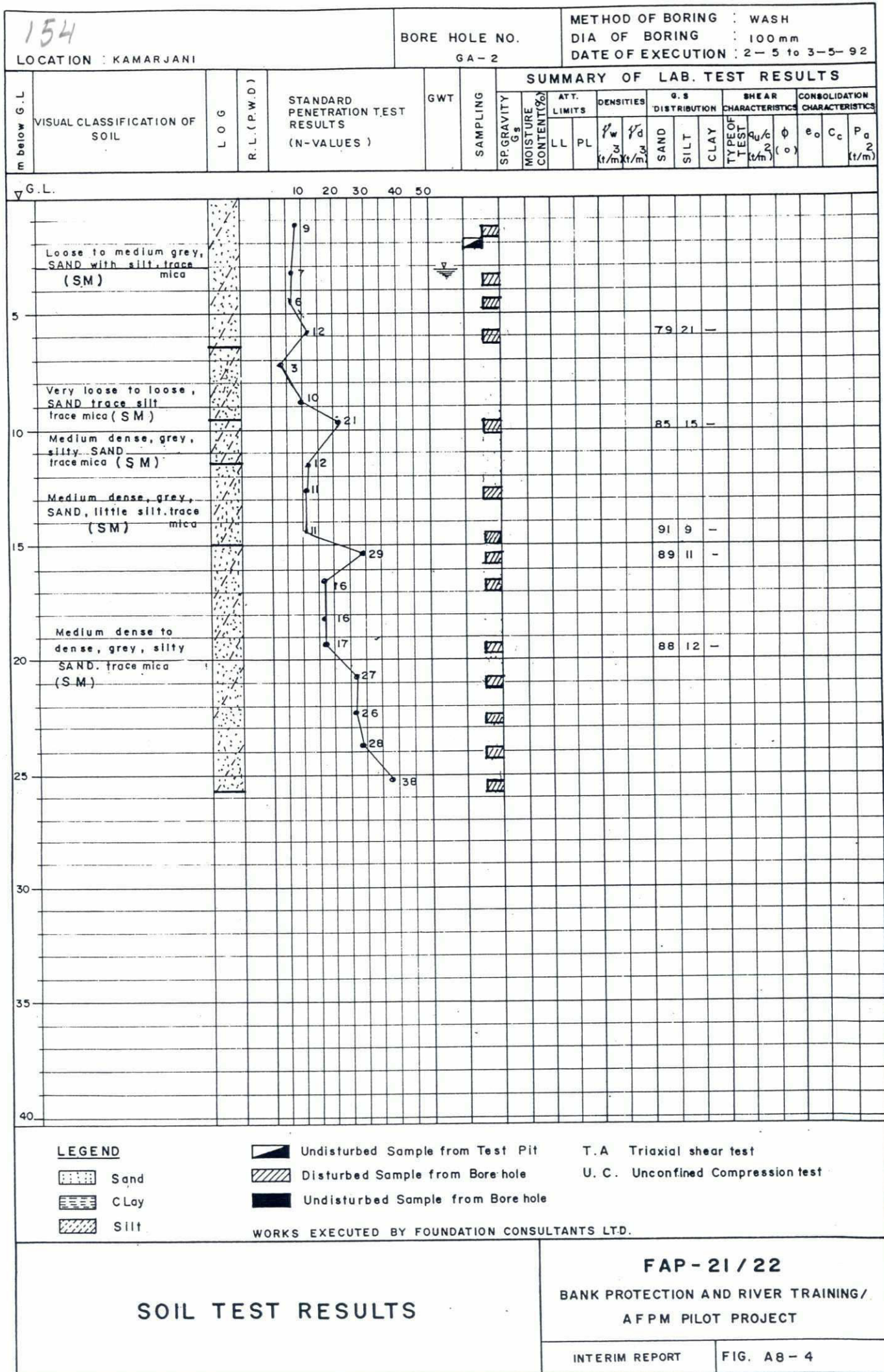
- CH 1 - Chandanbaisa Location 1
 ⊗ - Standard Penetration test
 ▢ - Cone Penetrometer test
 △ - Piezometer installation

**LOCATION MAP OF SOIL
EXPLORATORY WORK**
 (BASED ON SPOT IMAGE OF 1989)

FAP-21/22
 BANK PROTECTION AND RIVER TRAINING/
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INTERIM REPORT

FIG. A8 - 2



LOCATION : KAMARJANI			BORE HOLE NO. GA-4		METHOD OF BORING : WASH																			
					DIA OF BORING : 100 mm.																			
					DATE OF EXECUTION : 4-5 to 5-5-92																			
SUMMARY OF LAB. TEST RESULTS																								
m below G.L.	VISUAL CLASSIFICATION OF SOIL	LOG	R.L. (P.W.D.)	STANDARD PENETRATION TEST RESULTS (N-VALUES)	GWT	SAMPLING	SP. GRAVITY G_s		MOISTURE CONTENT (%)	ATT. LIMITS		DENSITIES		G.S. DISTRIBUTION			SHEAR CHARACTERISTICS		CONSOLIDATION CHARACTERISTICS					
							LL	PL		γ_w	γ_d	SAND	SILT	CLAY	TYPE OF TEST	$q_u/6$	ϕ	e_o	C_c	P_a				
							γ_w	γ_d																
							(t/m ³)	(t/m ³)																
V GL										10	20	30	40	50										
	Loose, brownish grey, SAND with silt, trace mica (SM)													72	28	-								
5	Loose to medium dense, brownish grey, silty SAND, trace mica (SM)													85	15	-								
														78	22	-								
10	Medium dense, brownish grey, SAND with silt trace mica (SM)																							
15	Medium dense, brownish grey, silty SAND, trace mica (SM)													86	14	-								
20	Dense, brownish grey, SAND little silt, trace mica (SM)													92	8	-								
25																								
30																								
35																								
40																								

LEGEND

Sand

Clay

Silt

Undisturbed Sample from Test Pit

Disturbed Sample from Bore hole

Undisturbed Sample from Bore hole

T.A

Triaxial shear test

U.C.

Unconfined Compression test

WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

SOIL TEST RESULTS

FAP-21/22

BANK PROTECTION AND RIVER TRAINING/
AFPM PILOT PROJECT

INTERIM REPORT

FIG. A8-5

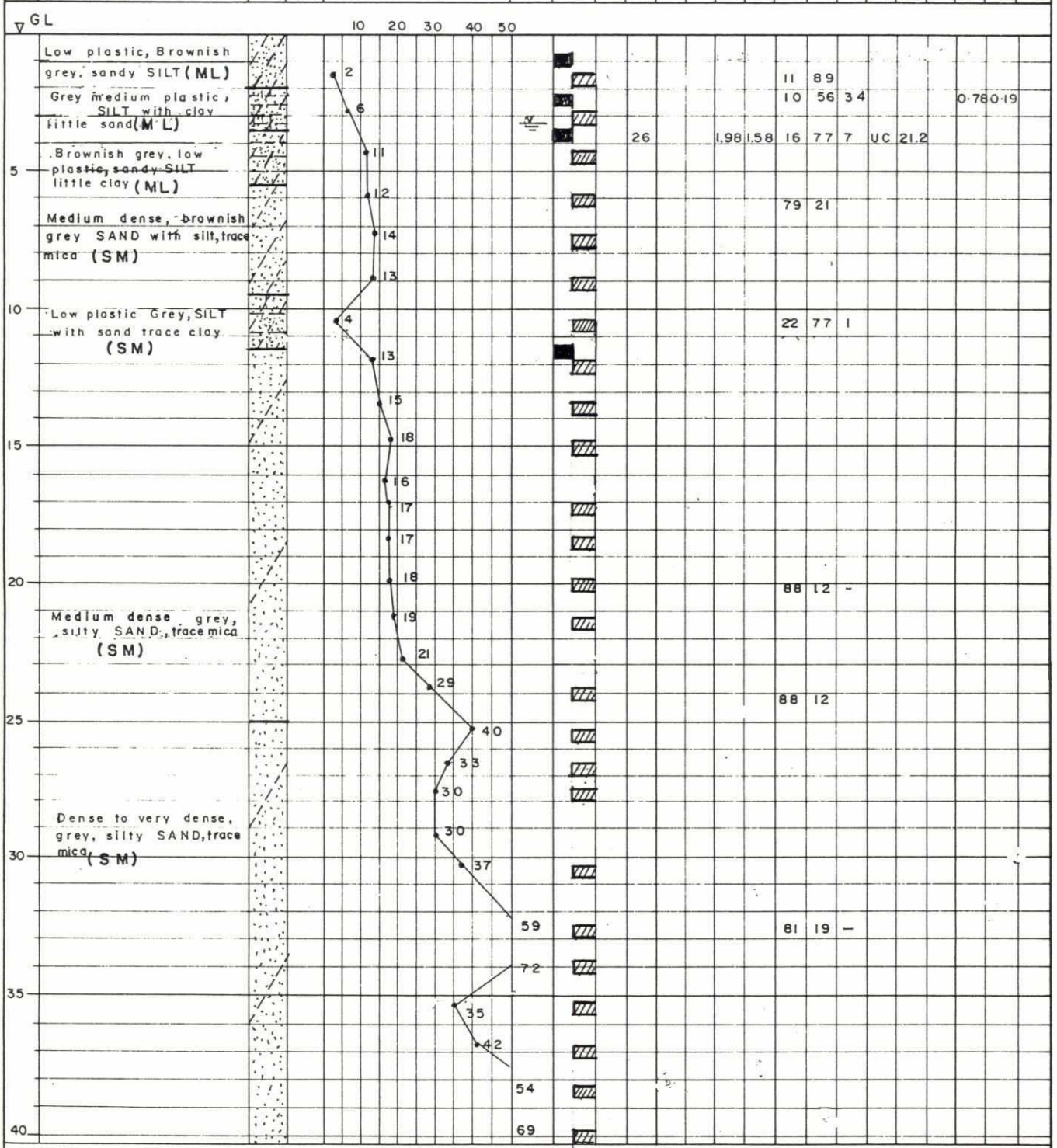
DATE OF EXECUTION : 3-5 to 5-5-92

FIG. A8-6

SOIL TEST RESULTS

LOCATION : BAHADURABAD	BORE HOLE NO. BA - I	METHOD OF BORING : WASH DIA OF BORING : 100 mm. DATE OF EXECUTION : 8-5 to 9-5-92
------------------------	-------------------------	---

m below G.L	VISUAL CLASSIFICATION OF SOIL	LOG	R.L.(P.W.D.)	STANDARD PENETRATION TEST RESULTS (N-VALUES)	GWT	SAMPLING	SUMMARY OF LAB. TEST RESULTS														
							SP.GRAVITY G _s	MOISTURE CONTENT(%)	ATT. LIMITS		DENSITIES			G.S DISTRIBUTION			SHEAR CHARACTERISTICS		CONSOLIDATION CHARACTERISTICS		
									LL	PL	P _w g/cm ³	P _d g/cm ³	SAND	SILT	CLAY	TYPE OF TEST	q _u /c (t/m ²)	φ (°)	e _o	C _c	P _{a2} (t/m ²)



LEGEND

- Sand
- Clay
- Silt

- Undisturbed Sample from Test Pit
- Disturbed Sample from Bore hole
- Undisturbed Sample from Bore hole

- T.A. Triaxial shear test
- U.C. Unconfined Compression test

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SOIL TEST RESULTS

FAP-21/22

BANK PROTECTION AND RIVER TRAINING/
AFPM PILOT PROJECT

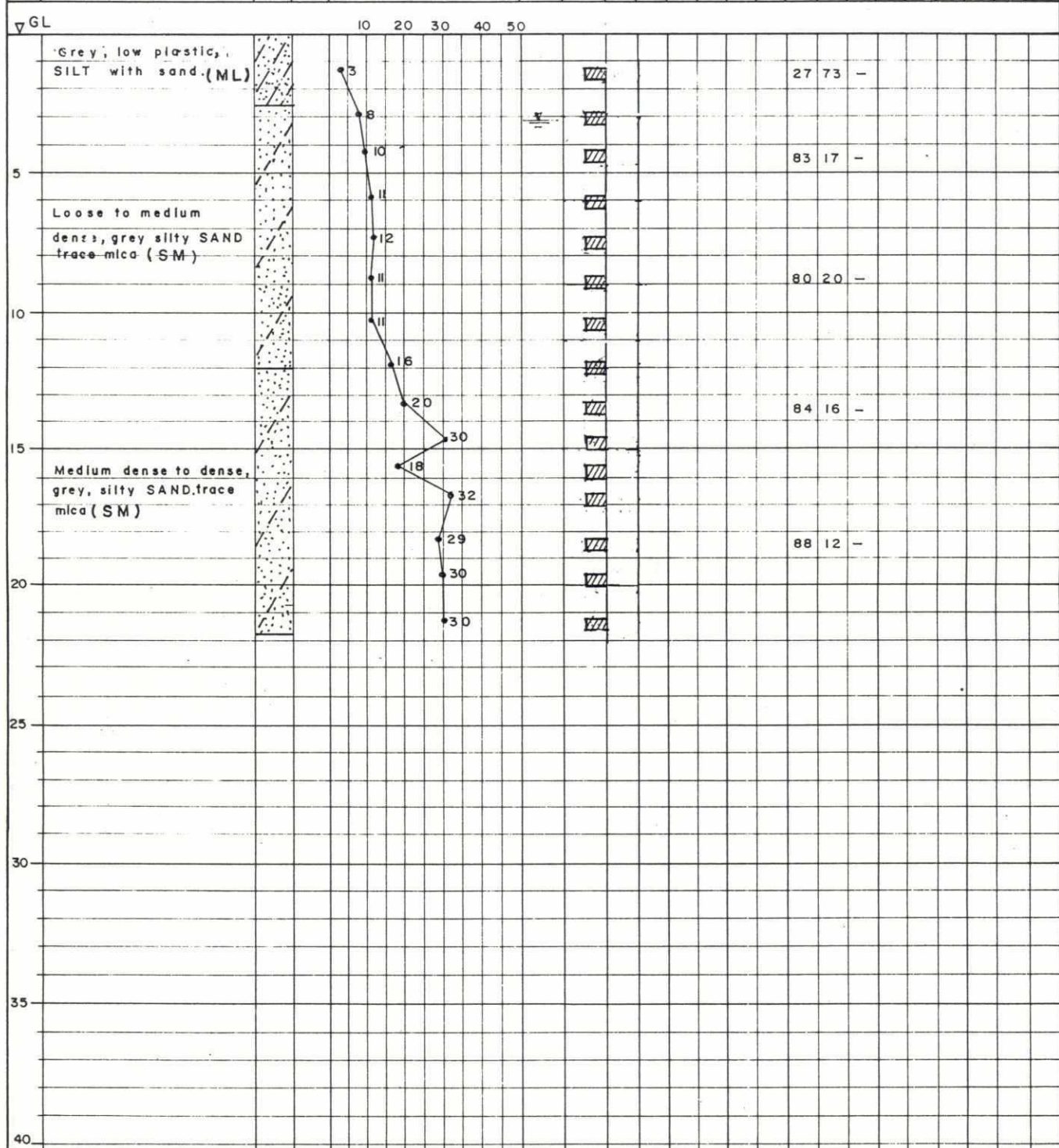
INTERIM REPORT

FIG. A8-7

FIG. A8-8

LOCATION : BAHADURABAD	BORE HOLE NO. BA - 4	METHOD OF BORING : WASH DIA OF BORING : 100 mm. DATE OF EXECUTION : 10-5 to 11-5-92
------------------------	-------------------------	---

m below G.L	VISUAL CLASSIFICATION OF SOIL	LOG	R.L.(P.W.D.)	STANDARD PENETRATION TEST RESULTS (N-VALUES)	GWT	SAMPLING	SUMMARY OF LAB. TEST RESULTS														
							SP.GRAVITY G _s	MOISTURE CONTENT(%)	ATT. LIMITS		DENSITIES		G. S DISTRIBUTION			SHEAR CHARACTERISTICS			CONSOLIDATION CHARACTERISTICS		
									LL	PL	γ_w (t/m ³)	γ_d (t/m ³)	SAND	SILT	CLAY	TYPE OF TEST	q_u/c (t/m ²)	ϕ (°)	e_o	C_c	P_a (t/m ²)



LEGEND

- Sand
- Clay
- Silt

- Undisturbed Sample from Test Pit
- Disturbed Sample from Bore hole
- Undisturbed Sample from Bore hole

- T.A Triaxial shear test
- U.C. Unconfined Compression test

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SOIL TEST RESULTS

FAP - 21 / 22

BANK PROTECTION AND RIVER TRAINING/
AFPM PILOT PROJECT

INTERIM REPORT

FIG. A8 - 9

LOCATION : BAHADURABAD

BORE HOLE NO.
BA-6

METHOD OF BORING	: WASH
DIA OF BORING	: 100 mm
DATE OF EXECUTION	: 12-5 to 13-5-92

[illegible]

LEGEND

	Sand
	Clay
	Silt

 Undisturbed Sample from Test Pit
 Disturbed Sample from Bore hole
 Undisturbed Sample from Bore hole

T. A. Triaxial shear test
U. C. Unconfined Compression test

WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

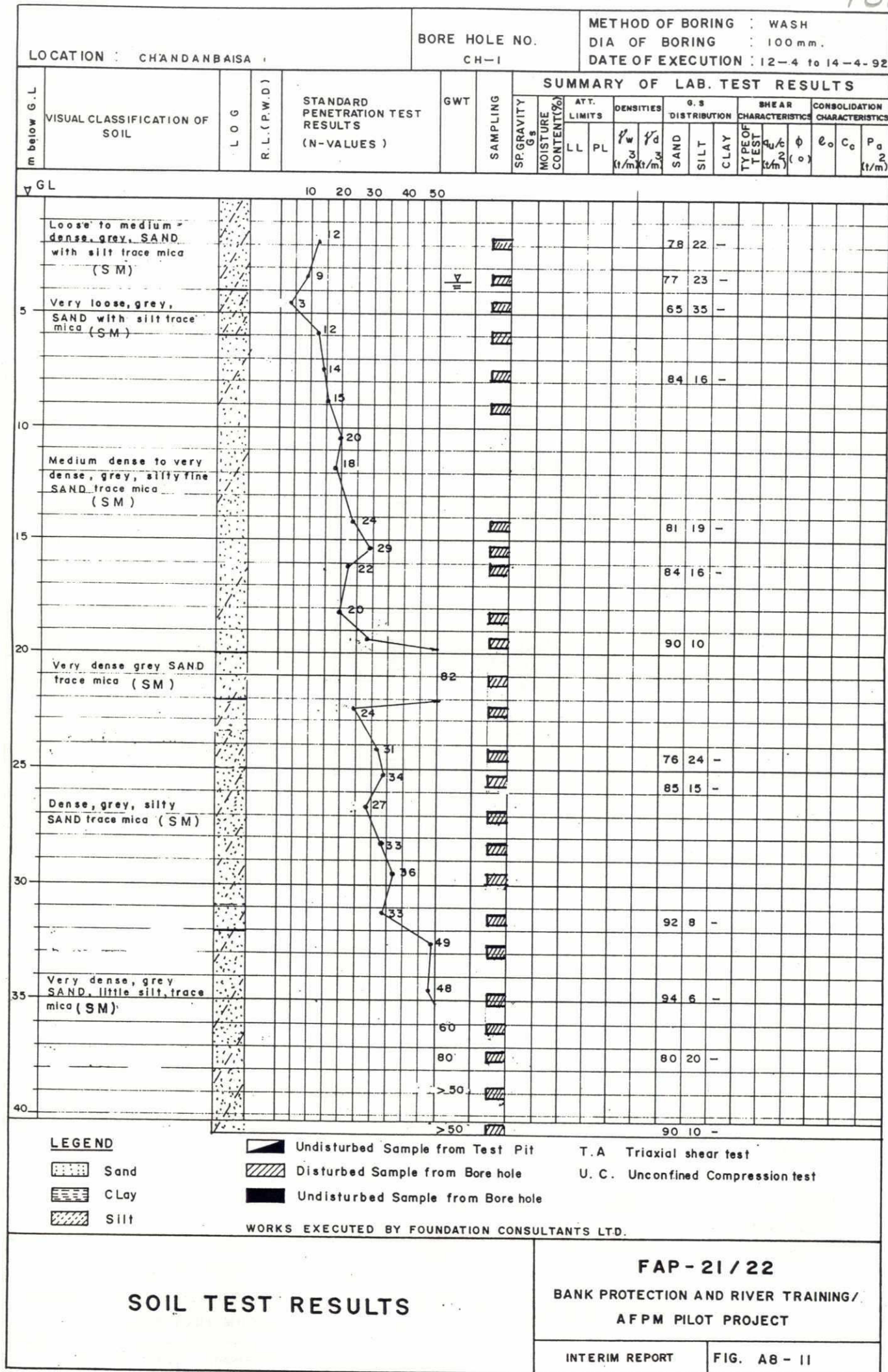
SOIL TEST RESULTS

FAP - 21 / 22

BANK PROTECTION AND RIVER TRAINING/
AFPM PILOT PROJECT

INTERIM REPORT

FIG. A8-10



LOCATION : CHANDANBAISA

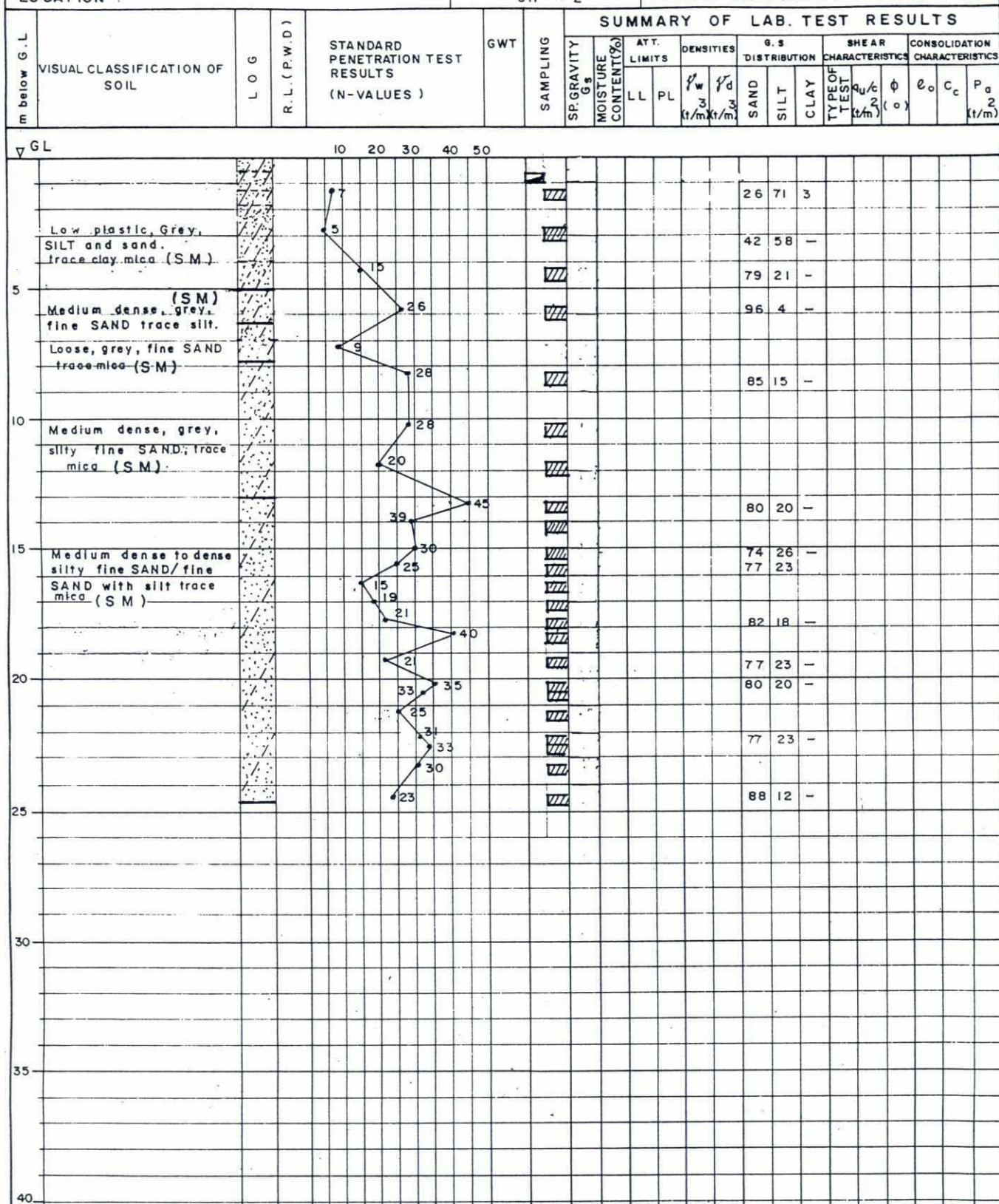
BORE HOLE NO.

CH - 2

METHOD OF BORING : WASH

DIA OF BORING : 100 mm.

DATE OF EXECUTION : 14-4 to 15-4-92



LEGEND

 Sand C Lay Silt

Undisturbed Sample from Test Pit

 Disturbed Sample from Bore hole

Undisturbed Sample from Bore hole

T.A Triaxial shear test.

U. C. Unconfined Compression test

WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

SOIL TEST RESULTS

FAP - 21 / 22

BANK PROTECTION AND RIVER TRAINING/
AFPM PILOT PROJECT

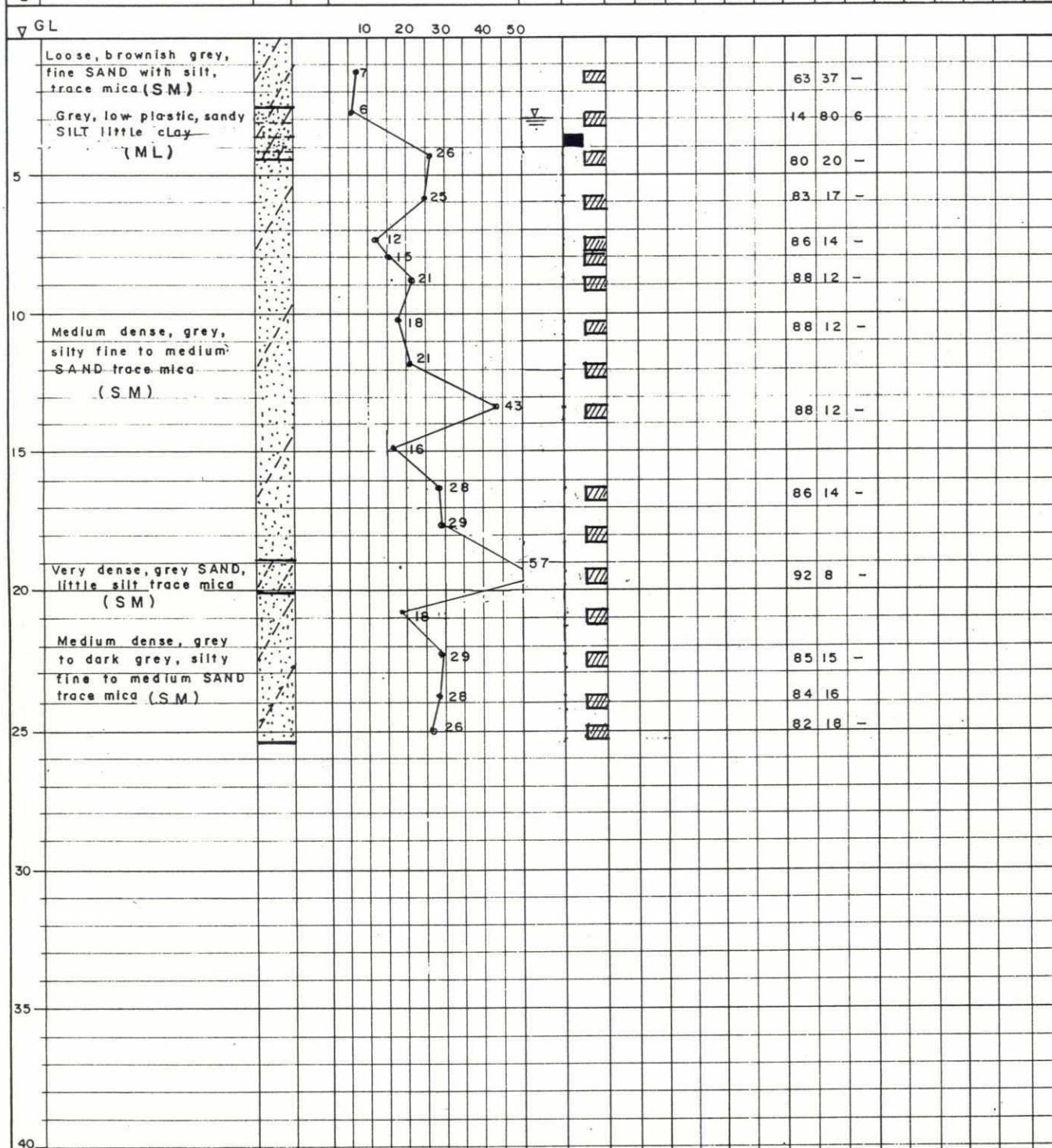
INTERIM REPORT

FIG. A8-12

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LOCATION : CHANDANBAISA.		BORE HOLE NO. CH - 3	METHOD OF BORING : WASH DIA OF BORING : 100mm. DATE OF EXECUTION : 22-4 to 23-4-92
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m below G.L	VISUAL CLASSIFICATION OF SOIL	LOG	R.L.(P.W.D.)	STANDARD PENETRATION TEST RESULTS (N-VALUES)	GWT	SAMPLING	SUMMARY OF LAB. TEST RESULTS													
							SP.GRAVITY G _s	MOISTURE CONTENT(%)	ATT. LIMITS		DENSITIES		G.S DISTRIBUTION			SHEAR CHARACTERISTICS		CONSOLIDATION CHARACTERISTICS		
									LL	PL	ρ_w (t/m ³)	ρ_d (t/m ³)	SAND	SILT	CLAY	TYPE OF TEST	q_u (t/m ²)	ϕ (°)	e_o	C_c



SOIL TEST RESULTS	FAP-21/22	
	BANK PROTECTION AND RIVER TRAINING/ AFPM PILOT PROJECT	
	INTERIM REPORT	FIG. A8-13

164 LOCATION : CHANDANBAISA		BORE HOLE NO. CH-4		METHOD OF BORING : WASH DIA OF BORING : 100 mm DATE OF EXECUTION : 23-4 to 24-4-92															
m below G.L.	VISUAL CLASSIFICATION OF SOIL	LOG	R.L. (P.W.D.)	STANDARD PENETRATION TEST RESULTS (N-VALUES)	GWT	SAMPLING	SUMMARY OF LAB. TEST RESULTS												
							SP. GRAVITY G _s	MOISTURE CONTENT (%)	ATT. LIMITS		DENSITIES		G.S. DISTRIBUTION			SHEAR CHARACTERISTICS		CONSOLIDATION CHARACTERISTICS	
									LL	PL	γ_w g/cm ³	γ_d g/cm ³	SAND	SILT	CLAY	TYPE OF TEST	q_u/c (t/m ²)	ϕ (°)	e_o
GL. 10 20 30 40 50																			
	Loose, grey, fine SAND little silt, trace mica (SM)																		
	Low plastic, Grey, sandy SILT little clay (ML)																		
	Low plastic, Grey, SILT with sand trace clay (ML)																		
5	Loose, grey, silty fine SAND trace mica (SM)																		
	Loose to medium dense, grey fine SAND, little silt, trace mica (SM)																		
10	Medium dense, grey, silty SAND, trace mica. (SM)																		
	Medium dense, grey SAND little silt, trace mica (SM)																		
15	Loose, grey, fine SAND trace mica (SM)																		
	Medium dense, grey silty, SAND trace mica (SM)																		
20																			
25																			
30																			
35																			
40																			

LEGEND

Sand

CLay

Silt

Undisturbed Sample from Test Pit

Disturbed Sample from Bore hole

Undisturbed Sample from Bore hole

T.A Triaxial shear test

U. C. Unconfined Compression test

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SOIL TEST RESULTS

FAP-21/22

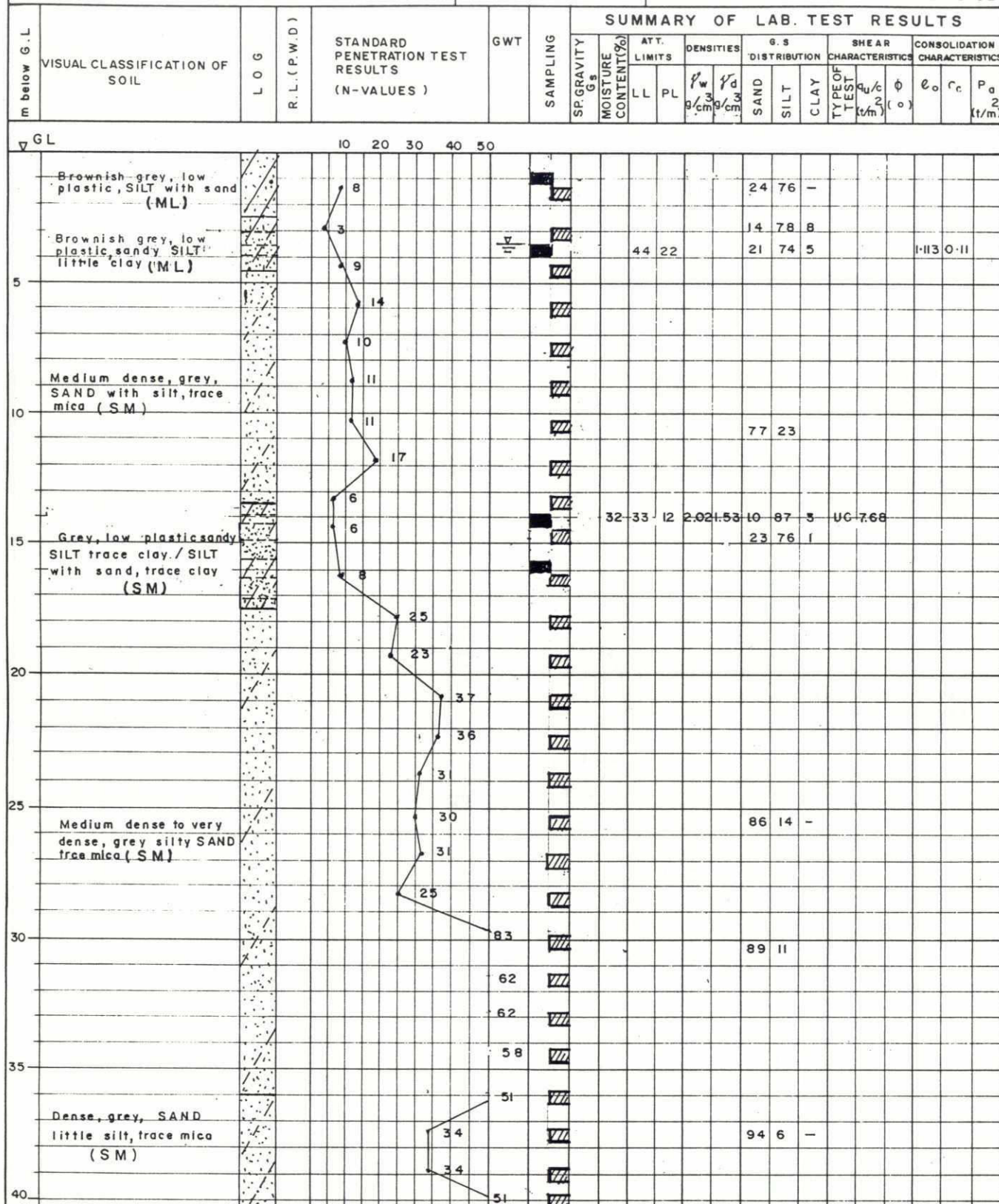
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AFPM PILOT PROJECT

INTERIM REPORT

FIG. A8-14

BORE HOLE NO.
NK-1

LOCATION : NAKALIA



LEGEND

	Sand
	Clay
	Silt

 Undisturbed Sample from Test Pit
 Disturbed Sample from Bore hole
 Undisturbed Sample from Bore hole

T. A. Triaxial shear test.
U. C. Unconfined Compression test

WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

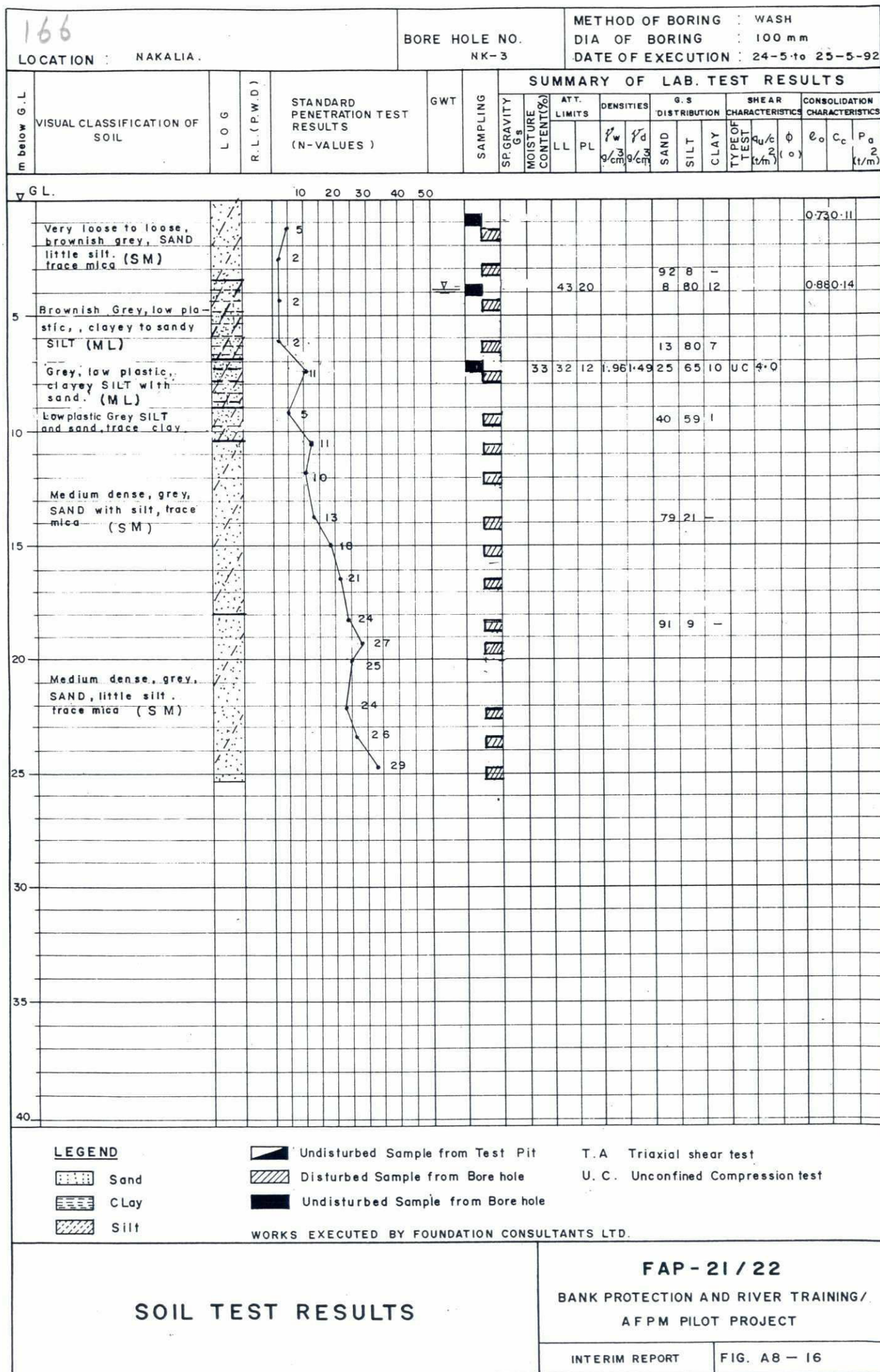
SOIL TEST RESULTS

FAP - 21 / 22

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AFPM PILOT PROJECT**

INTERIM REPORT

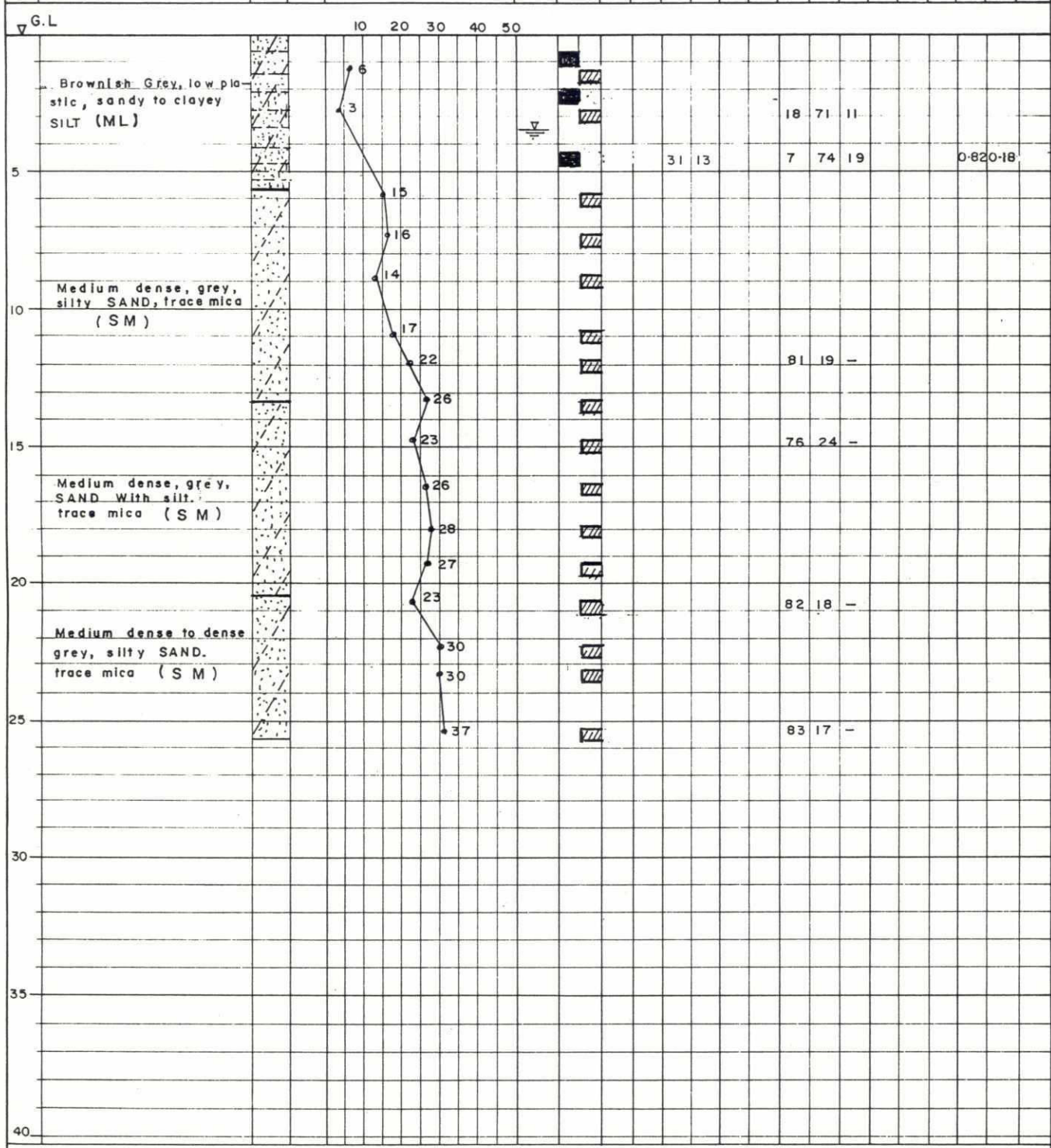
FIG. A8-15



167

LOCATION : NAKALIA		BORE HOLE NO. NK-5	METHOD OF BORING : WASH DIA OF BORING : 100 mm DATE OF EXECUTION : 27-5 to 28-5-92
--------------------	--	-----------------------	--

m below G.L	VISUAL CLASSIFICATION OF SOIL	LOG	R.L.(P.W.D.)	STANDARD PENETRATION TEST RESULTS (N-VALUES)	GWT	SAMPLING	SUMMARY OF LAB. TEST RESULTS													
							SP.GRAVITY G _s	MOISTURE CONTENT(%)	ATT. LIMITS		DENSITIES			G.S DISTRIBUTION			SHEAR CHARACTERISTICS		CONSOLIDATION CHARACTERISTICS	
									LL	PL	ρ_w (t/m ³)	ρ_d (t/m ³)	SAND	SILT	CLAY	TYPE OF TEST	$\tau_{u/c}$ (t/m ²)	ϕ (°)	e_o	C_c



LEGEND

Sand

CLay

Silt

Undisturbed Sample from Test Pit

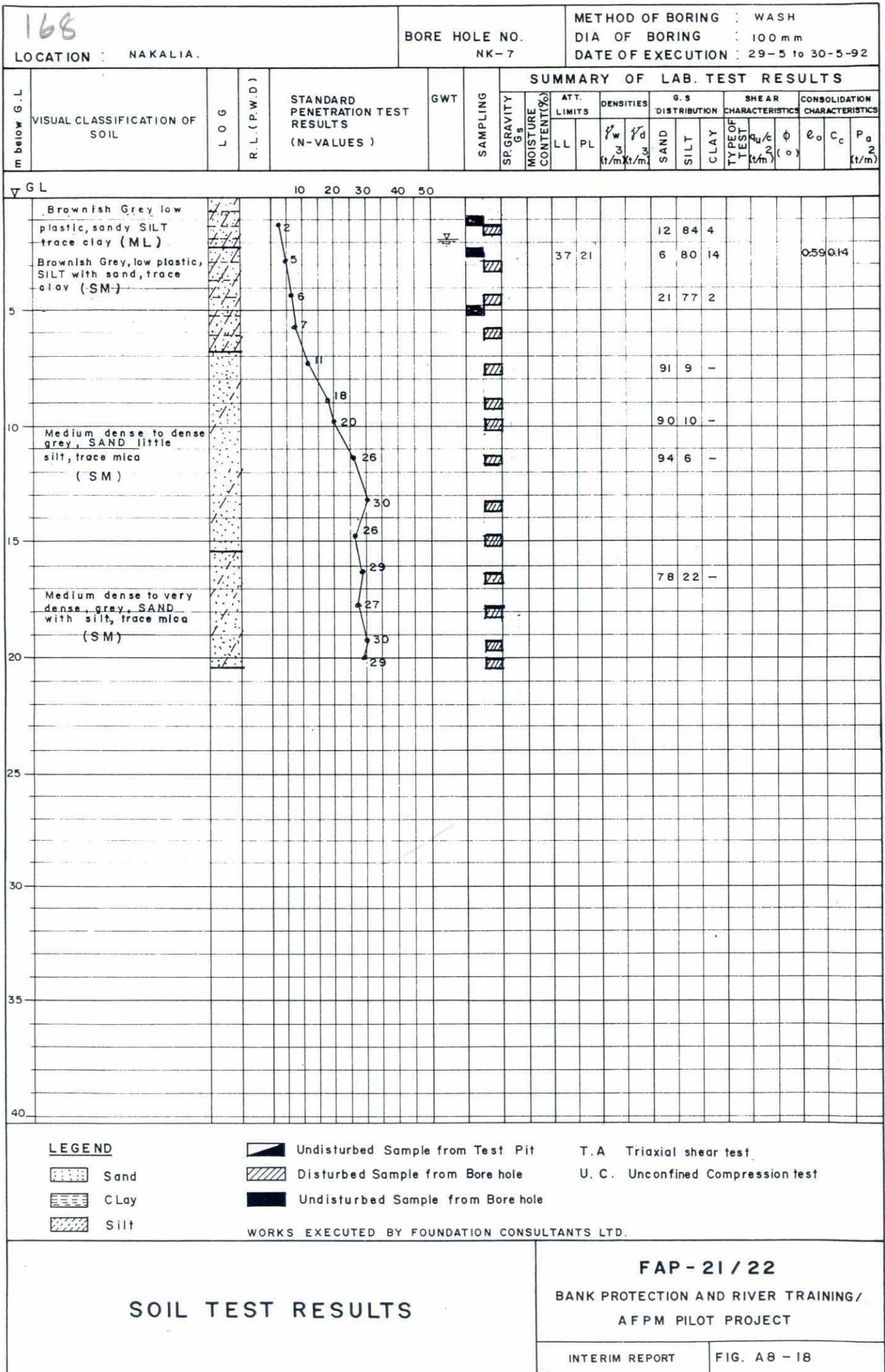
Disturbed Sample from Bore hole

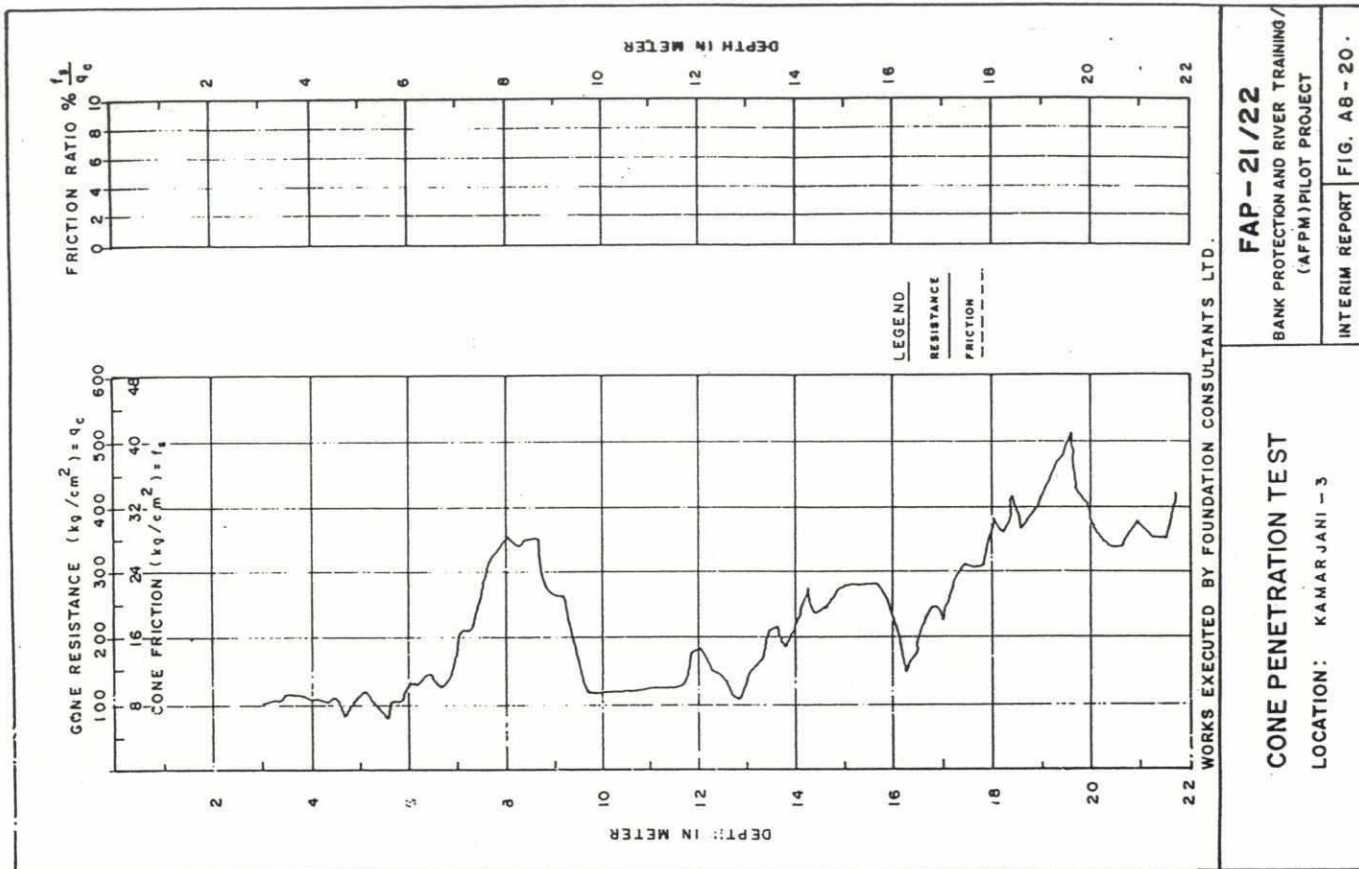
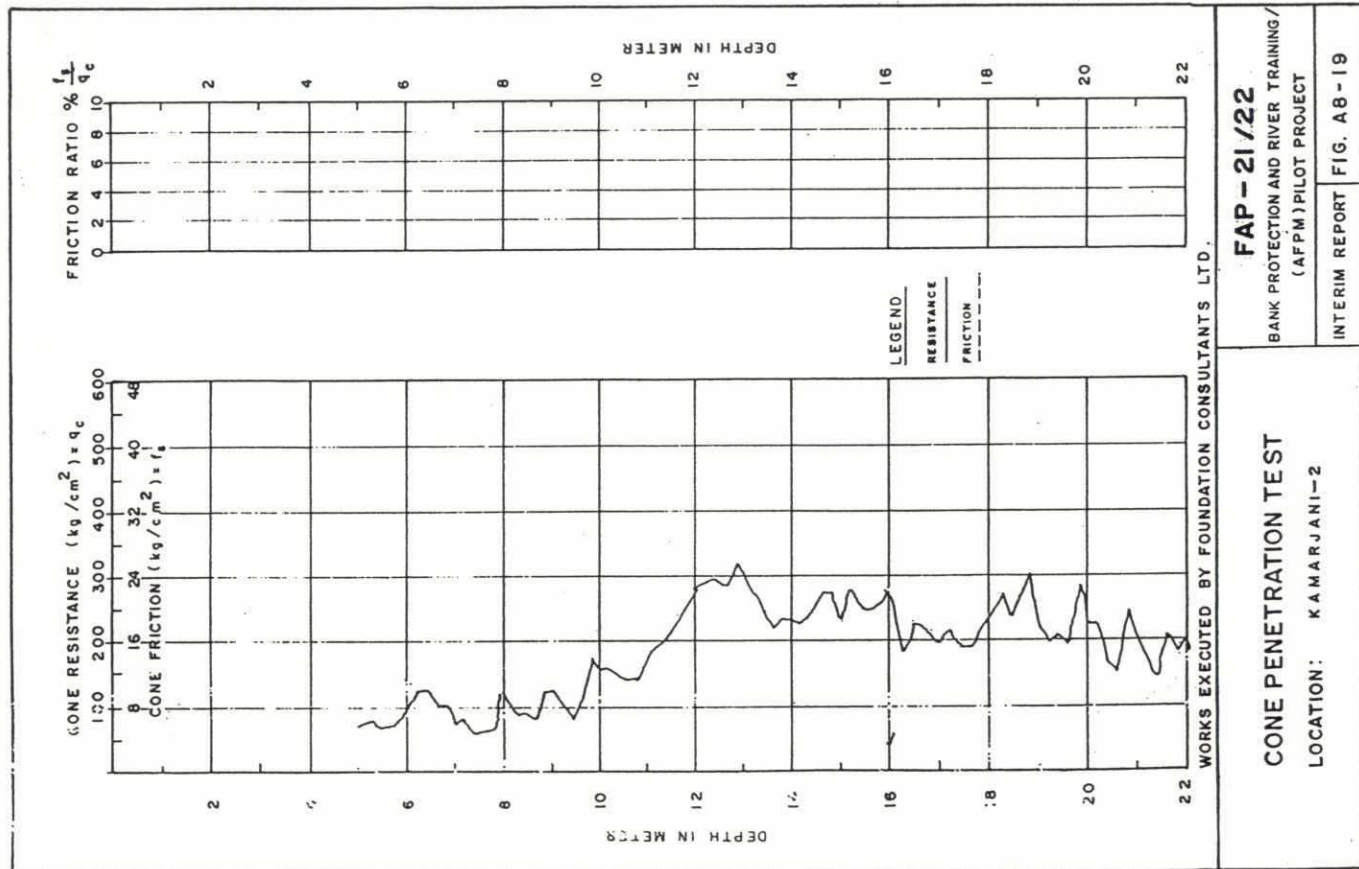
Undisturbed Sample from Bore hole

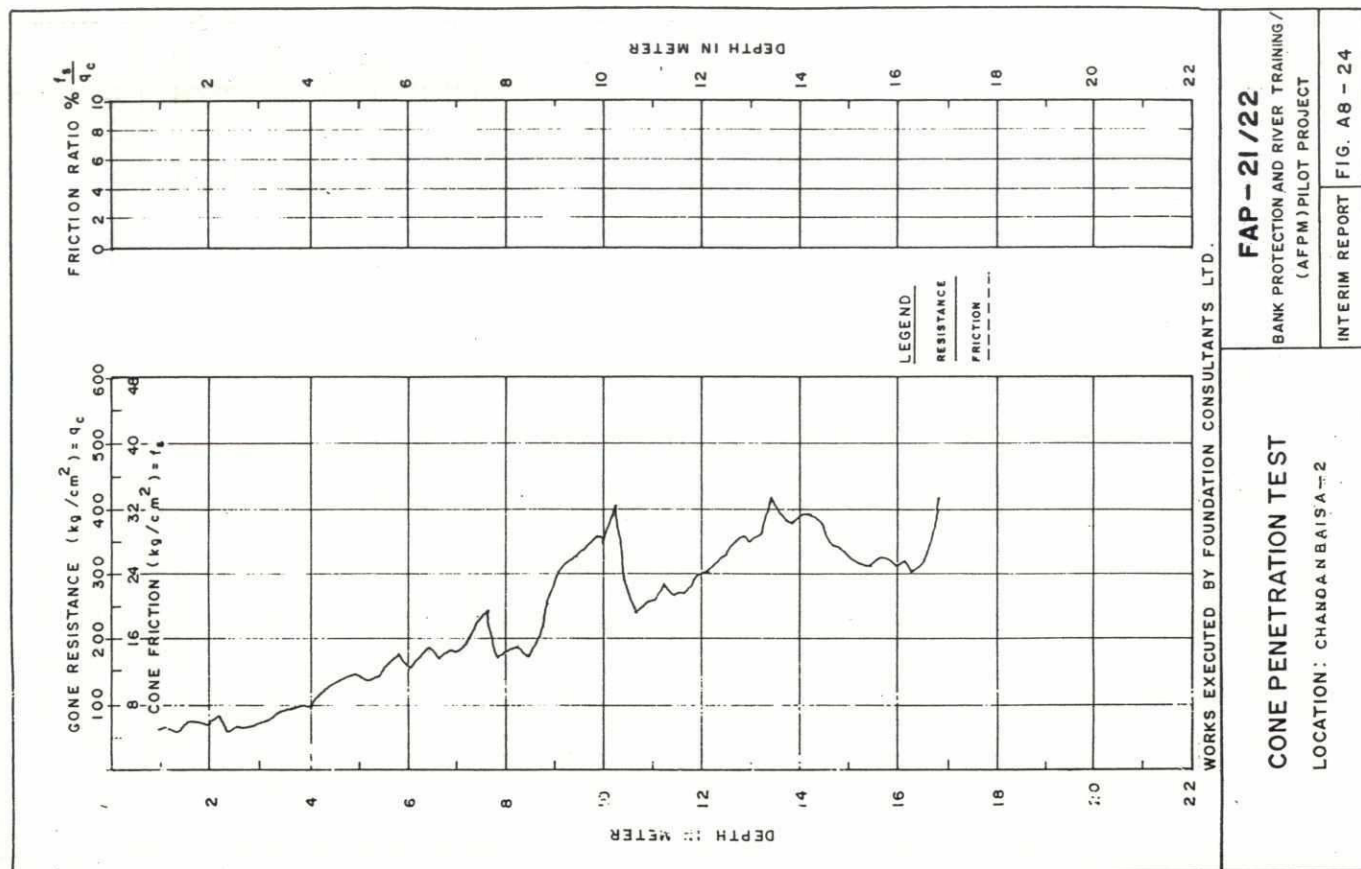
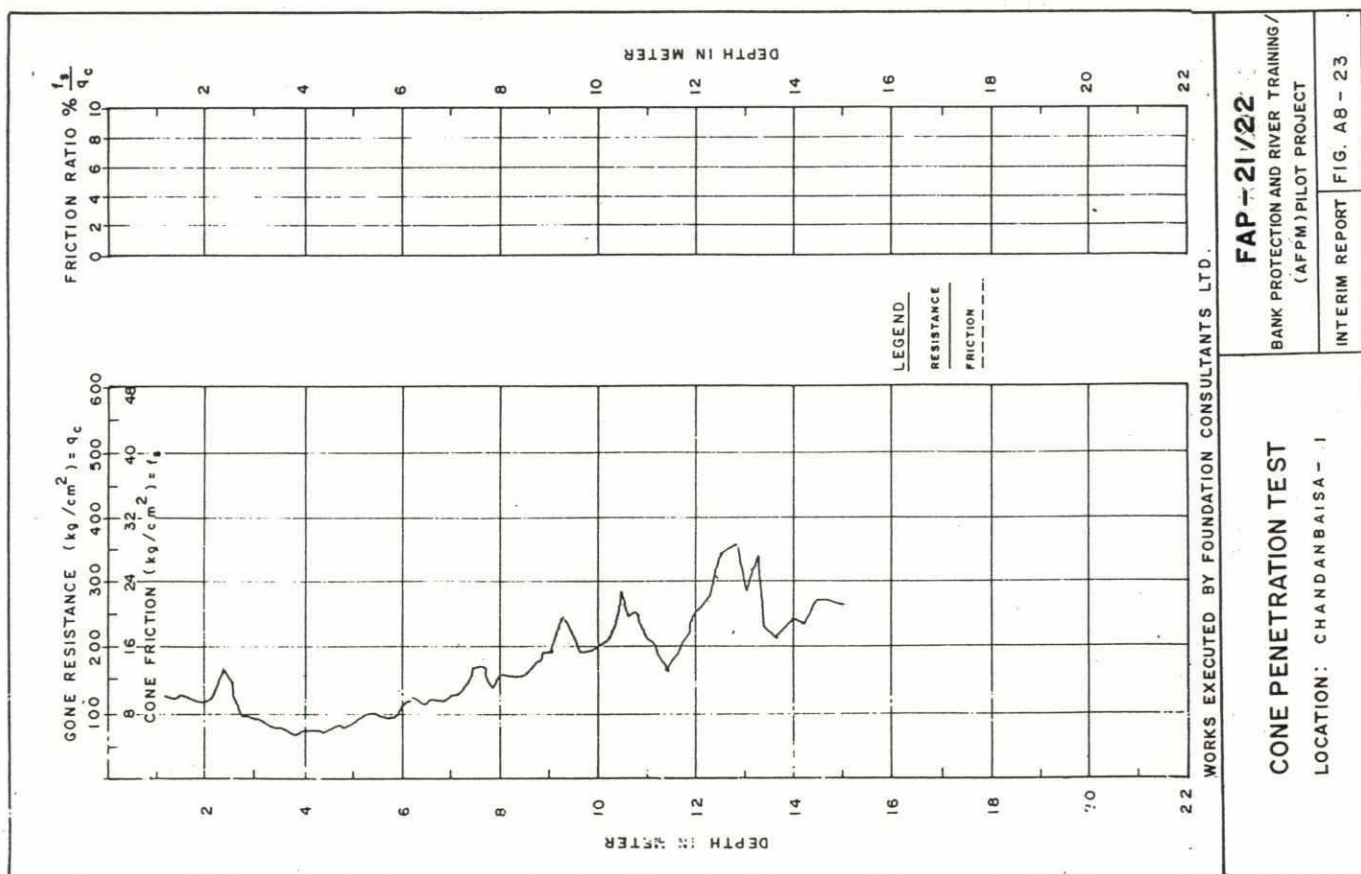
T.A Triaxial shear test
U.C. Unconfined Compression test

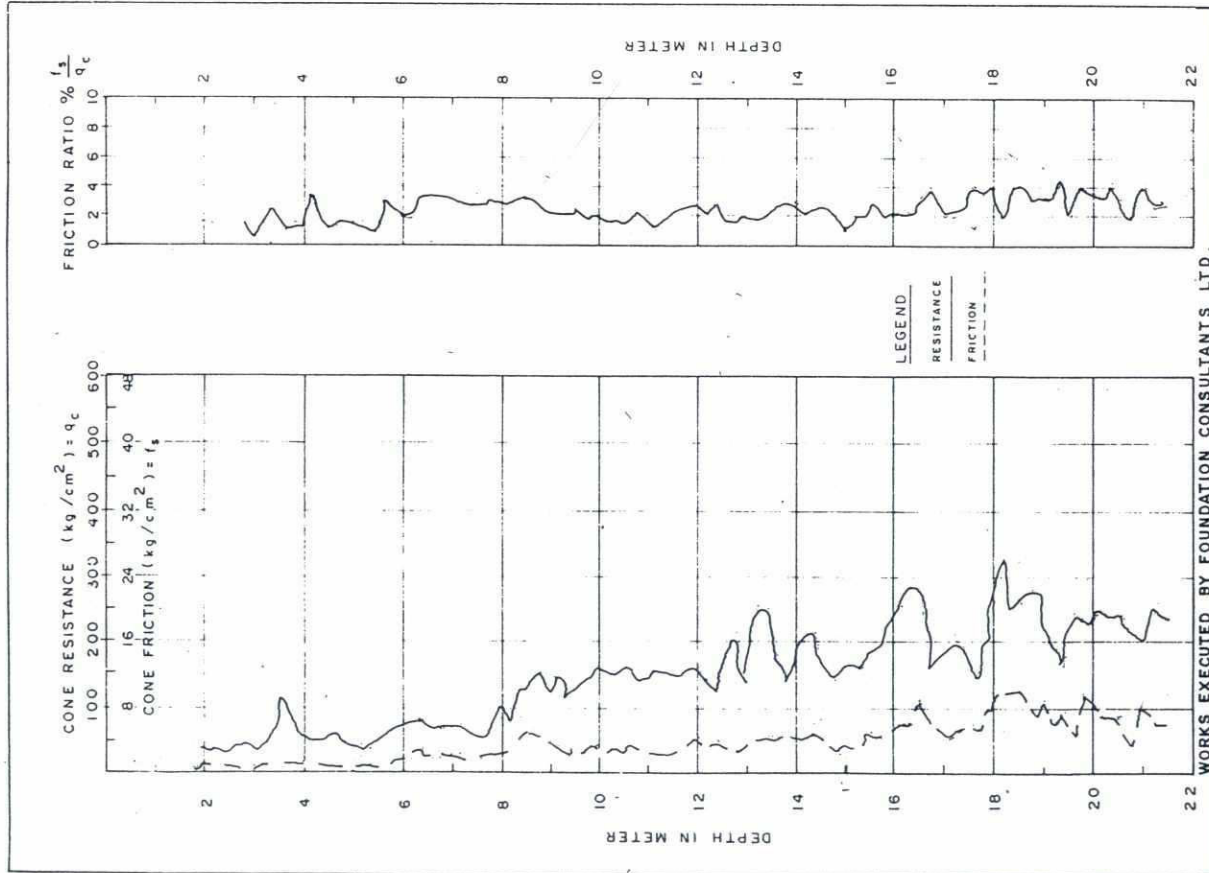
WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

SOIL TEST RESULTS	FAP - 21 / 22
	BANK PROTECTION AND RIVER TRAINING/ AFPM PILOT PROJECT
INTERIM REPORT	FIG. A 8 - 17

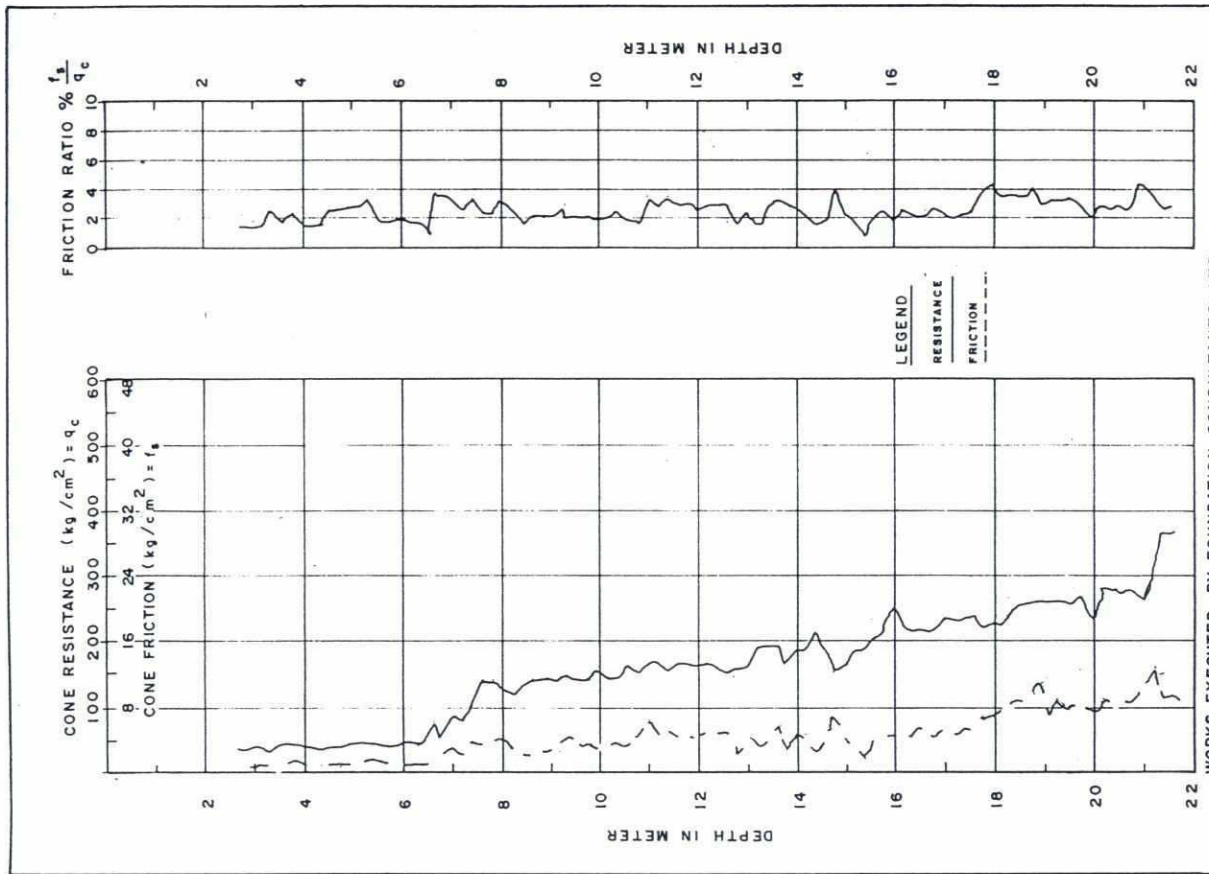




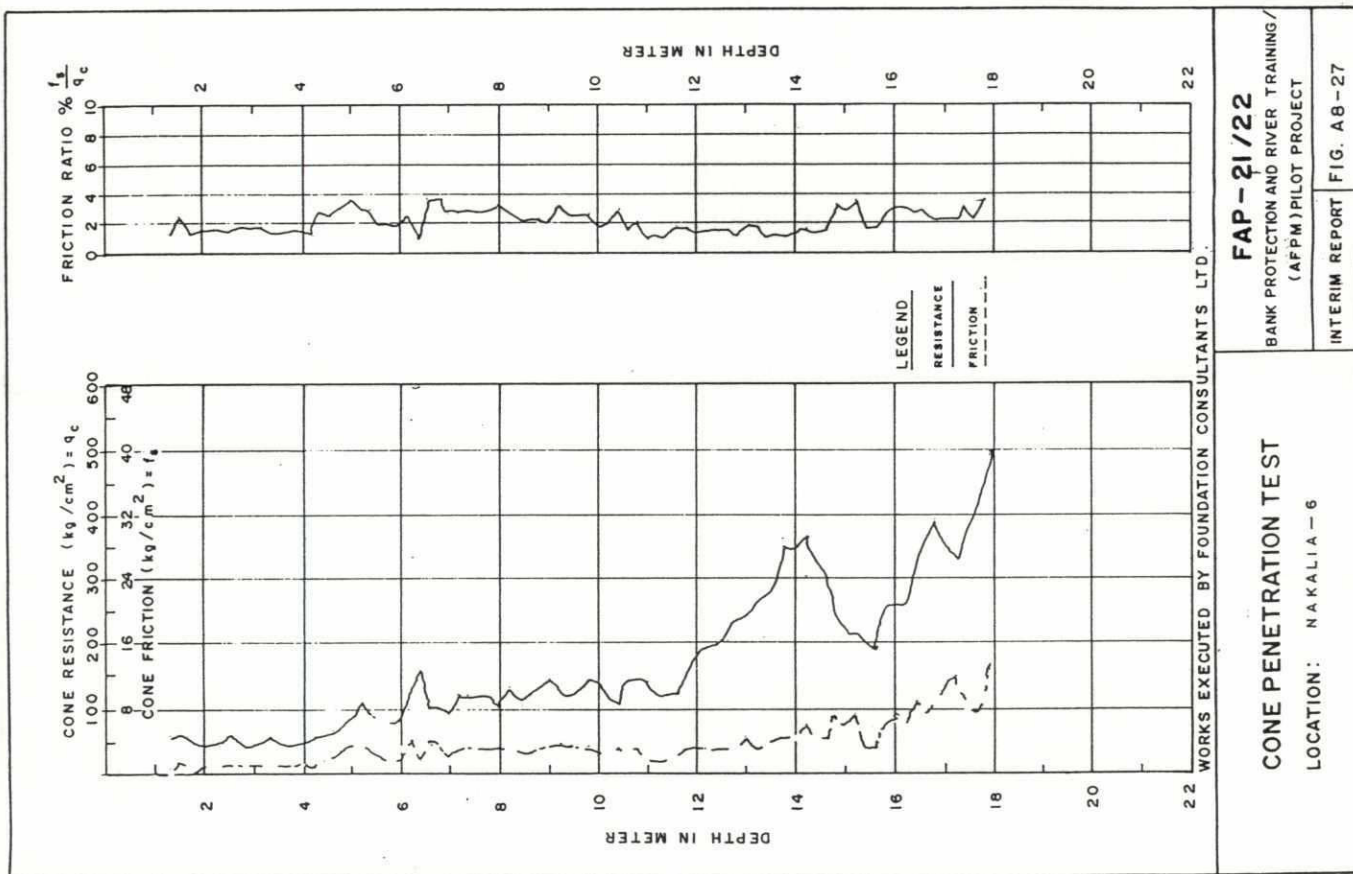




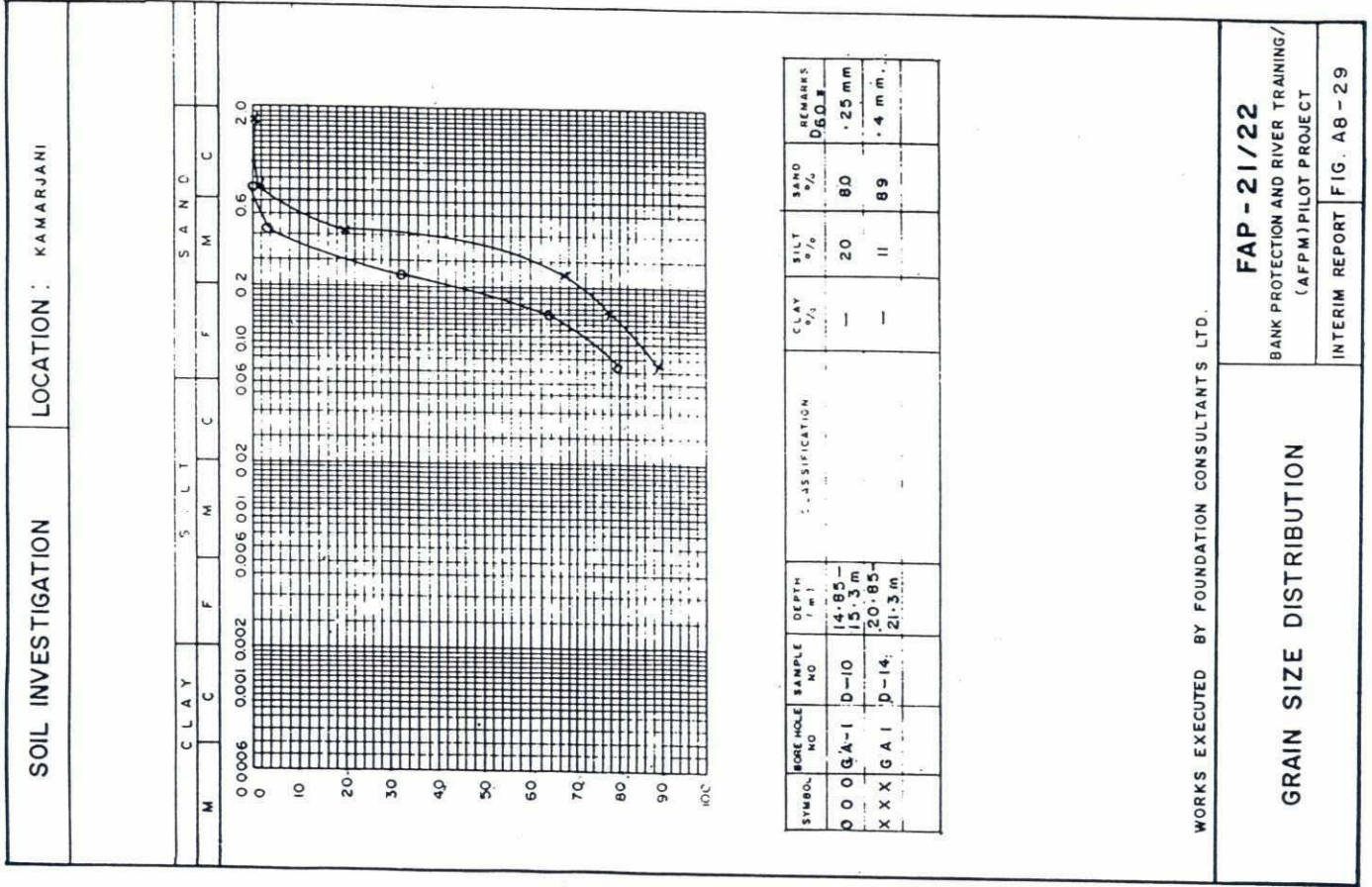
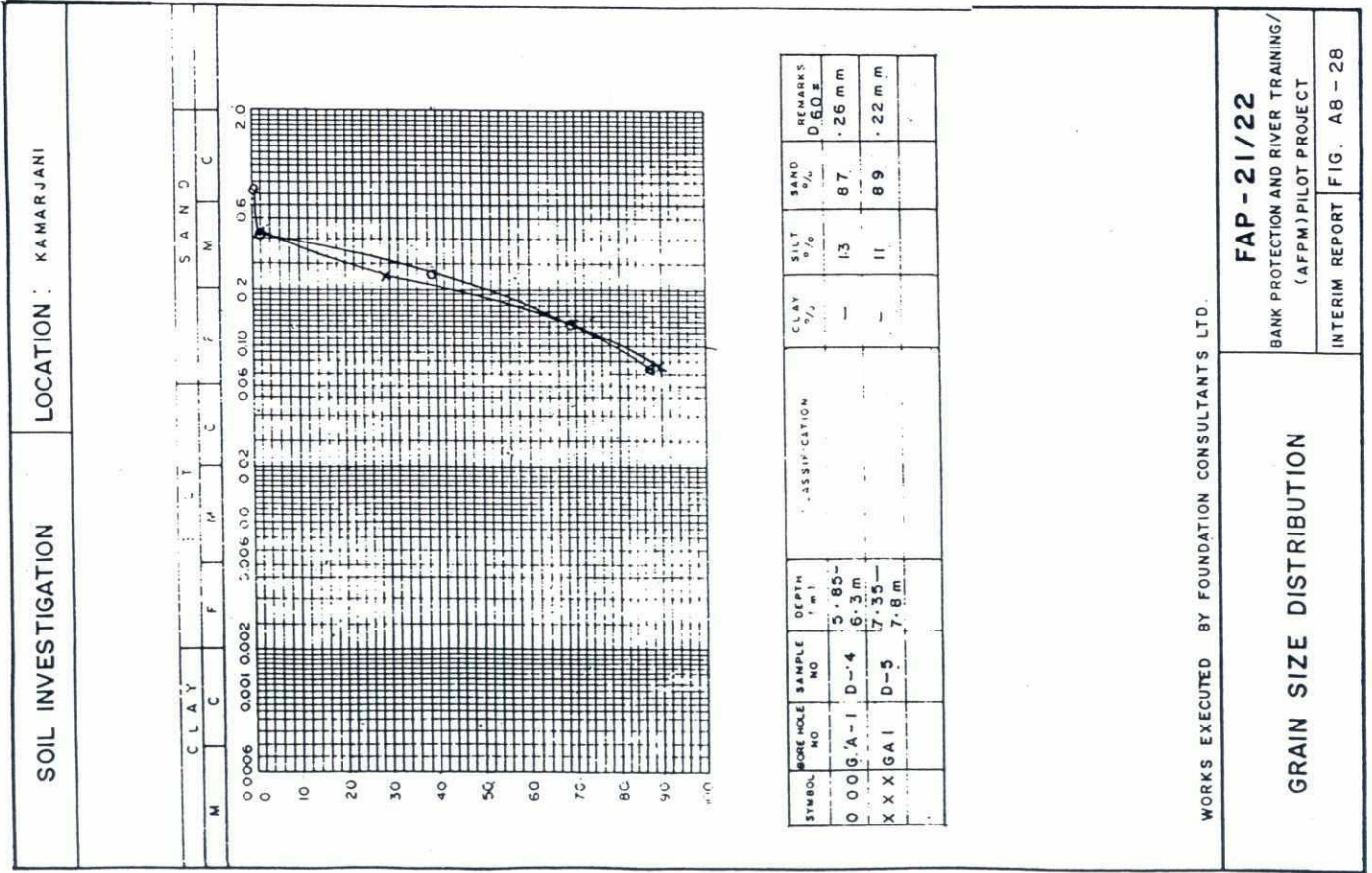
WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.	
CONE PENETRATION TEST	
LOCATION: NAKALIA - 2	
FAP - 21/22	
BANK PROTECTION AND RIVER TRAINING/ (AFPM) PILOT PROJECT	
INTERIM REPORT	FIG. AB - 25

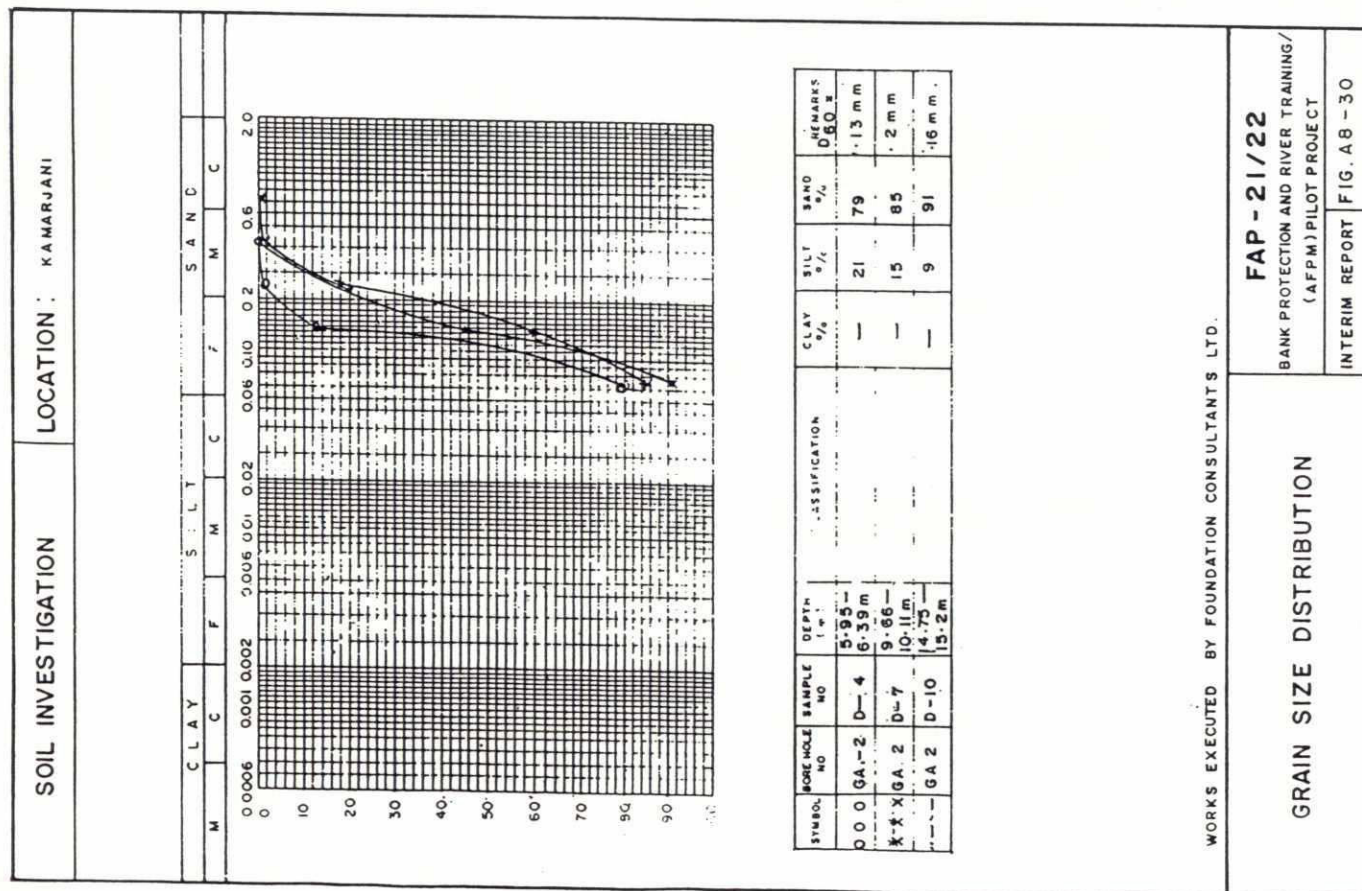


WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.	
CONE PENETRATION TEST	
LOCATION: NAKALIA - 4	
FAP - 21/22	
BANK PROTECTION AND RIVER TRAINING/ (AFPM) PILOT PROJECT	
INTERIM REPORT	FIG. AB - 26



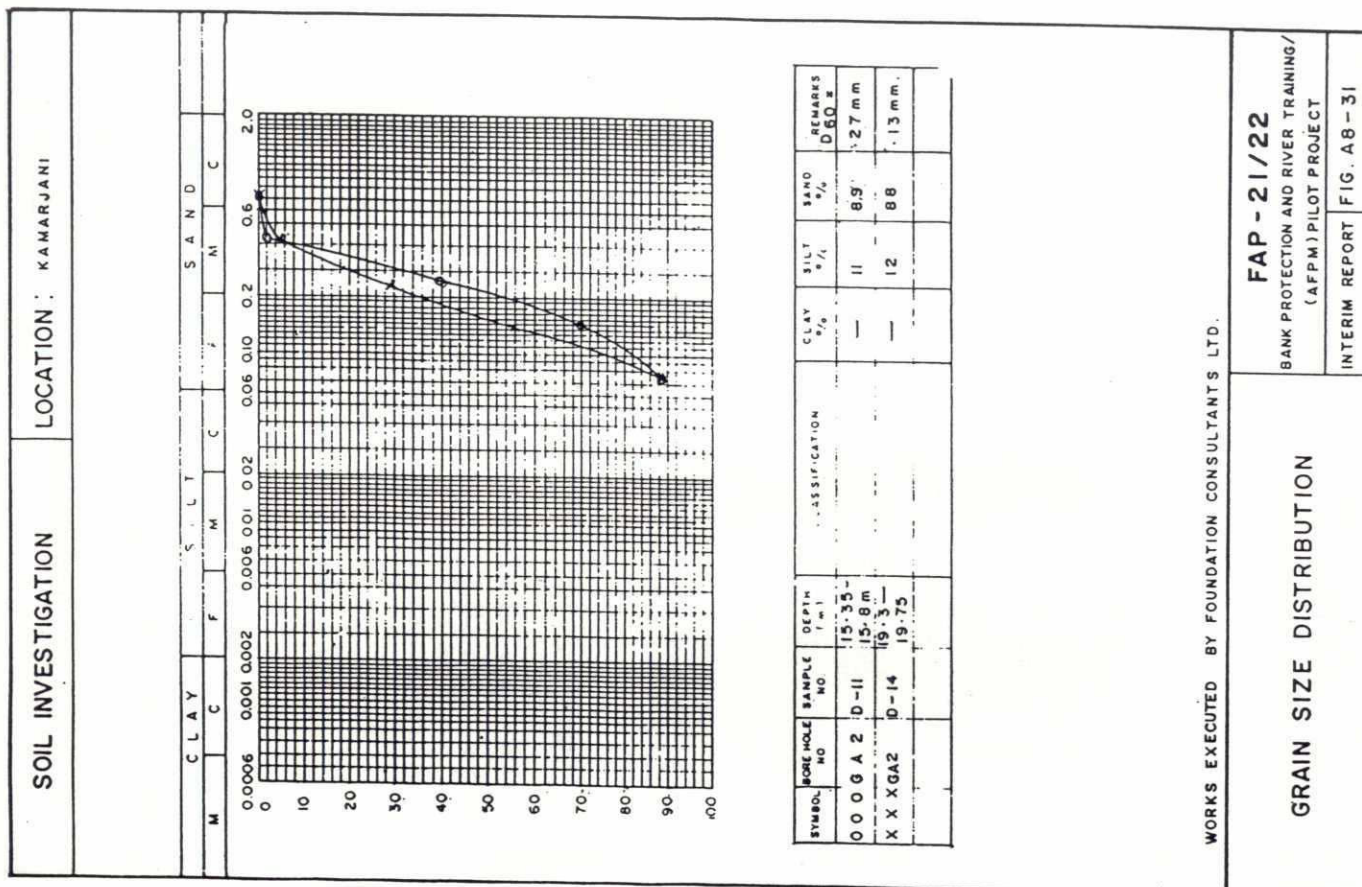
174





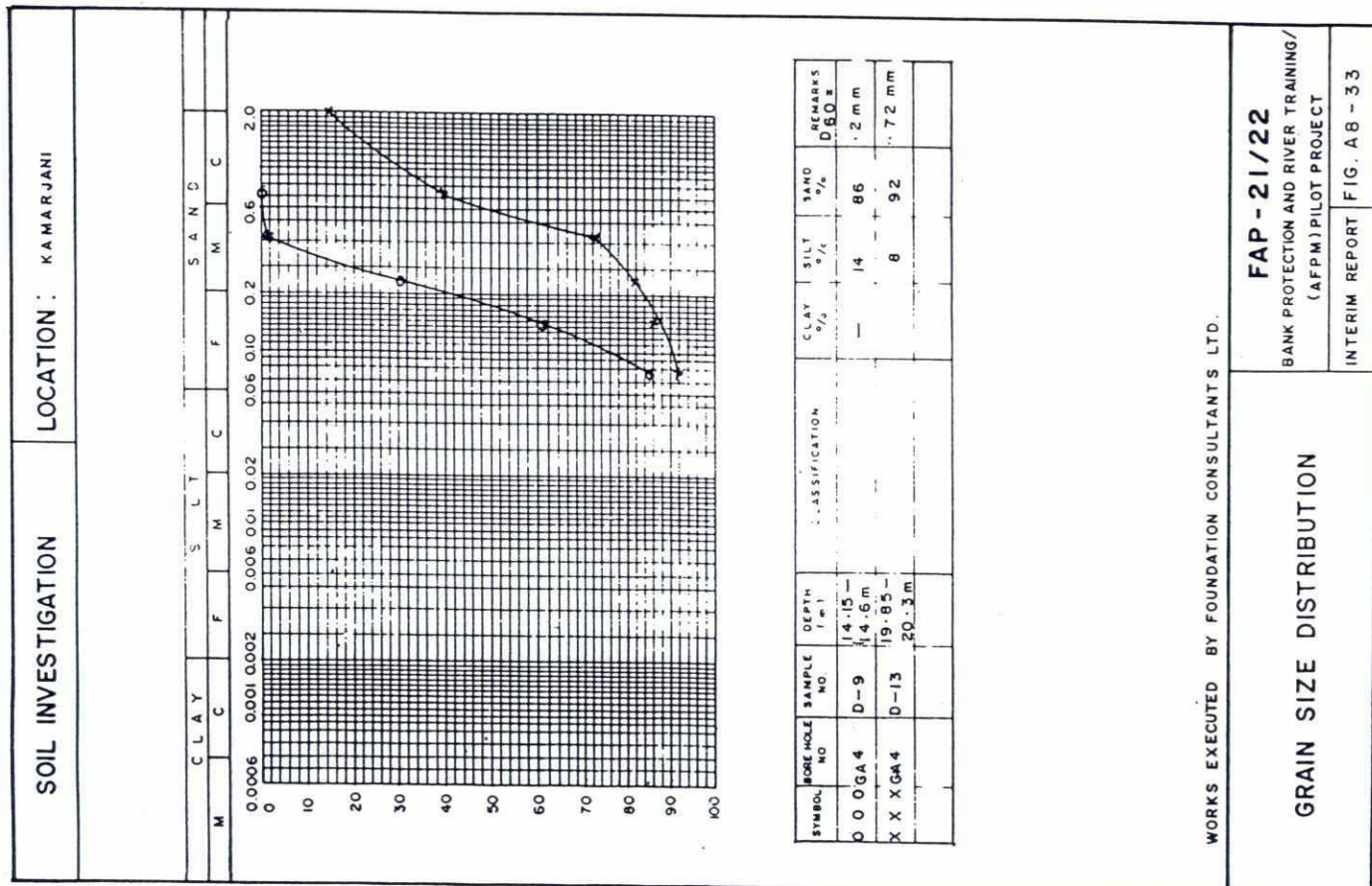
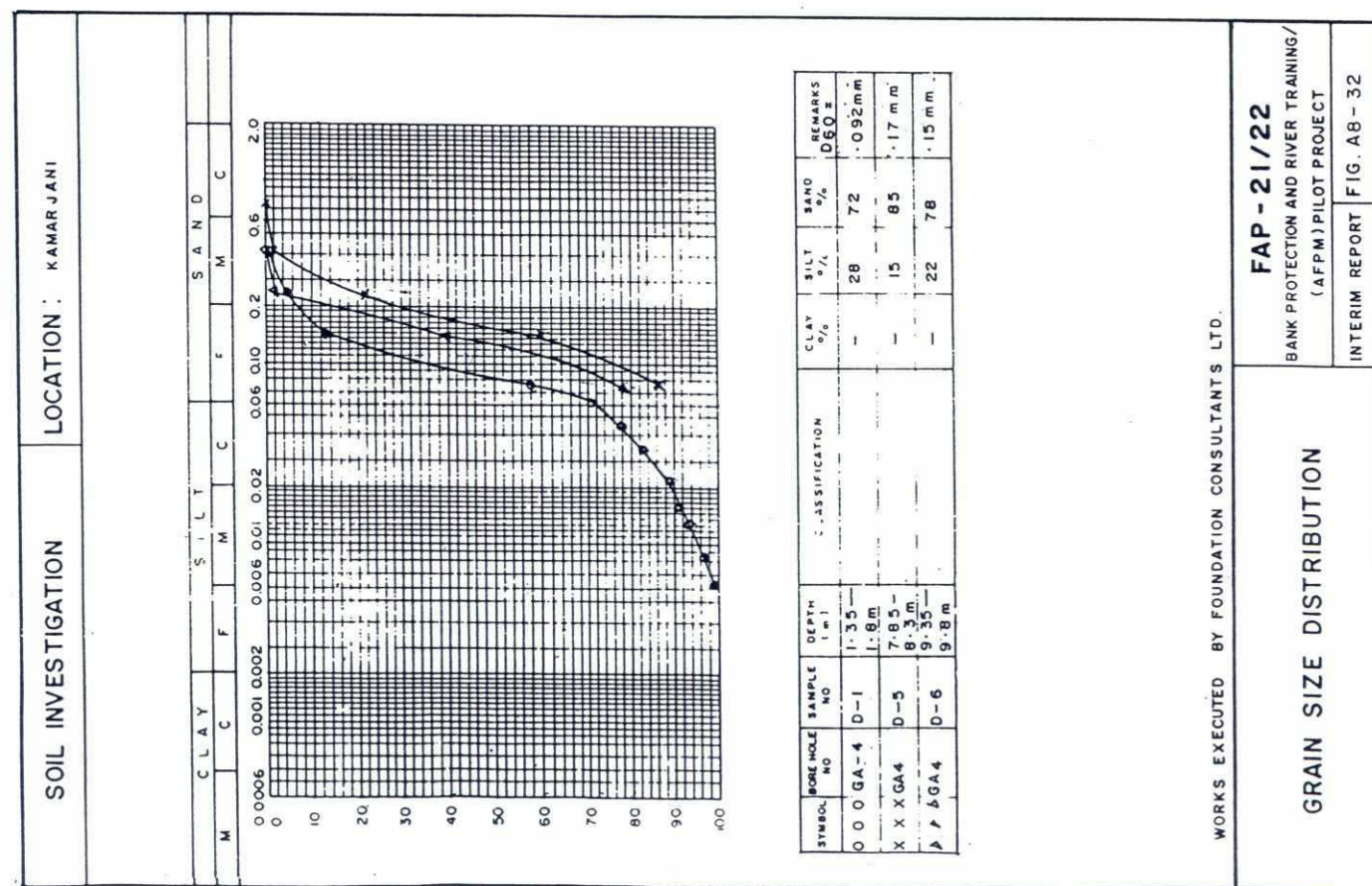
WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

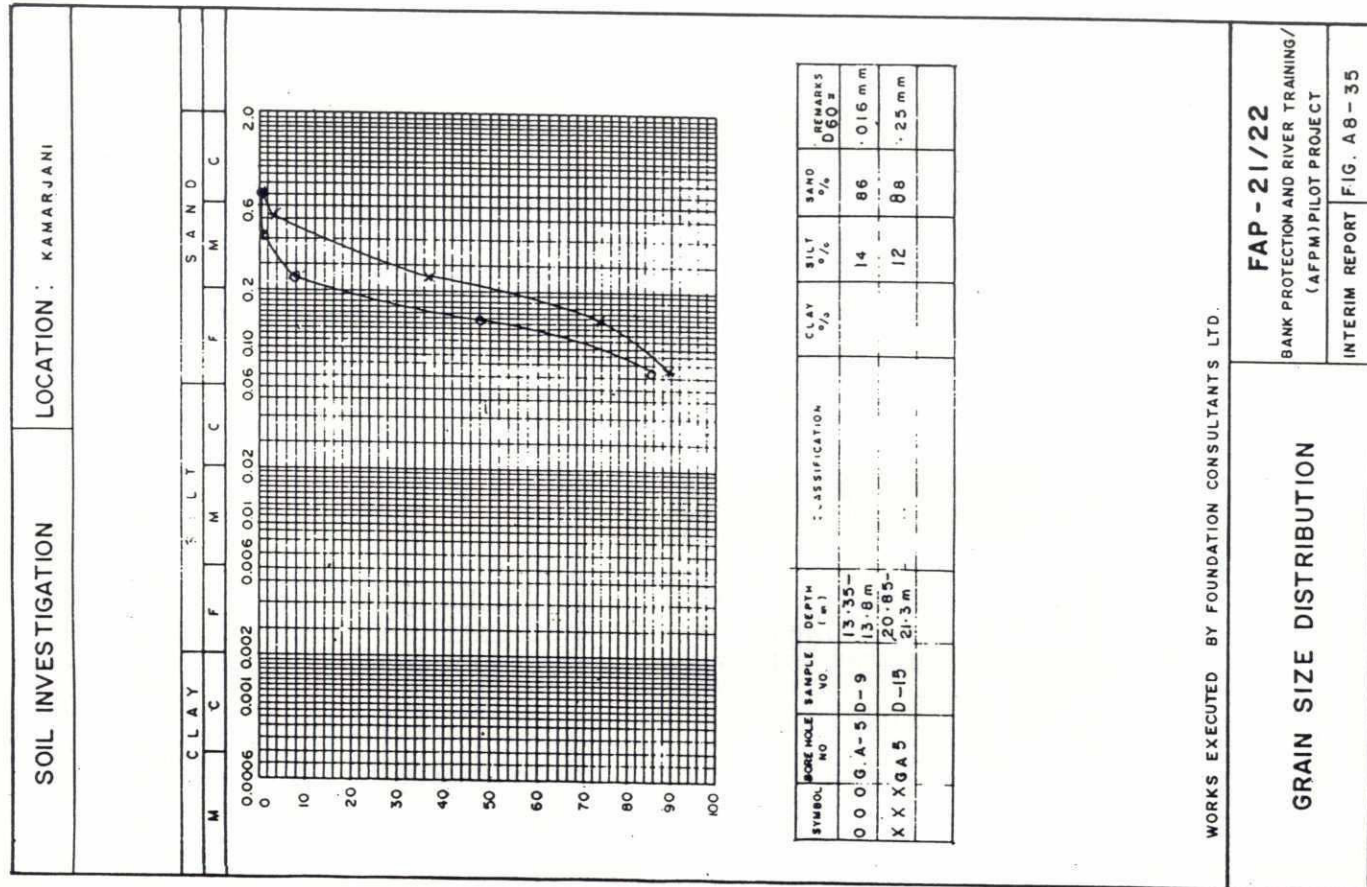
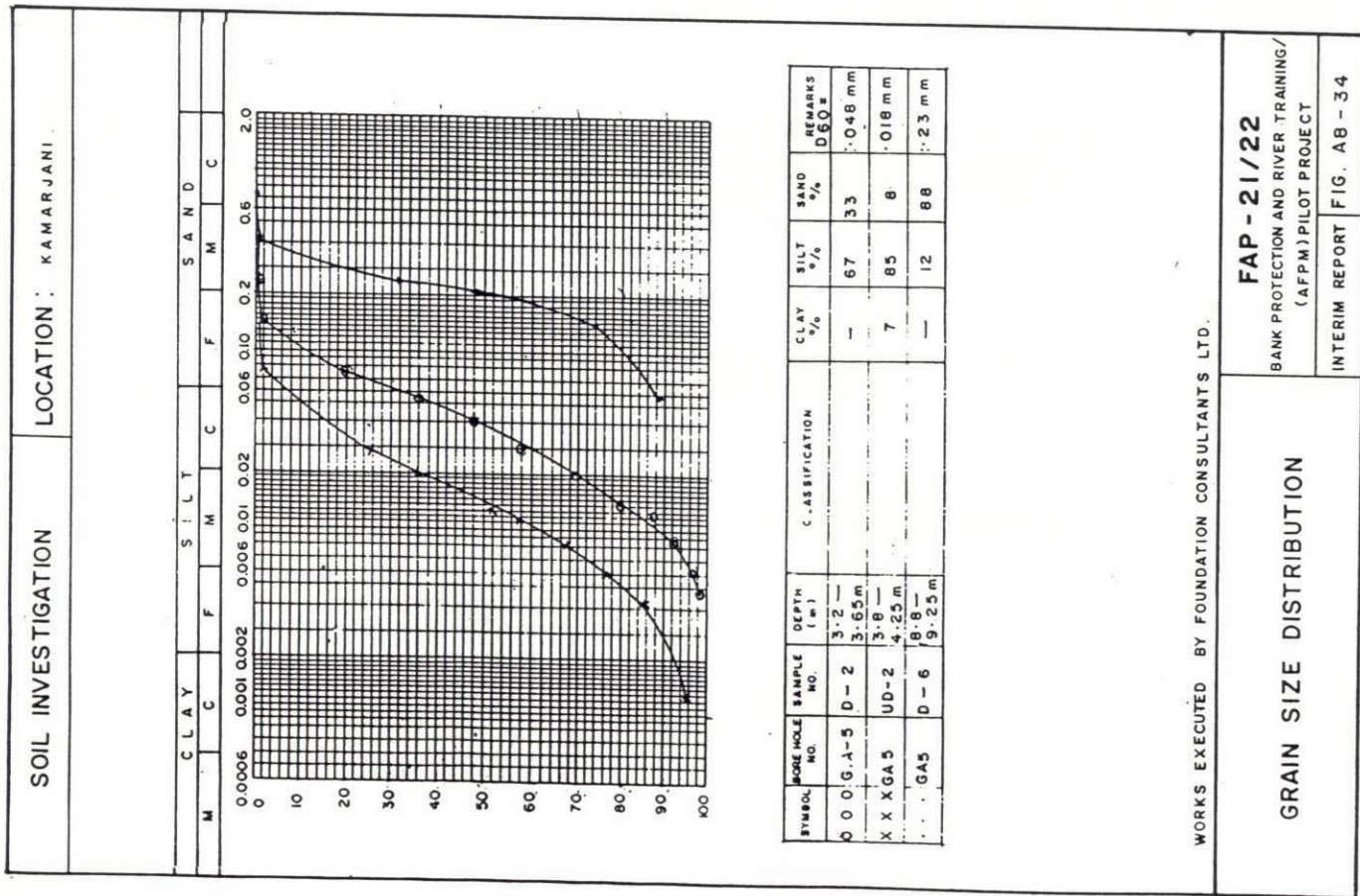
GRAIN SIZE DISTRIBUTION	FAP - 21/22	
	BANK PROTECTION AND RIVER TRAINING/ (AFPM) PILOT PROJECT	
	INTERIM REPORT	FIG. A8 - 30



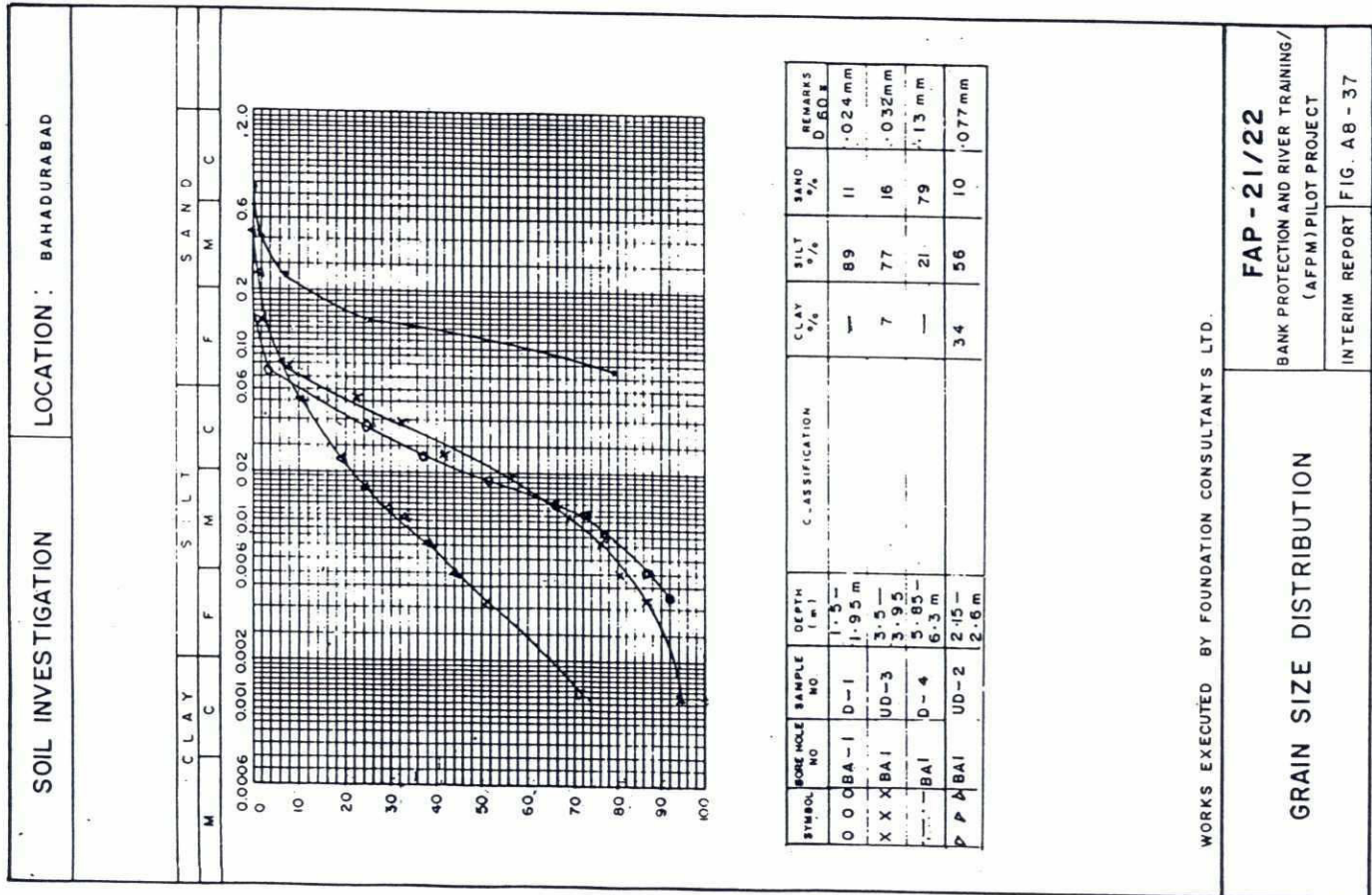
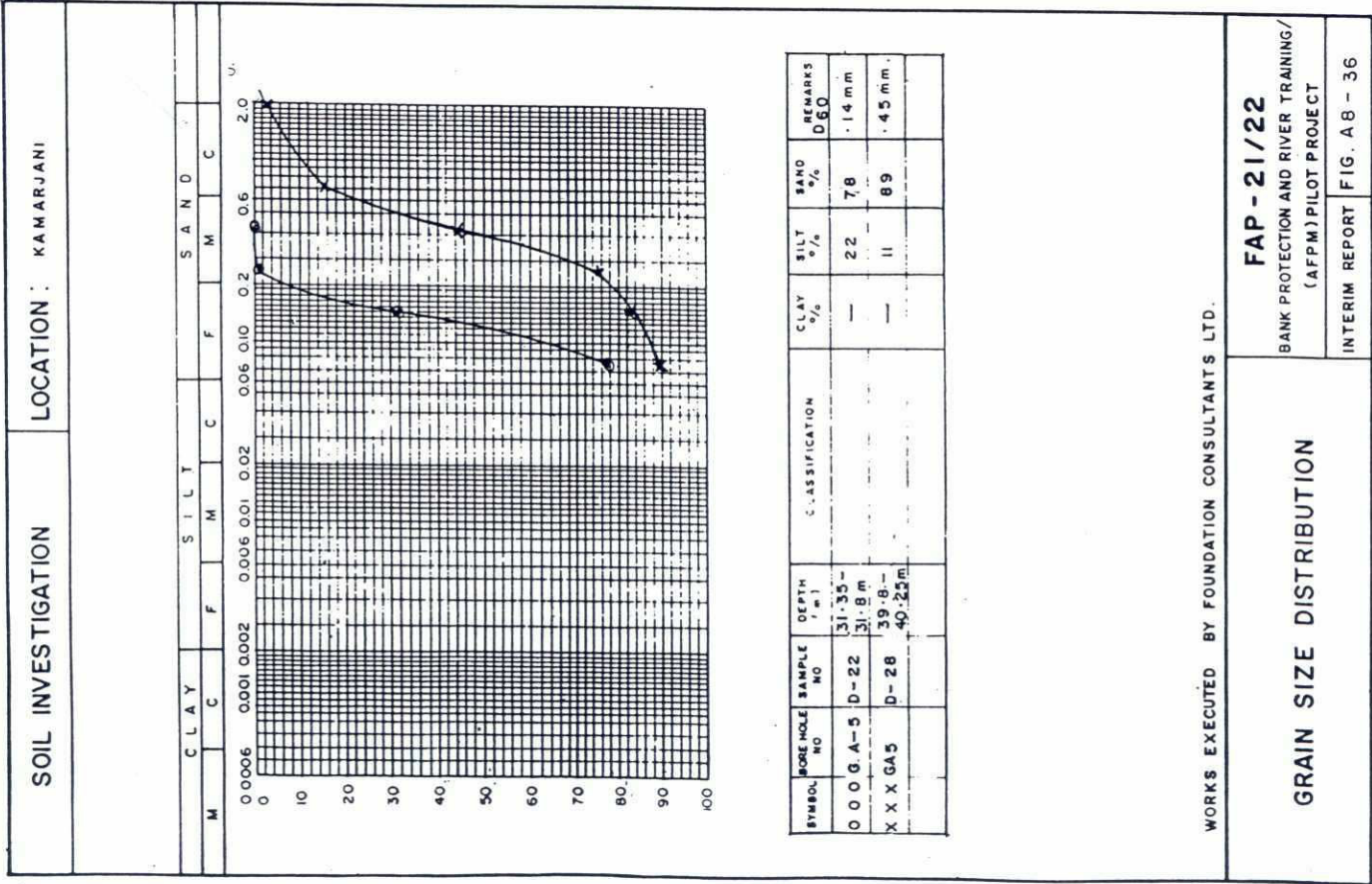
WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

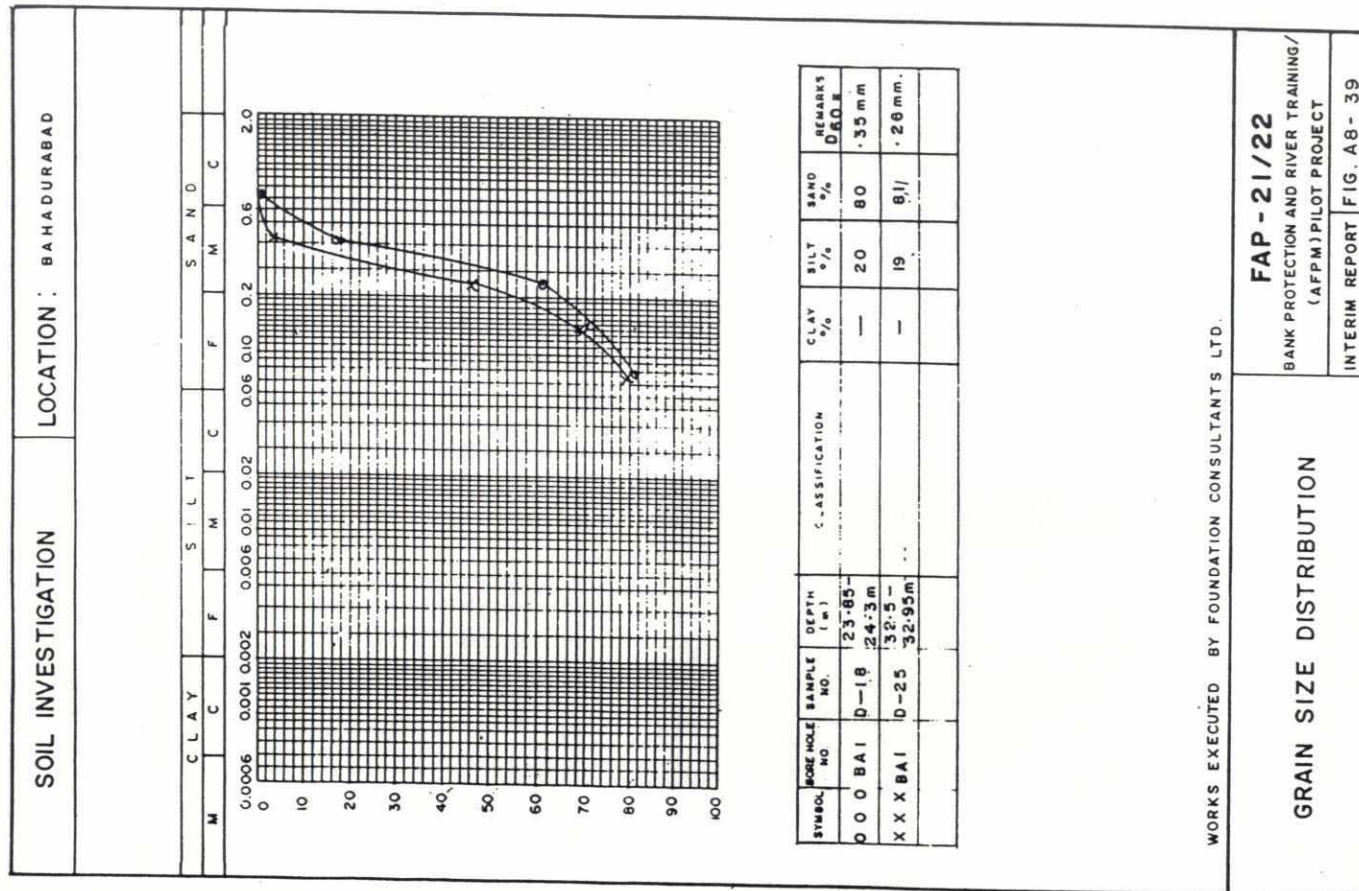
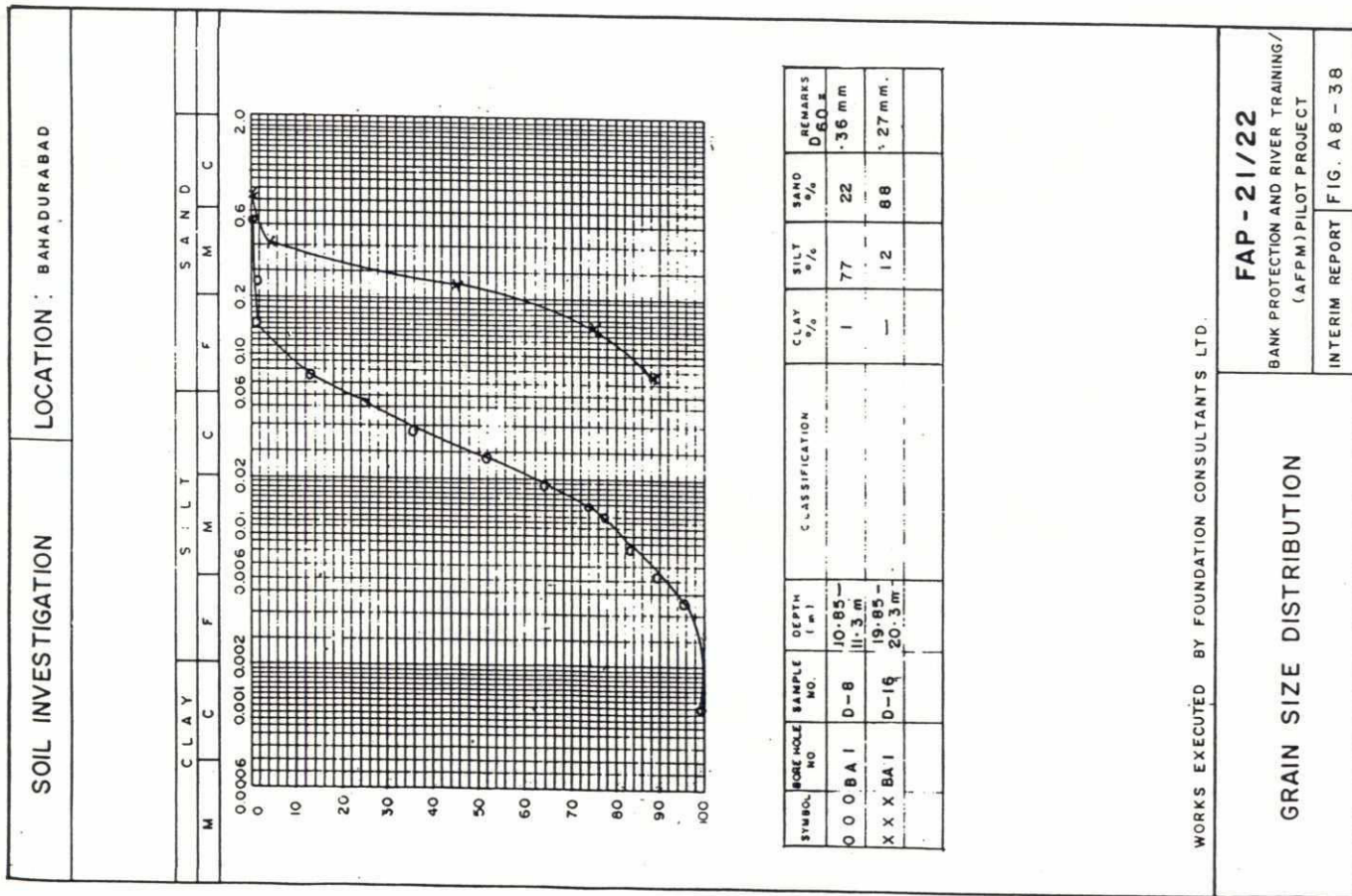
GRAIN SIZE DISTRIBUTION	FAP - 21/22	
	BANK PROTECTION AND RIVER TRAINING/ (AFPM) PILOT PROJECT	
	INTERIM REPORT	FIG. A8 - 31

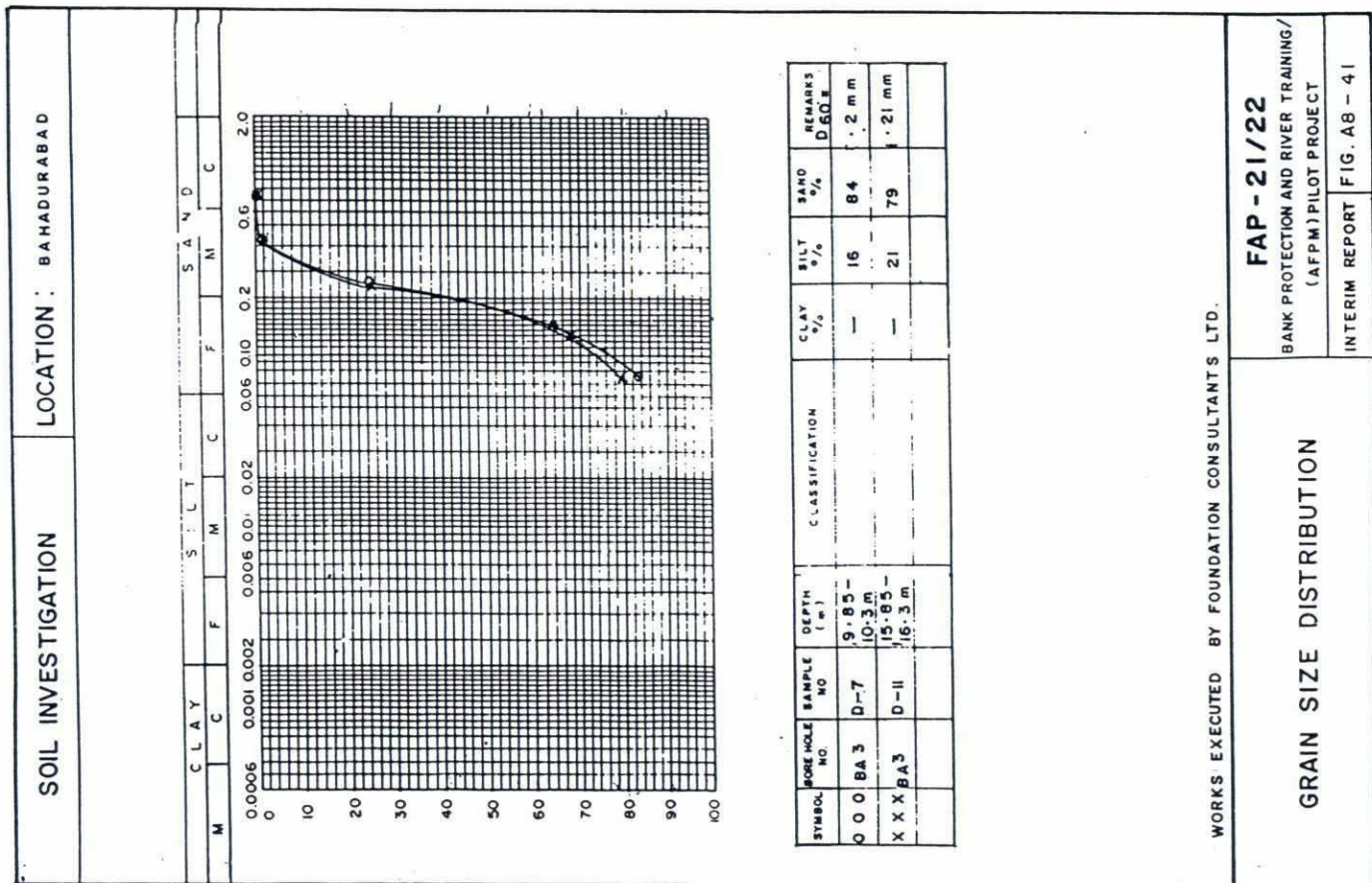
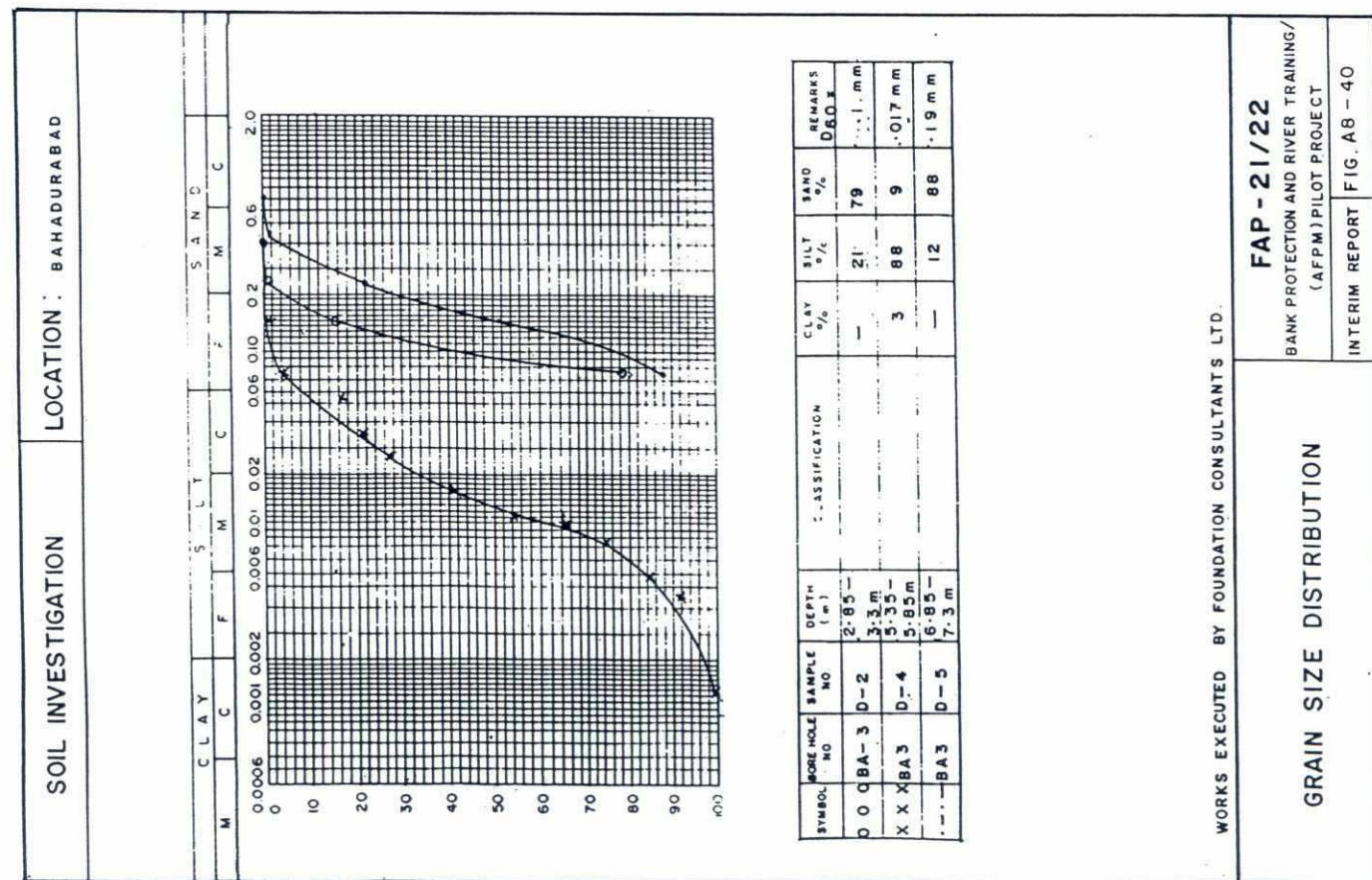


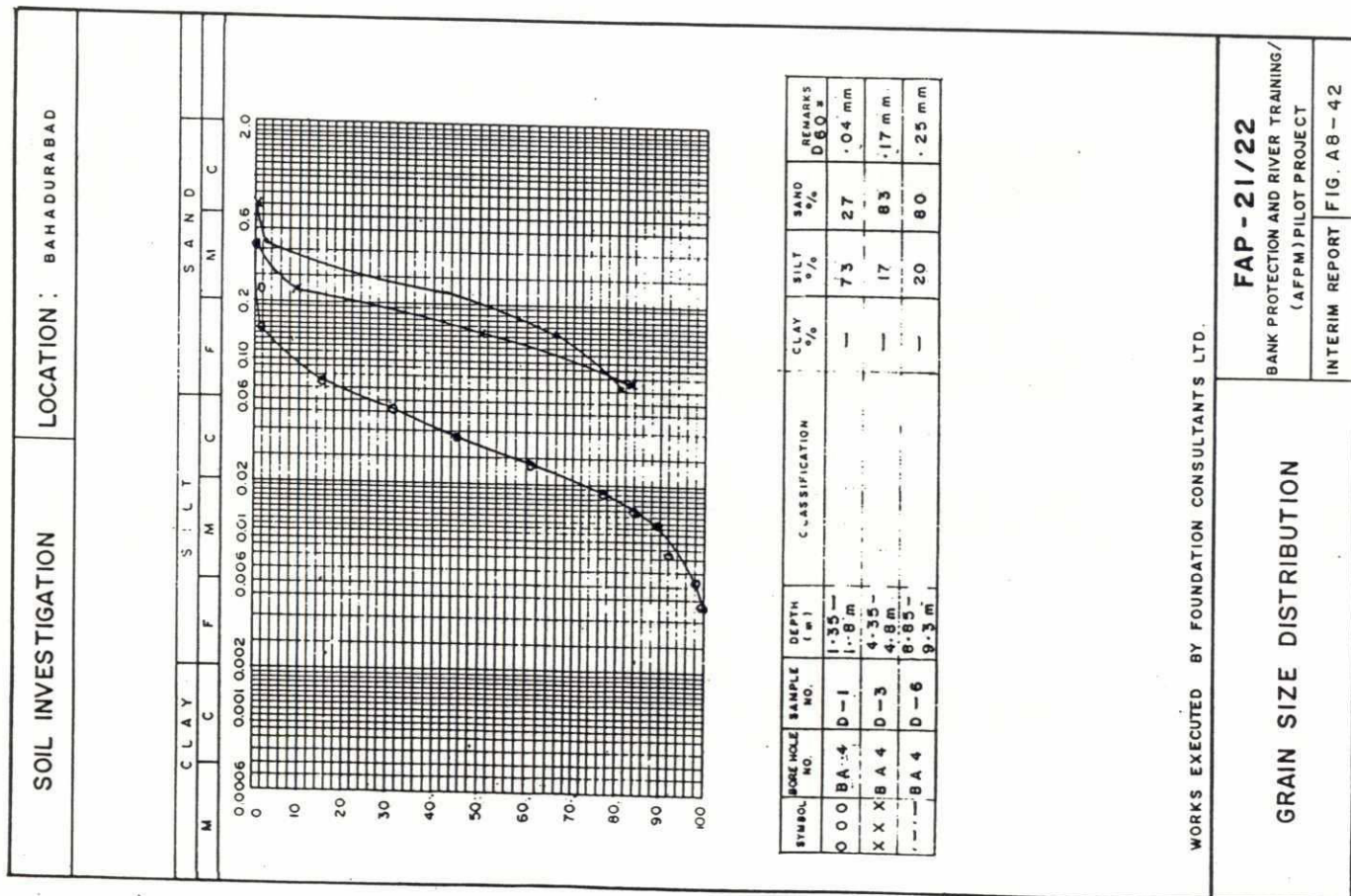


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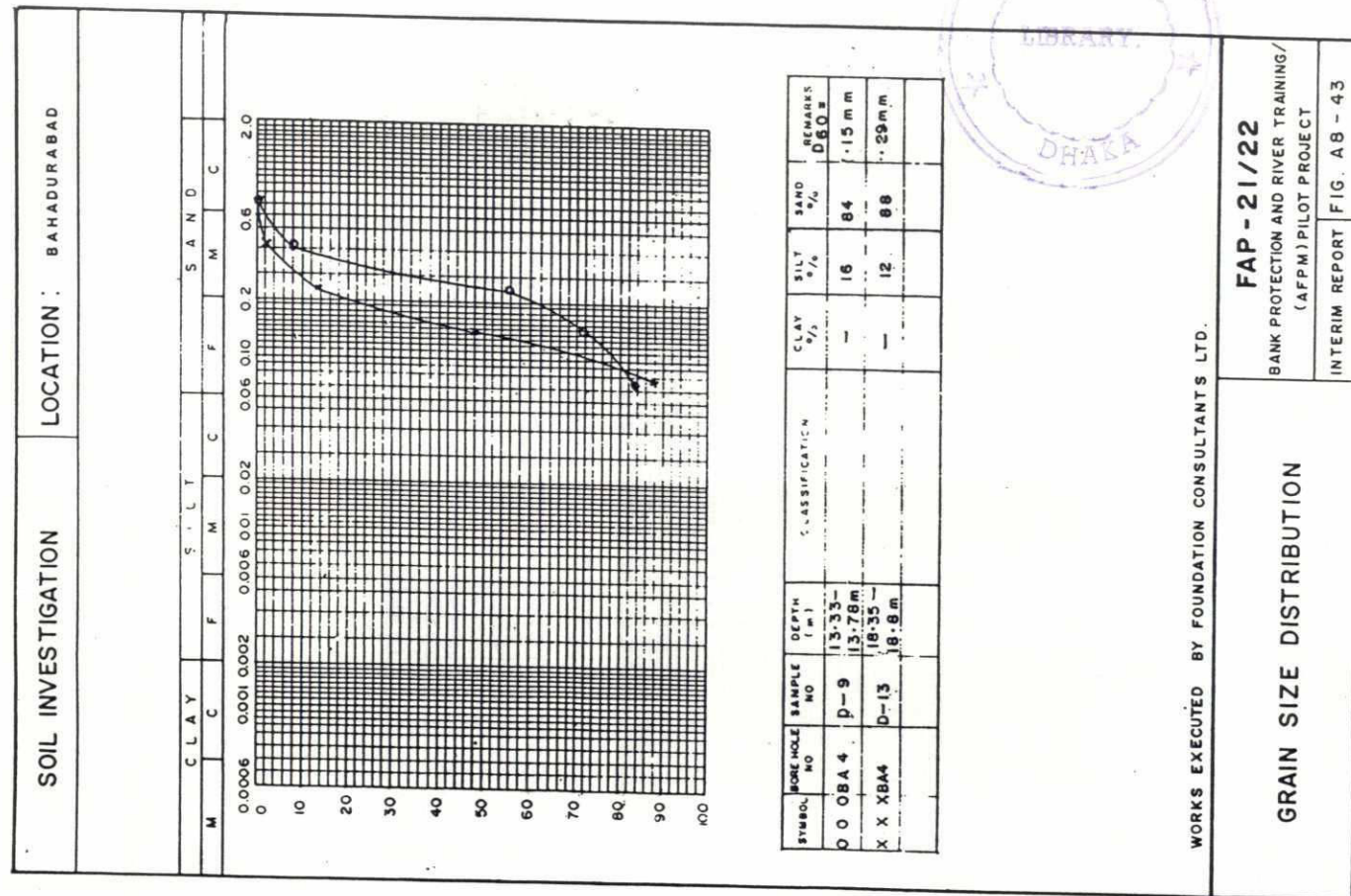






WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

GRAIN SIZE DISTRIBUTION	FAP - 21/22	
	BANK PROTECTION AND RIVER TRAINING/ (AFPM) PILOT PROJECT	
	INTERIM REPORT	FIG. A8 - 42

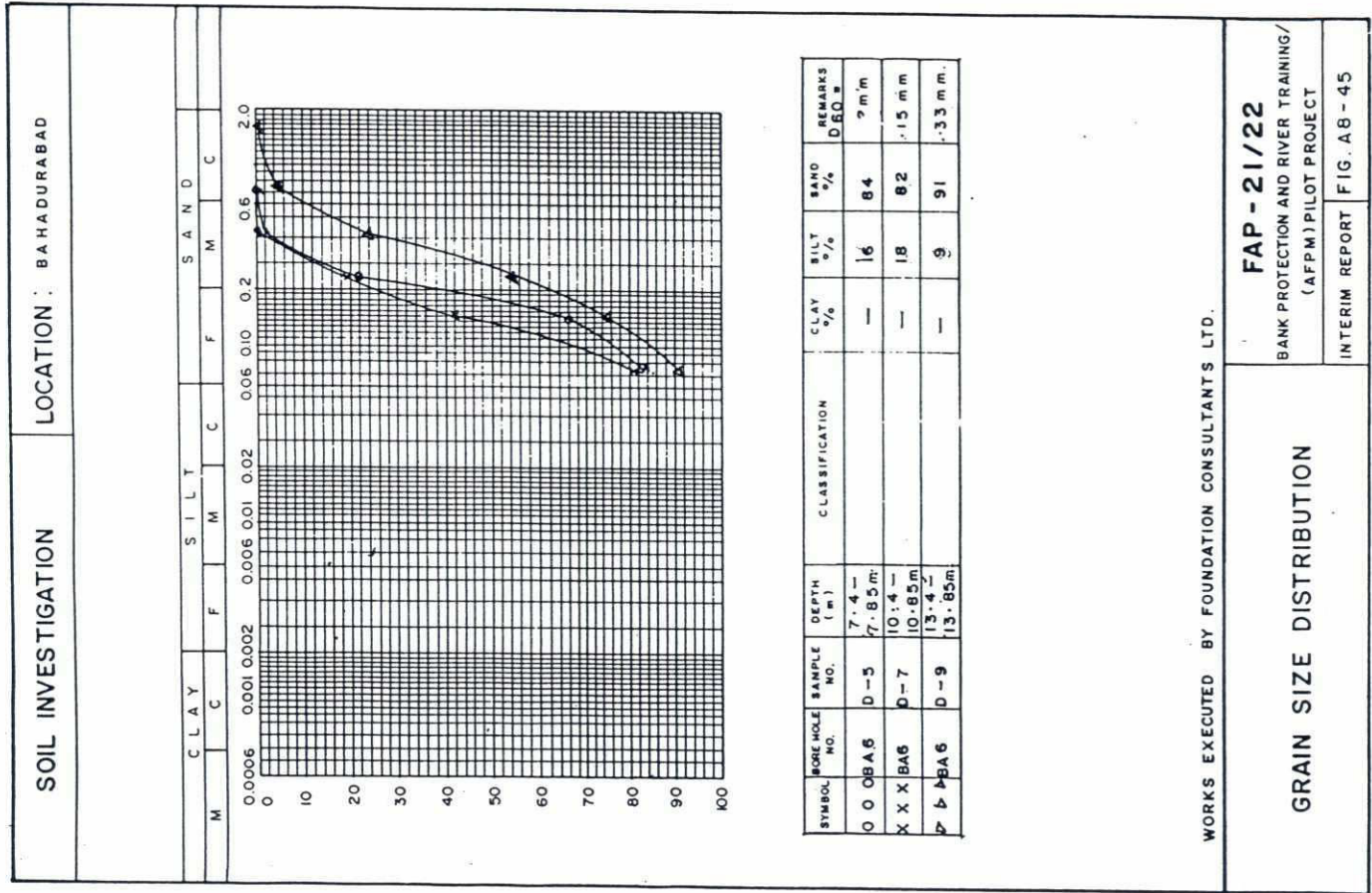
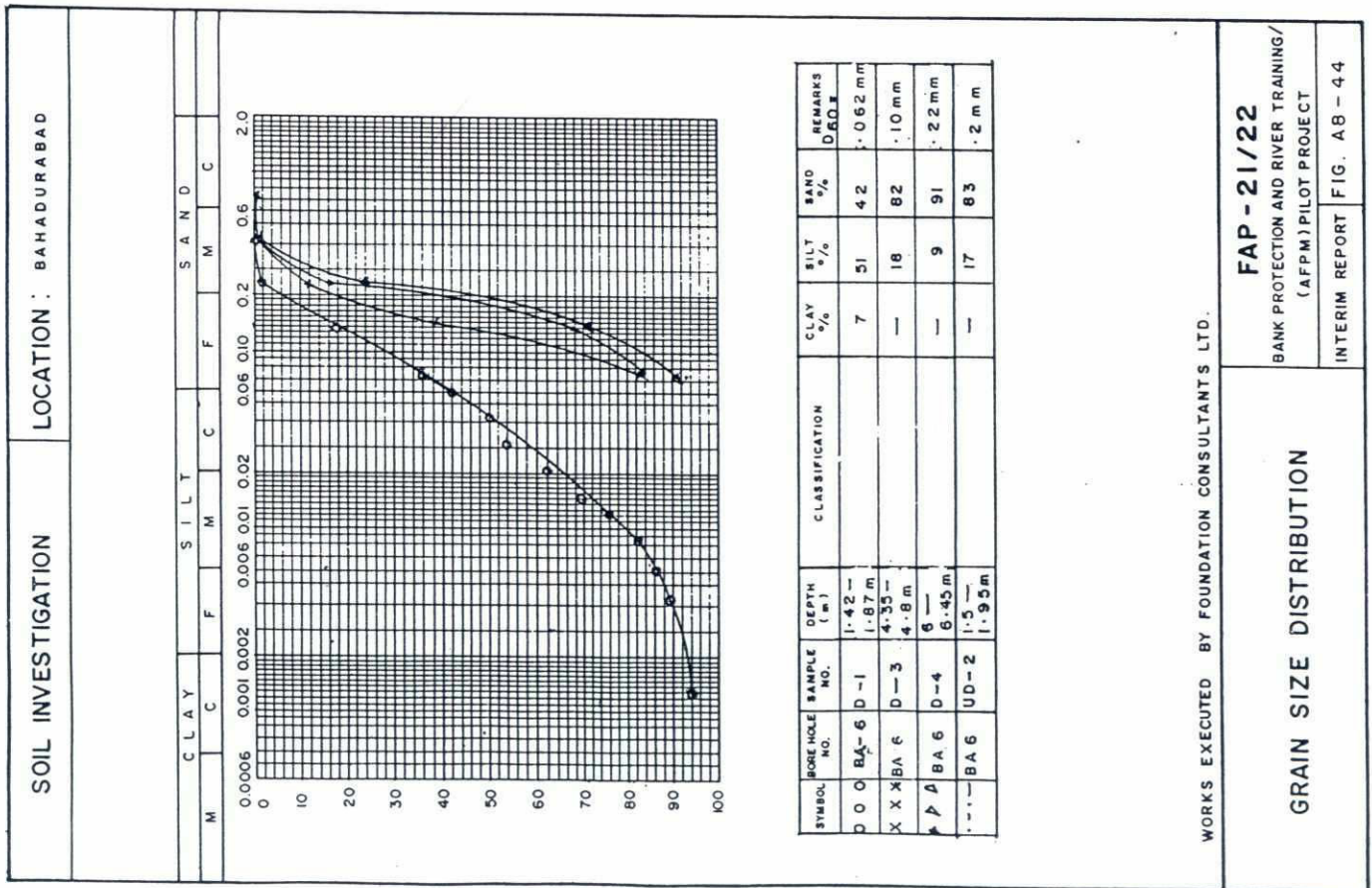


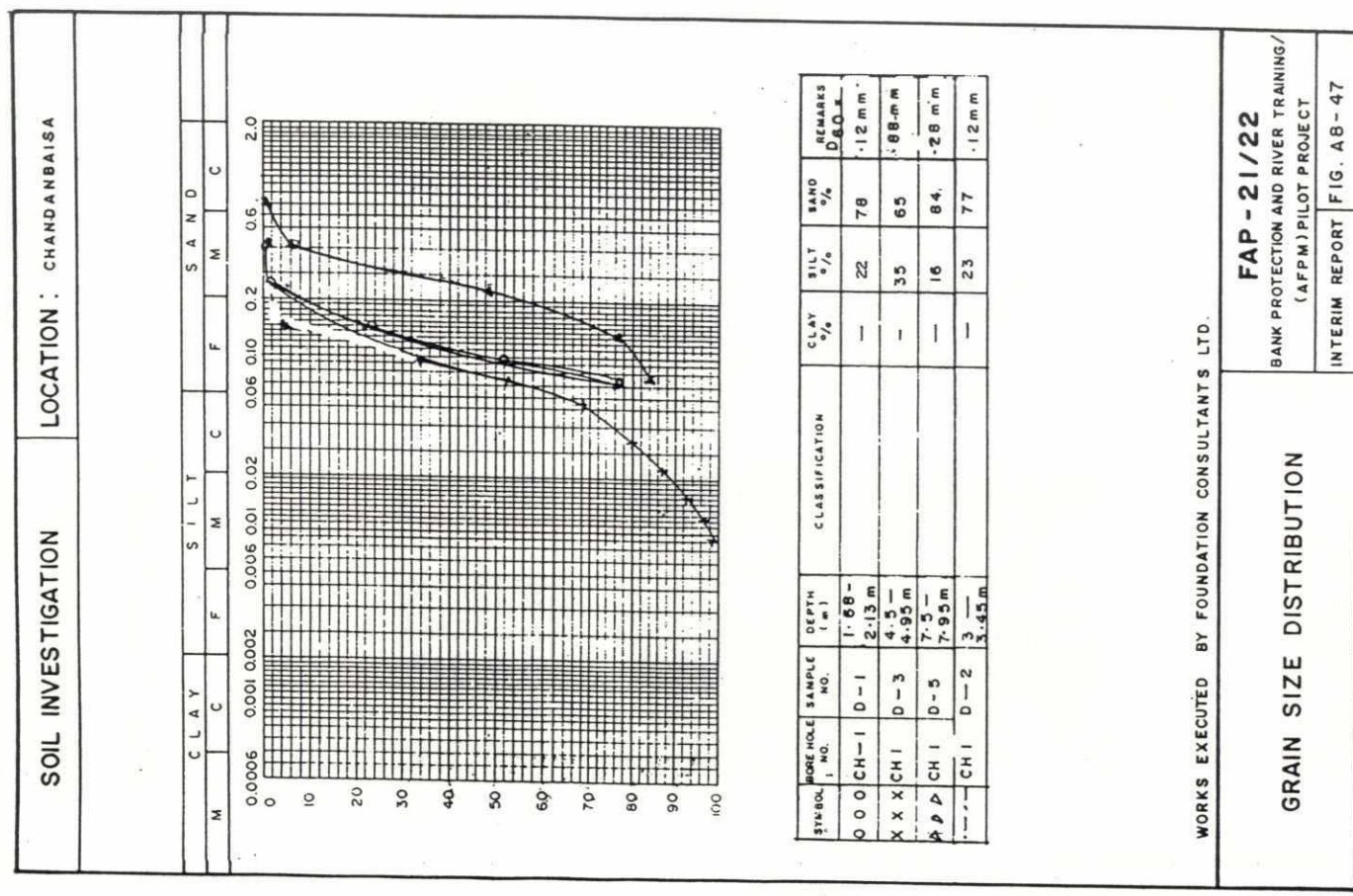
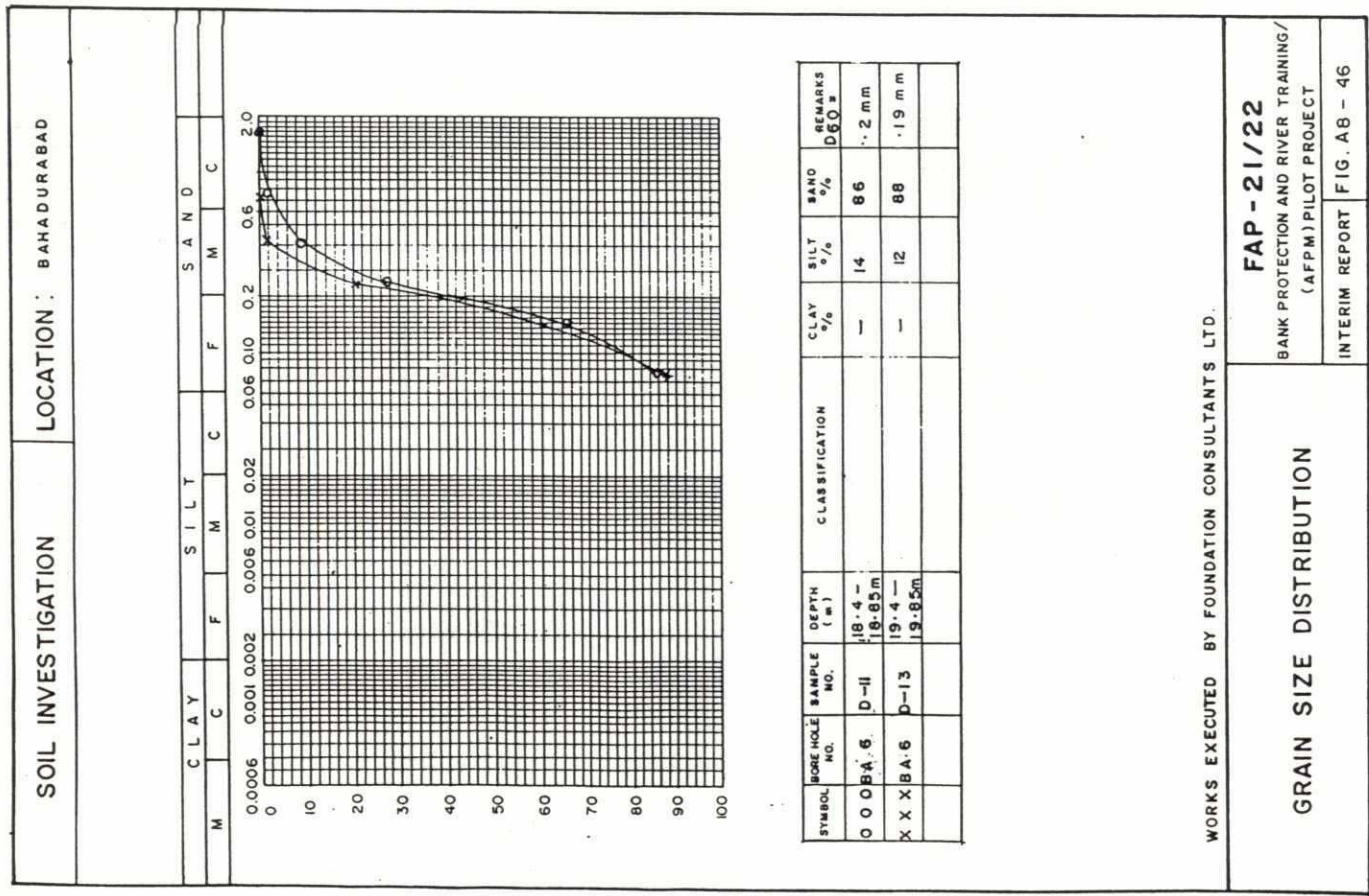
WORKS EXECUTED BY FOUNDATION CONSULTANTS LTD.

GRAIN SIZE DISTRIBUTION	FAP - 21/22	
	BANK PROTECTION AND RIVER TRAINING/ (AFPM) PILOT PROJECT	
	INTERIM REPORT	FIG. A8 - 43

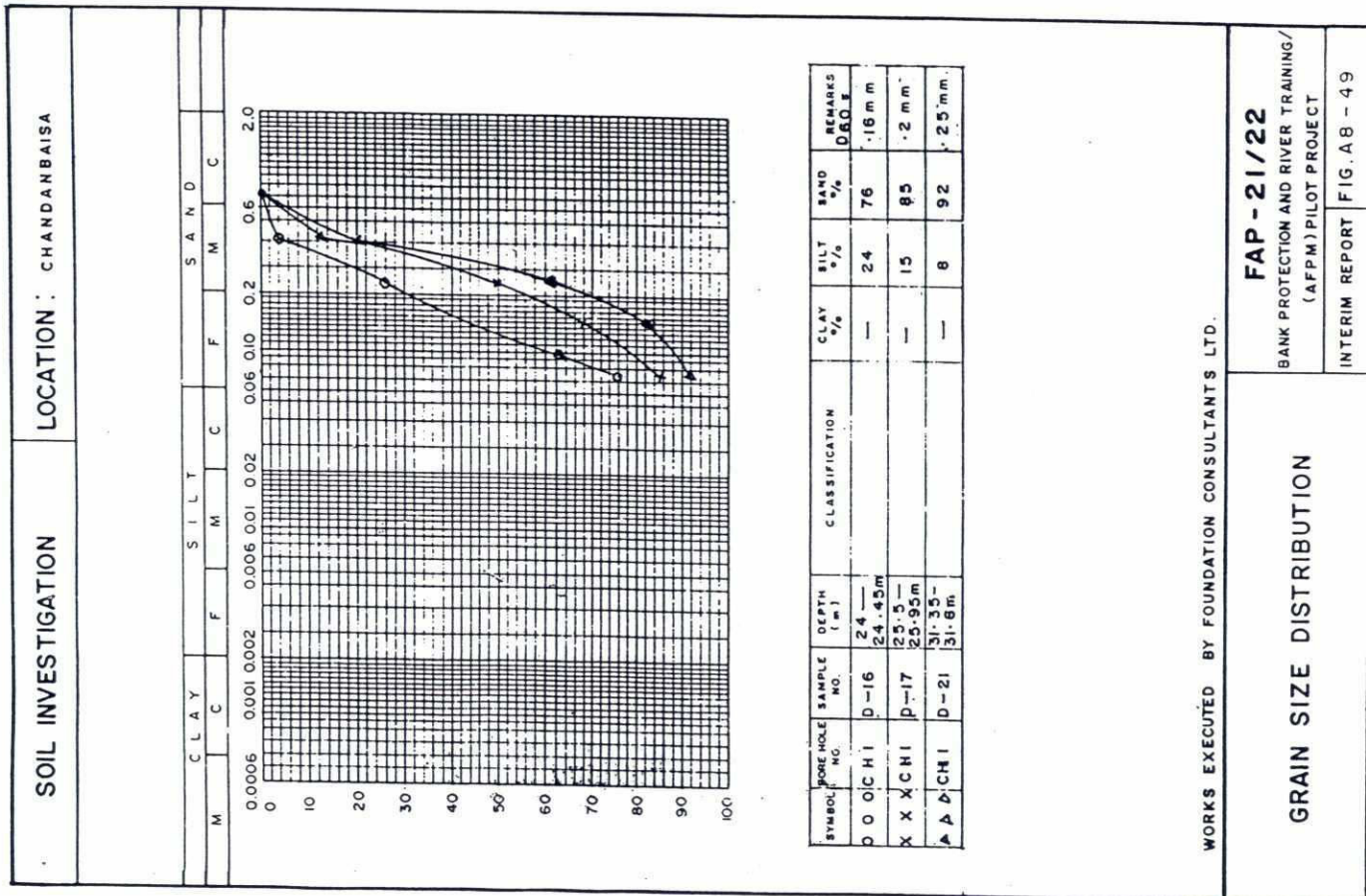
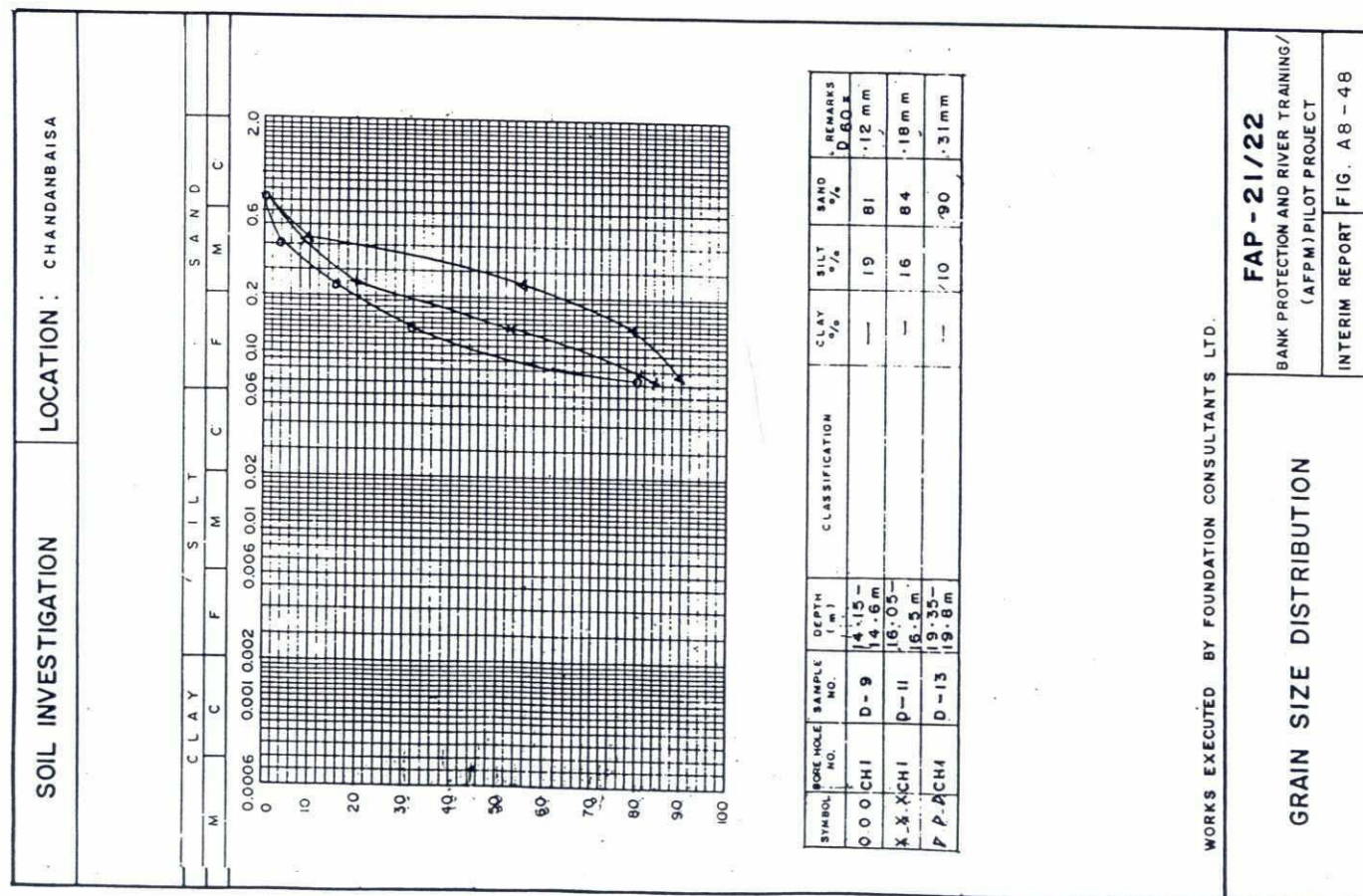


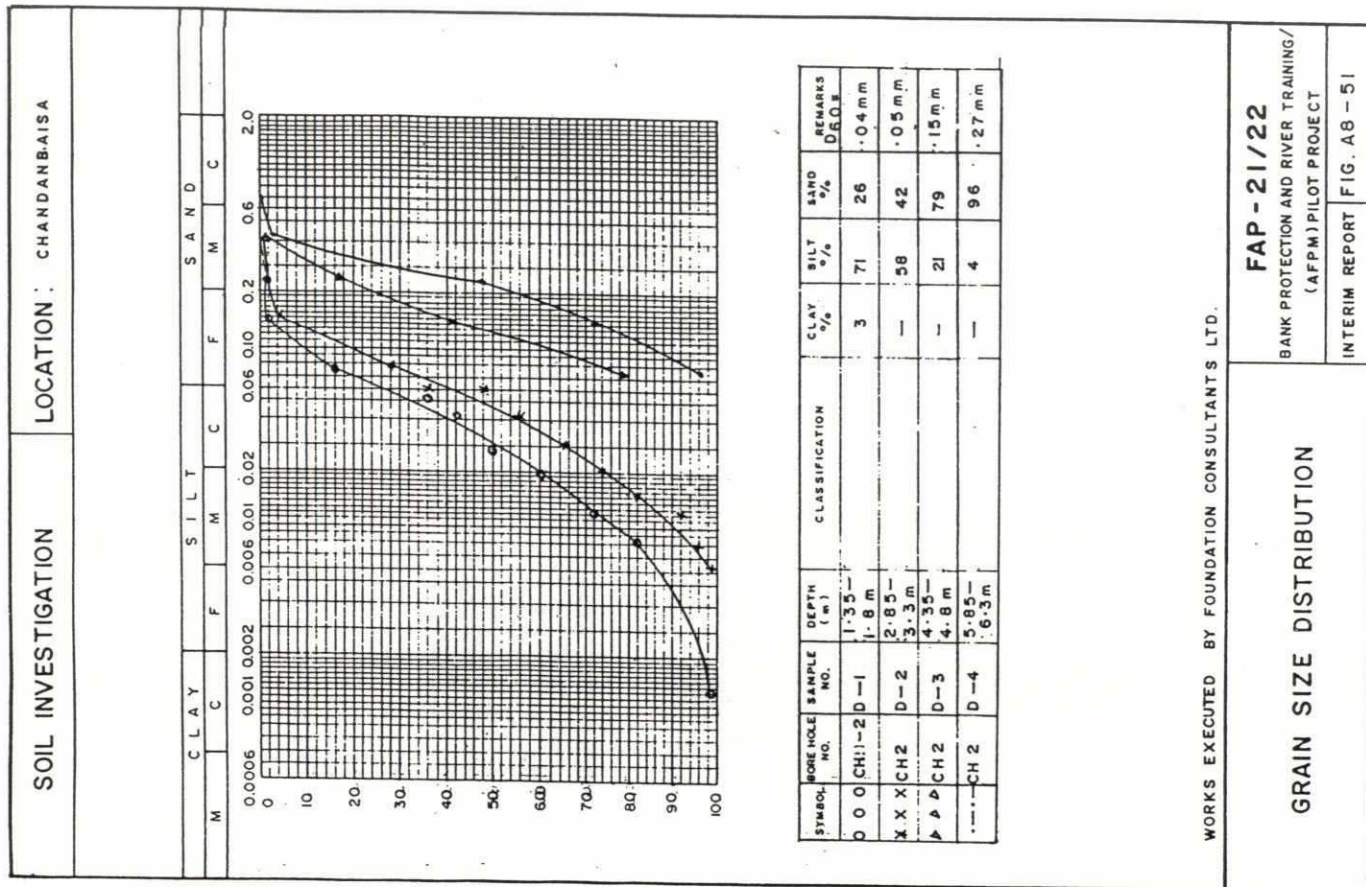
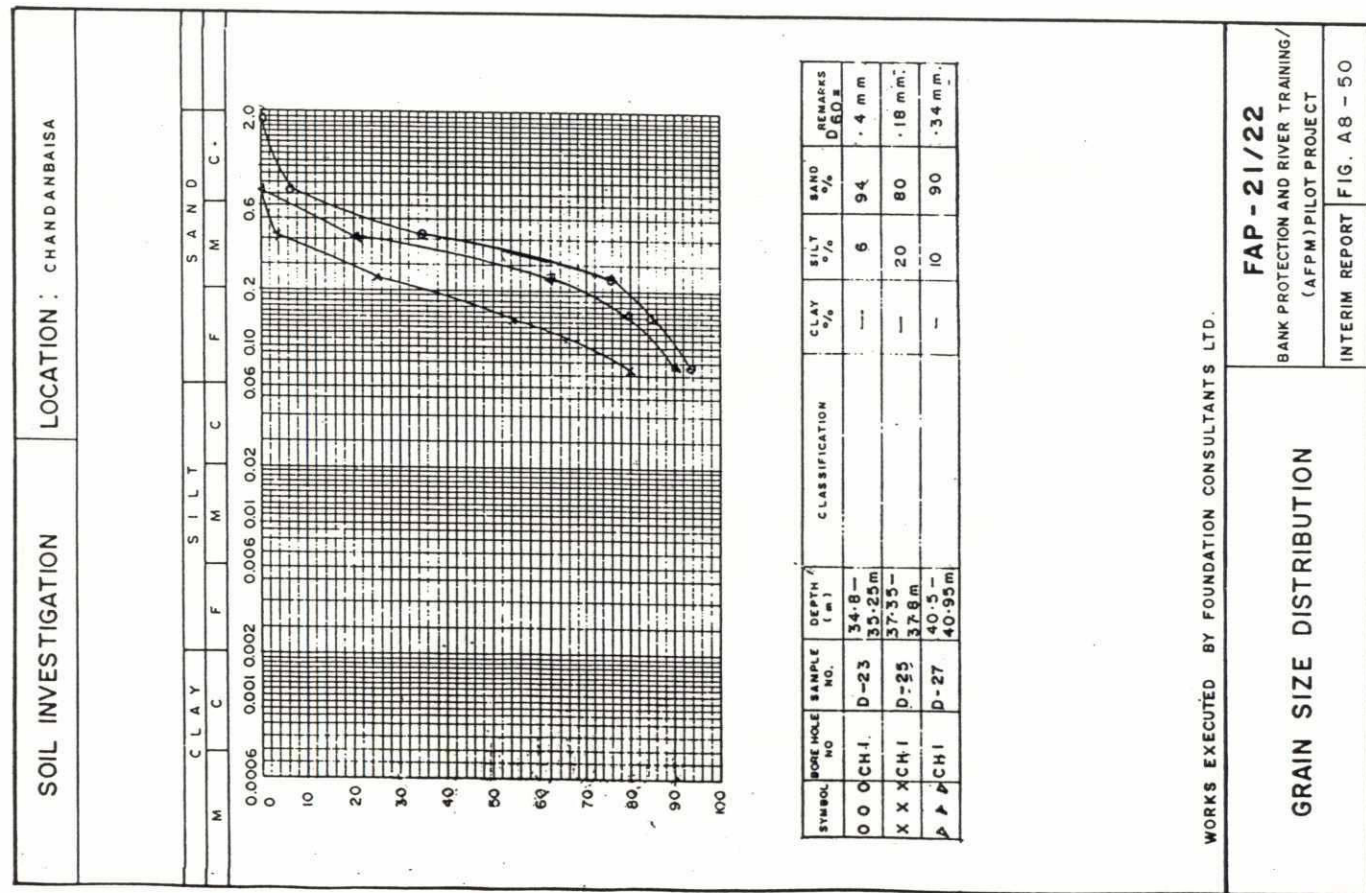
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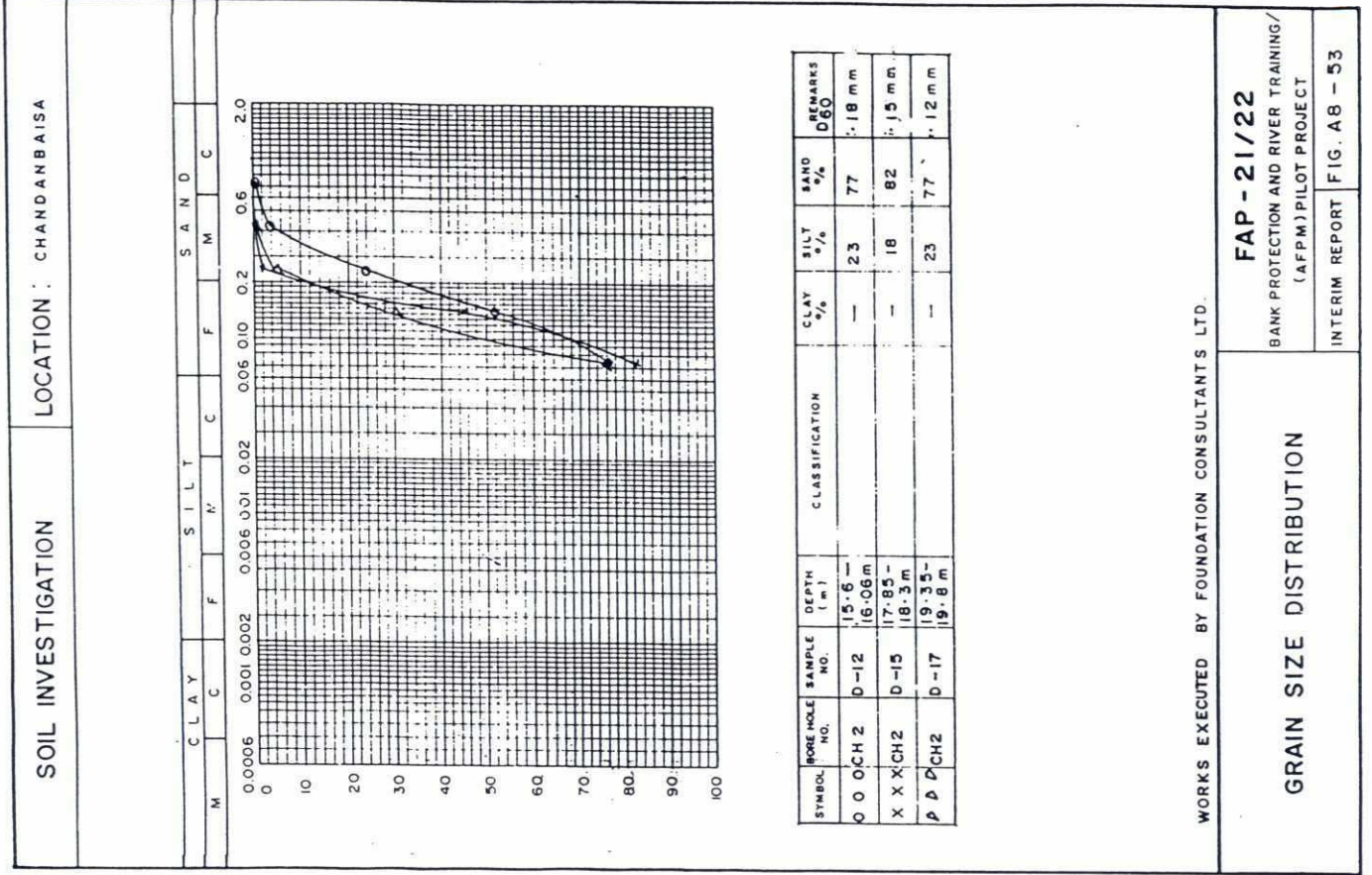
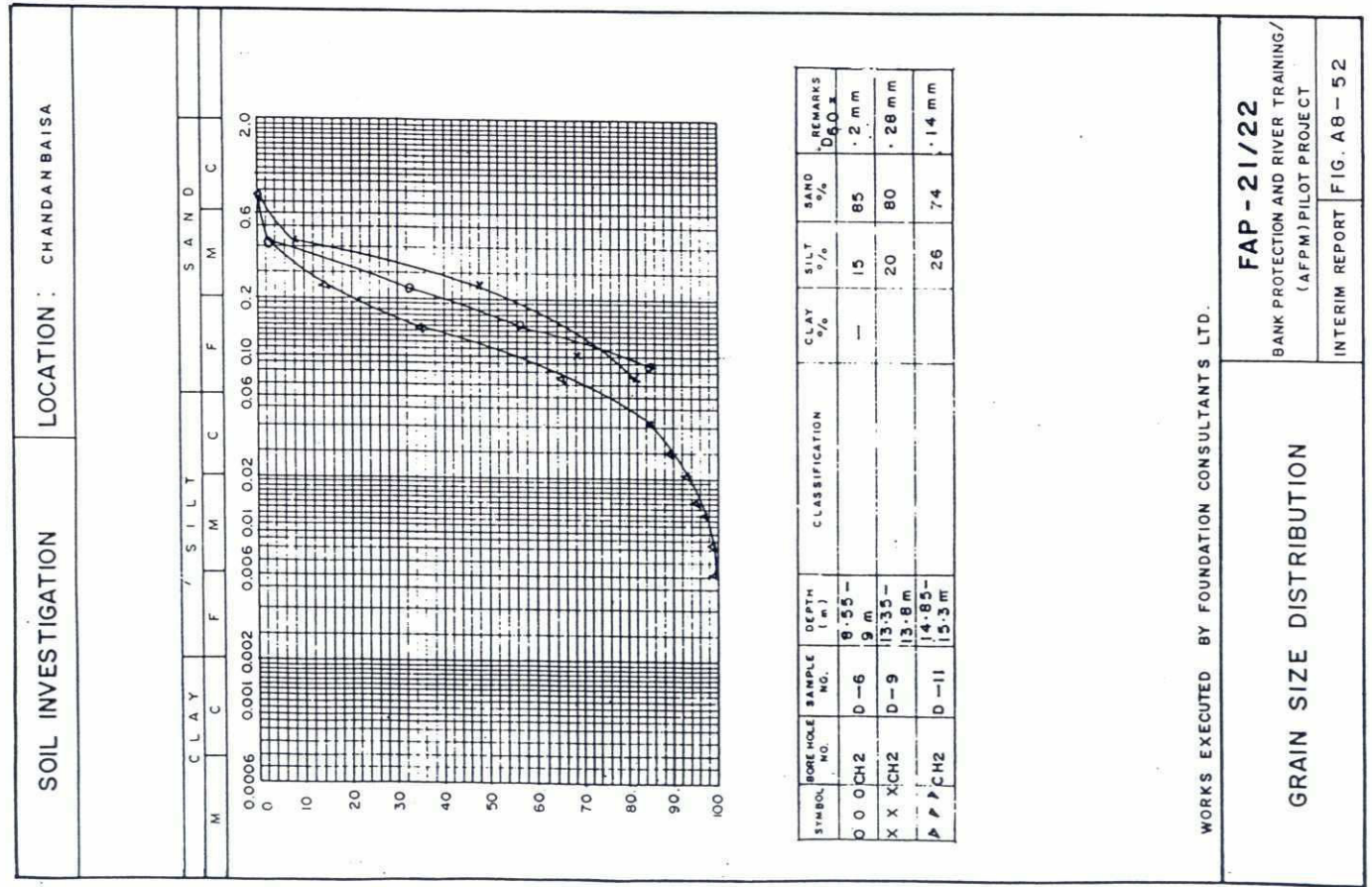


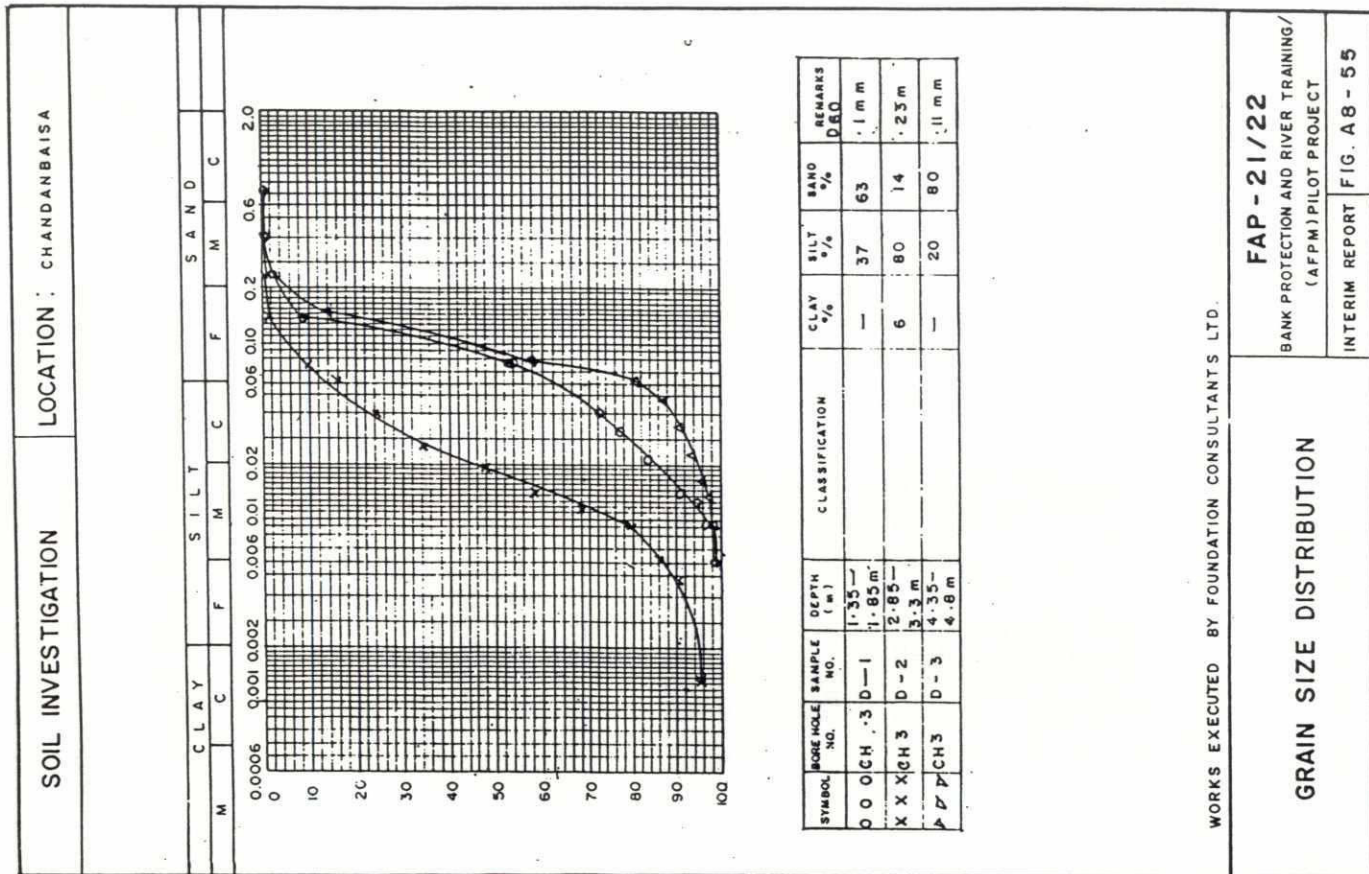


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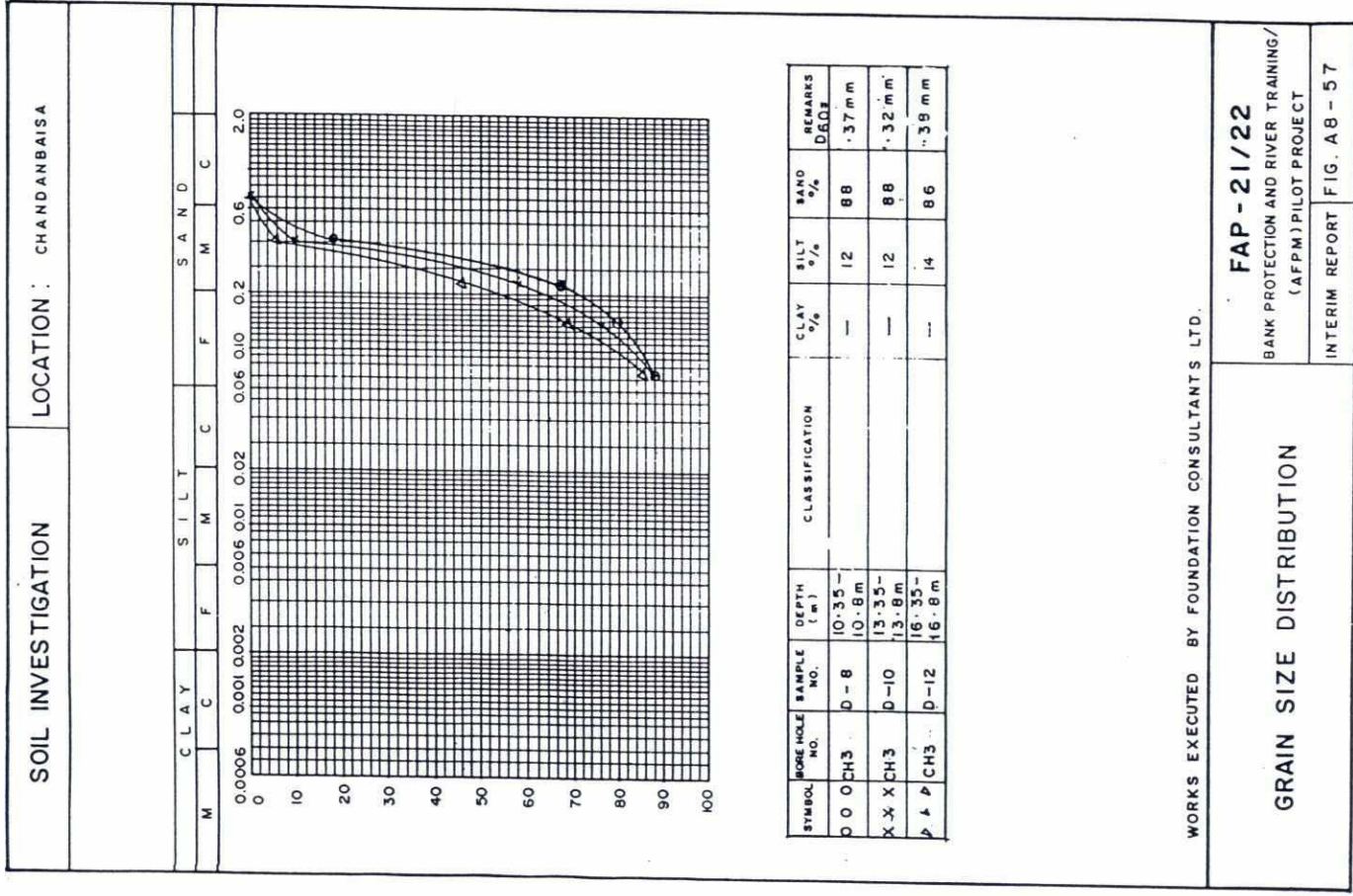
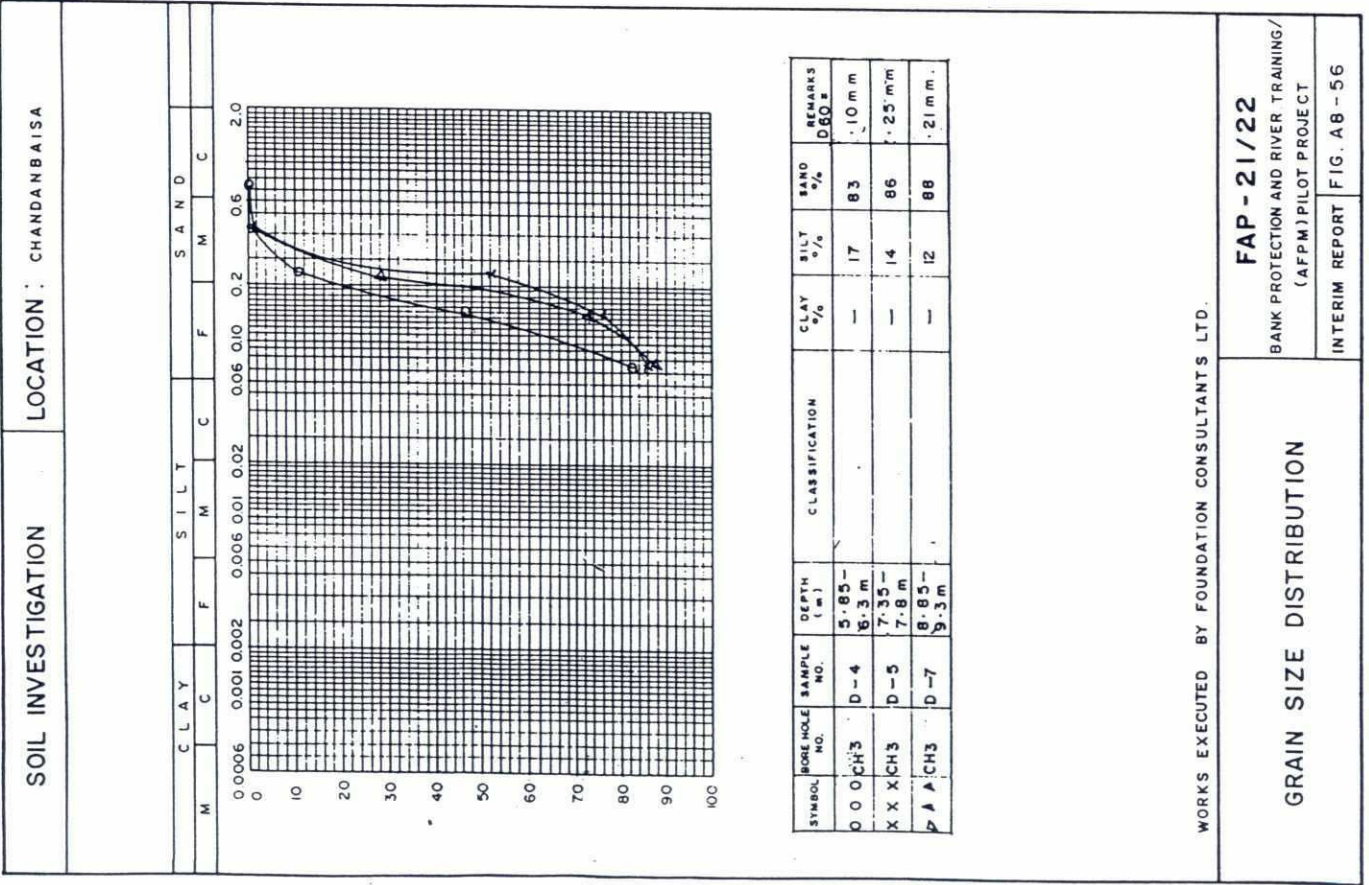
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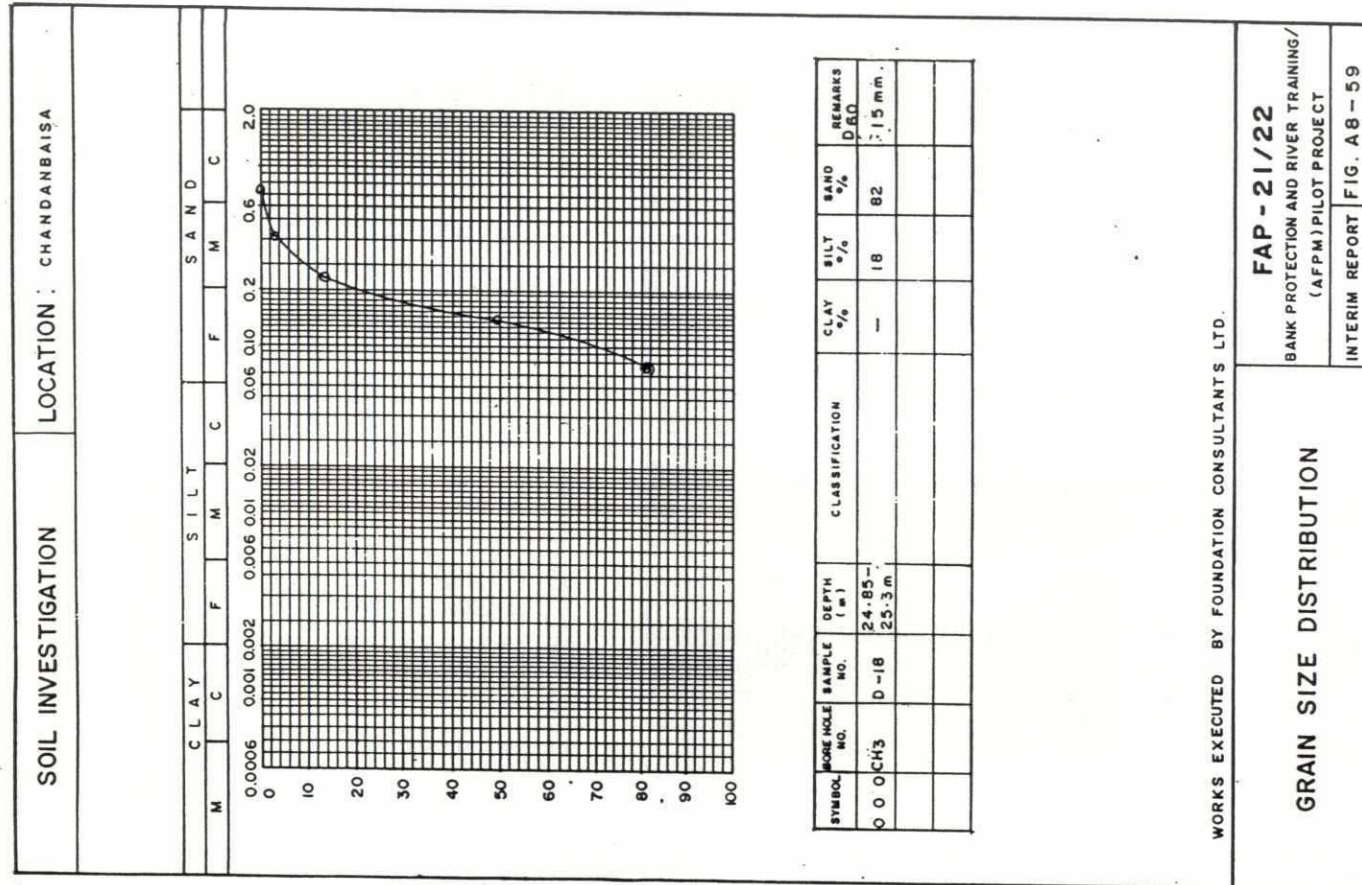
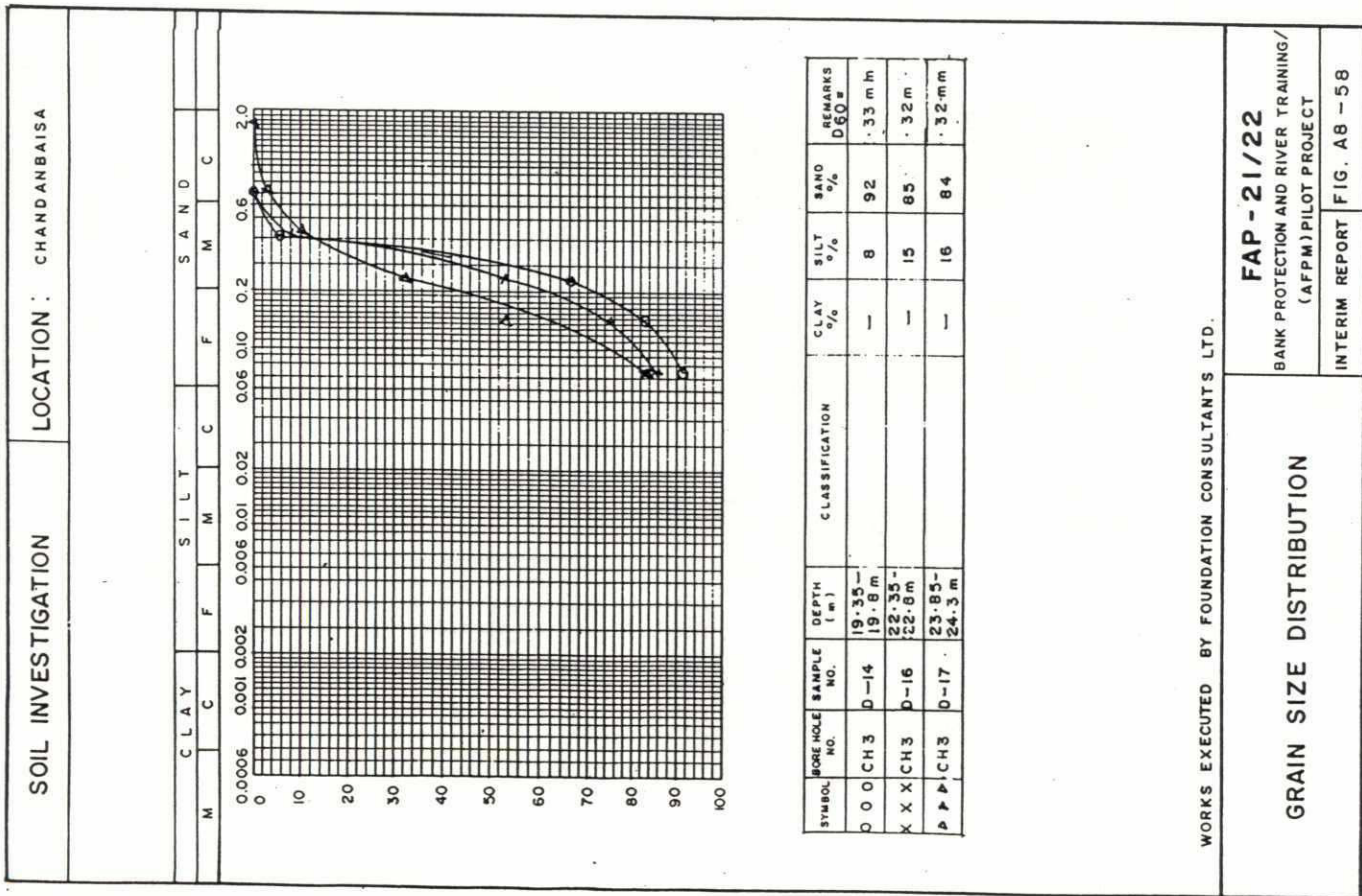
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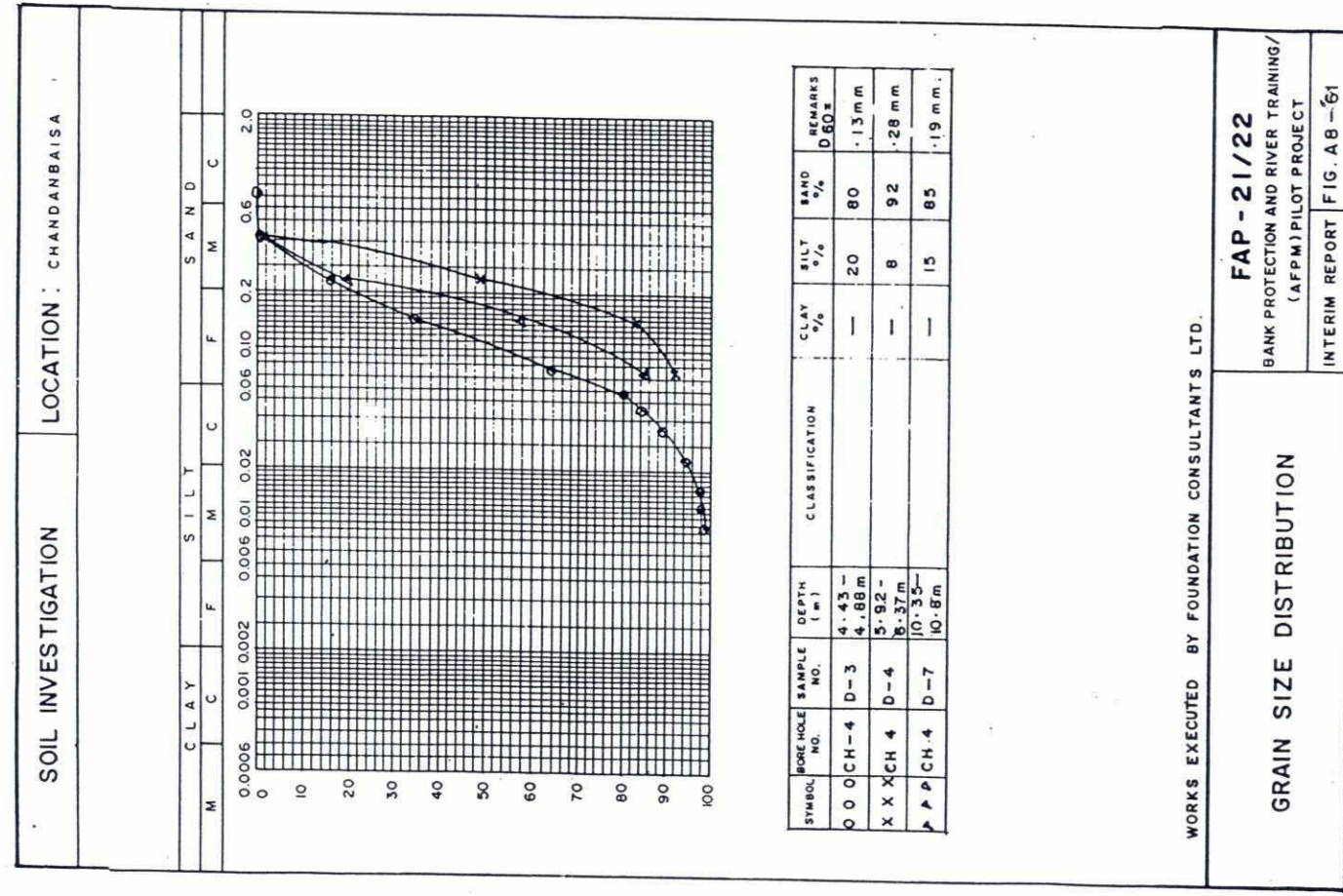
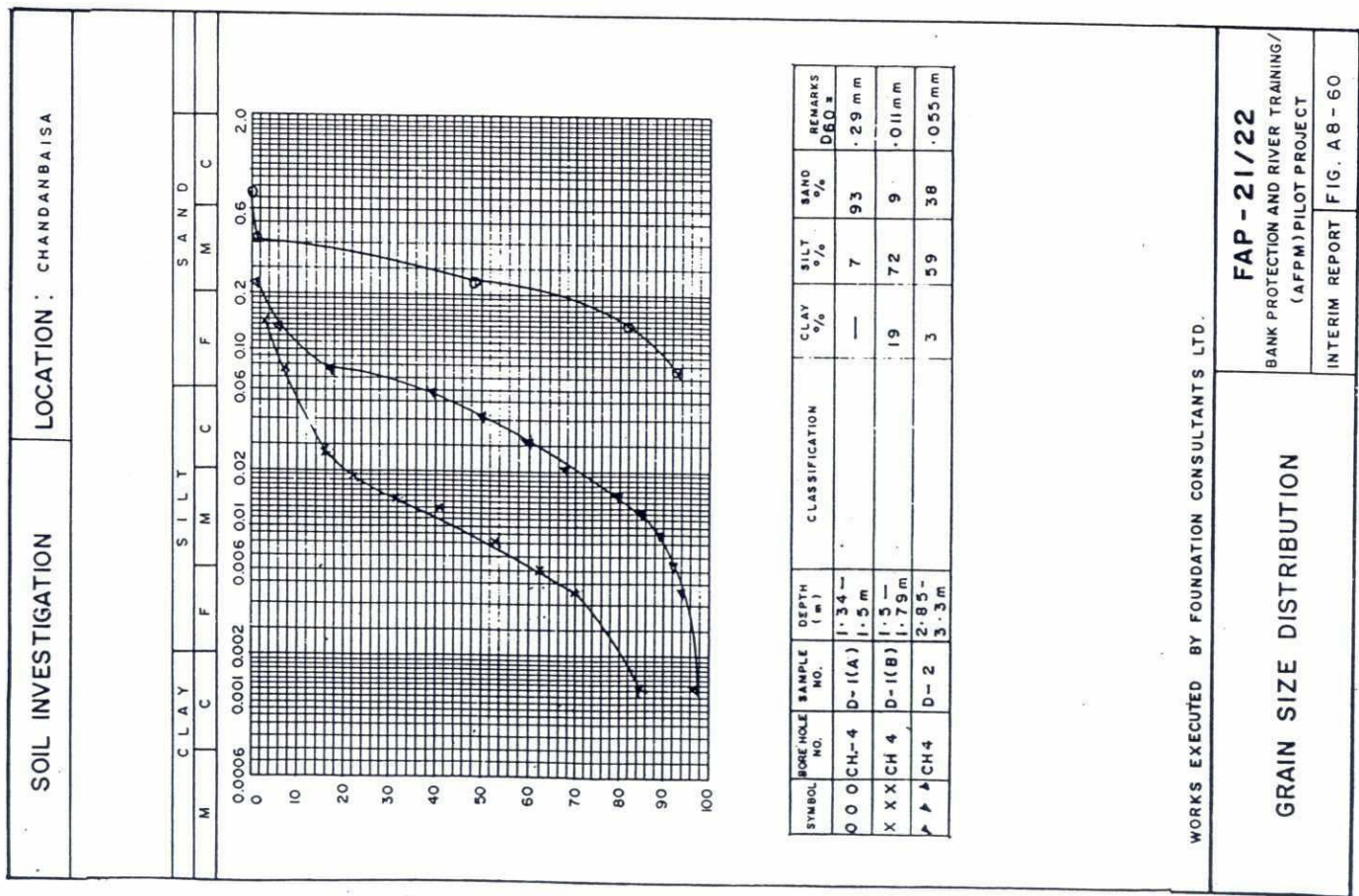
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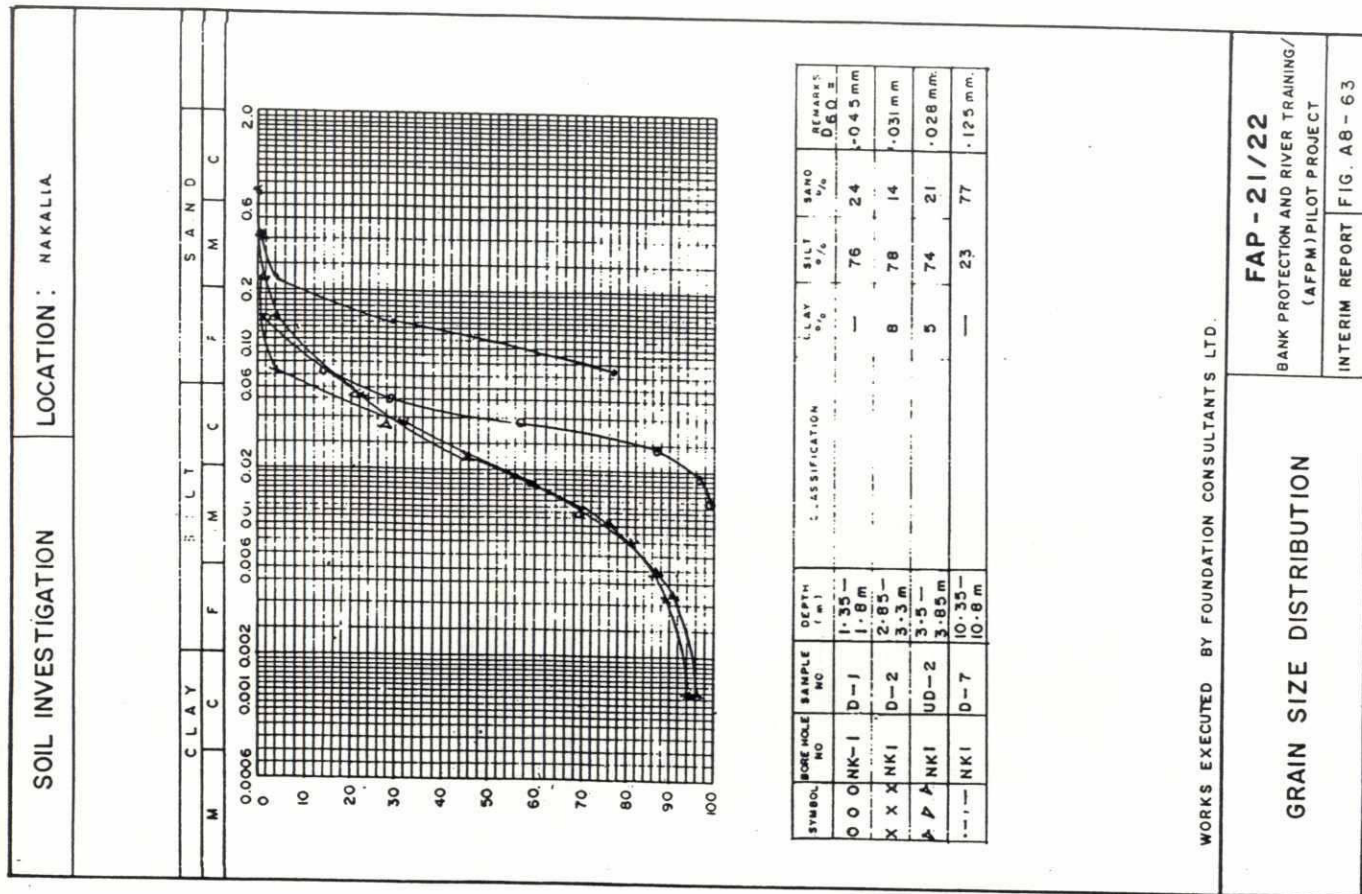
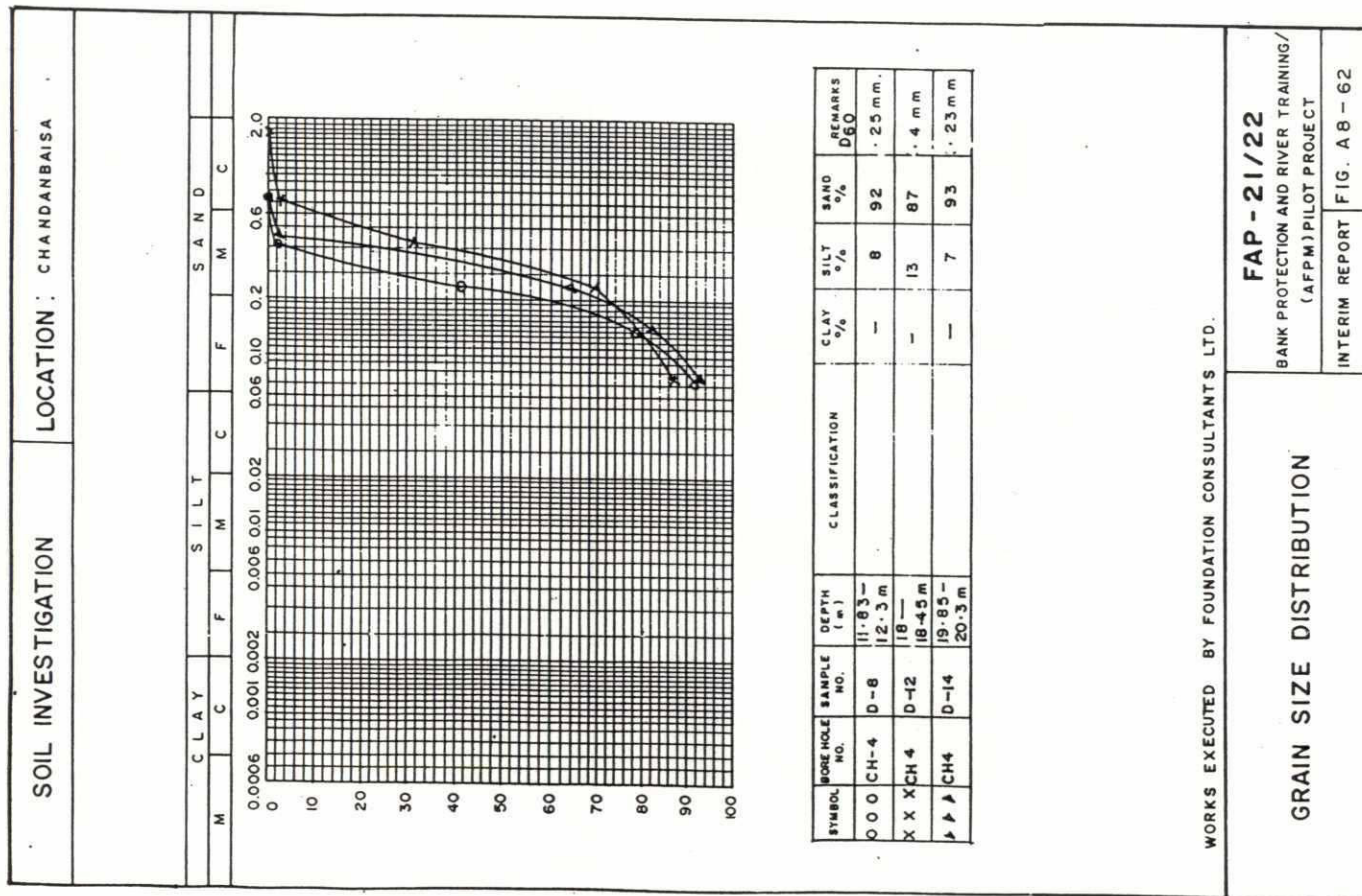
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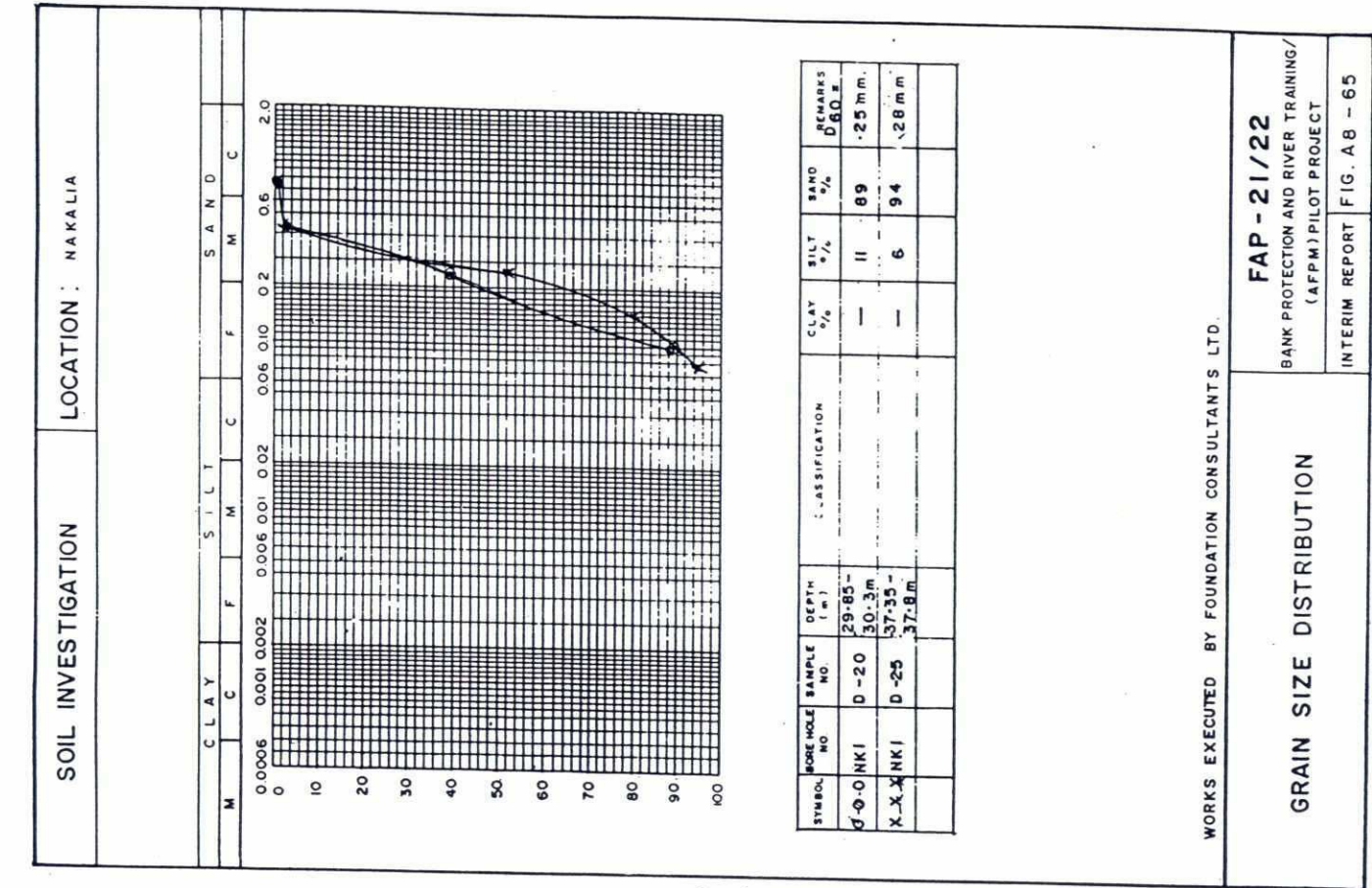
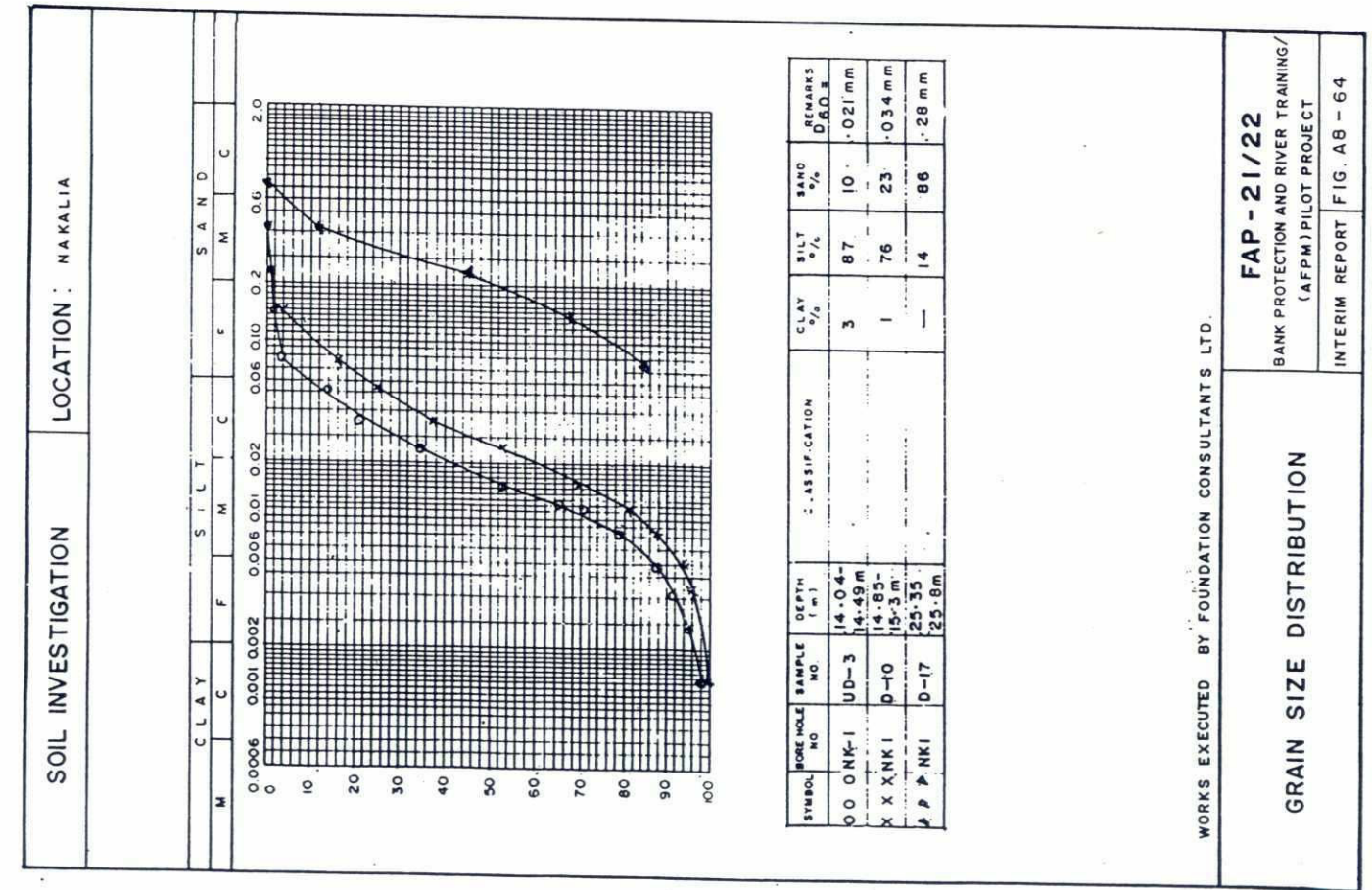
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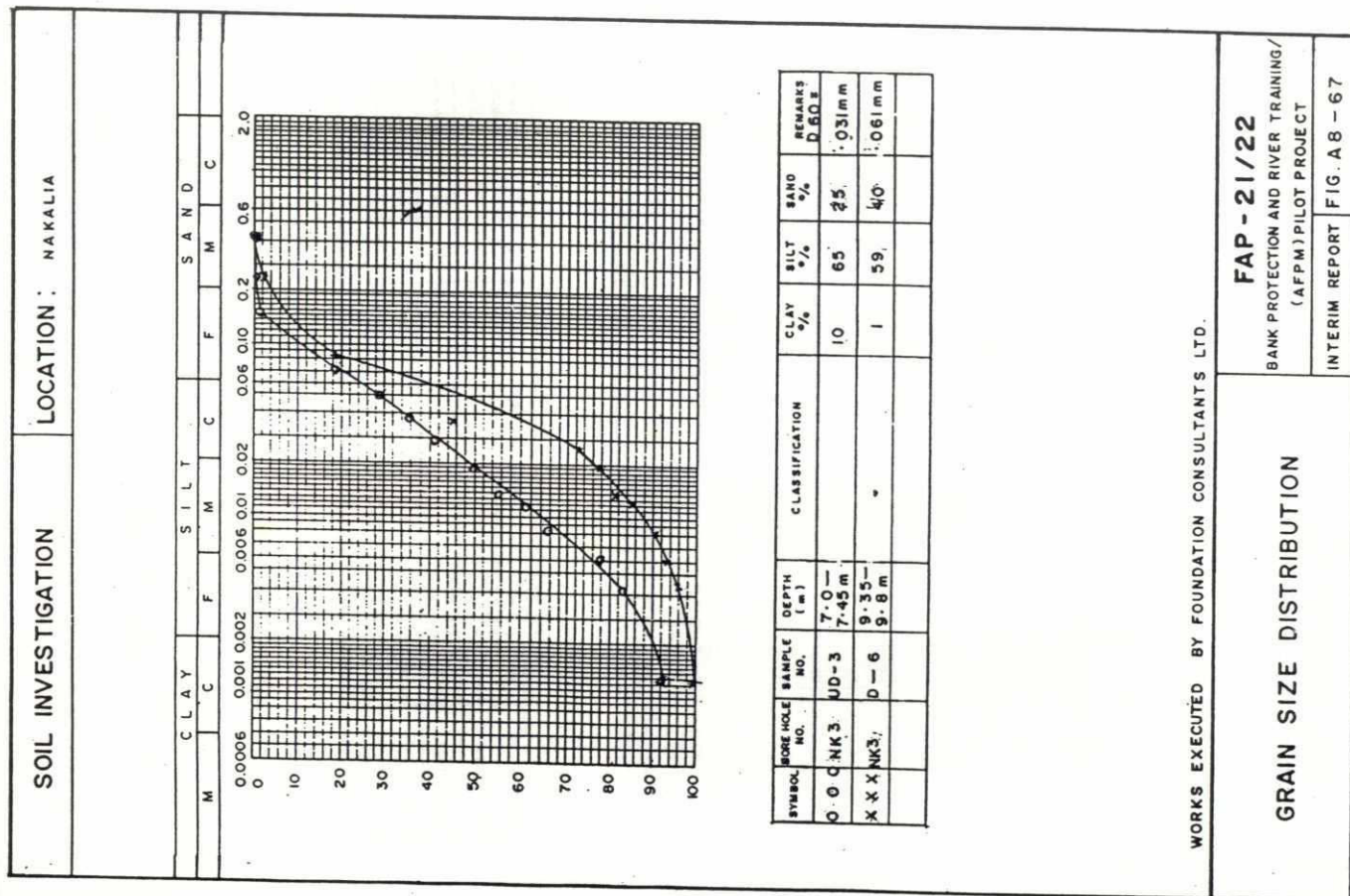
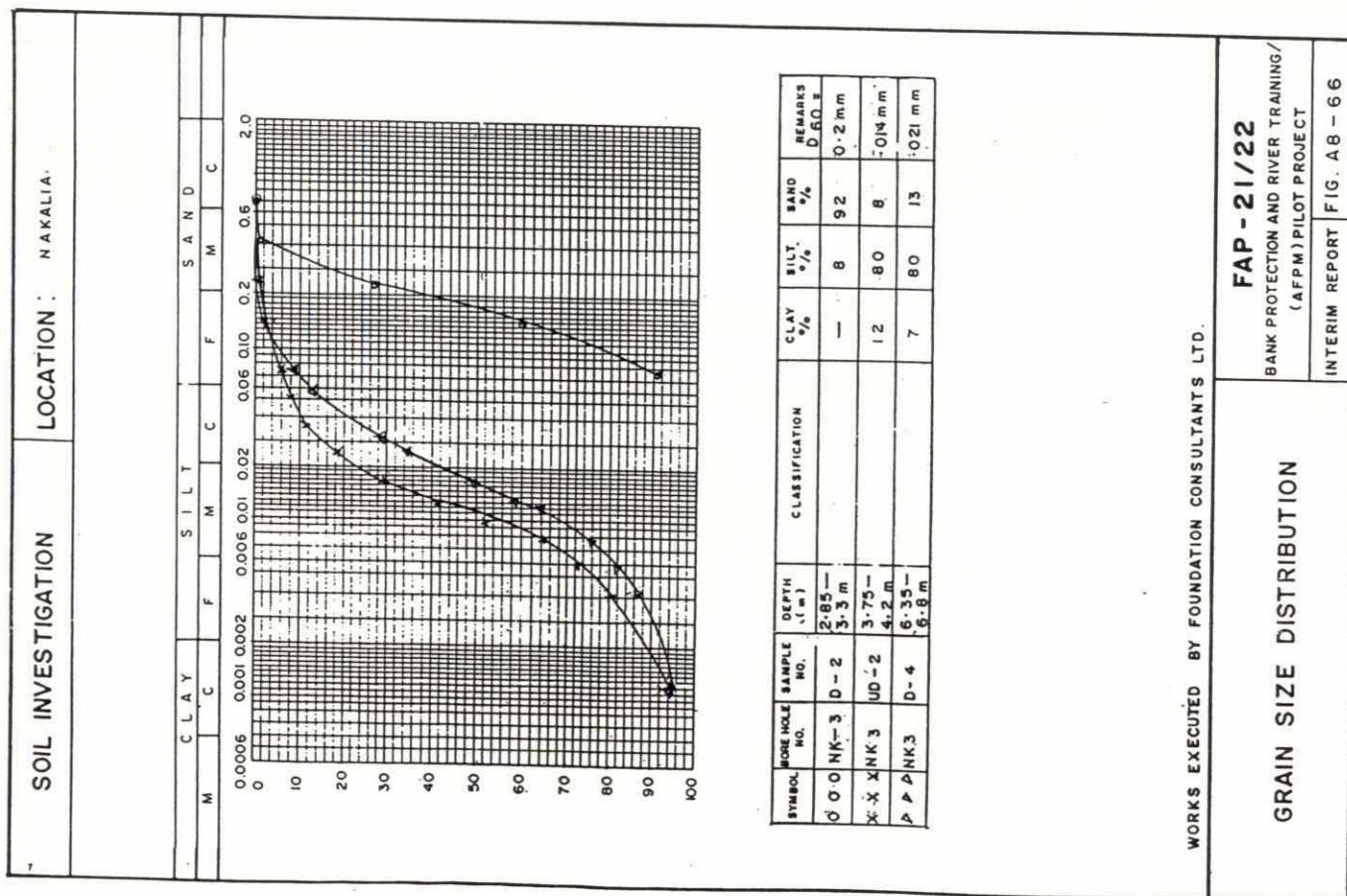
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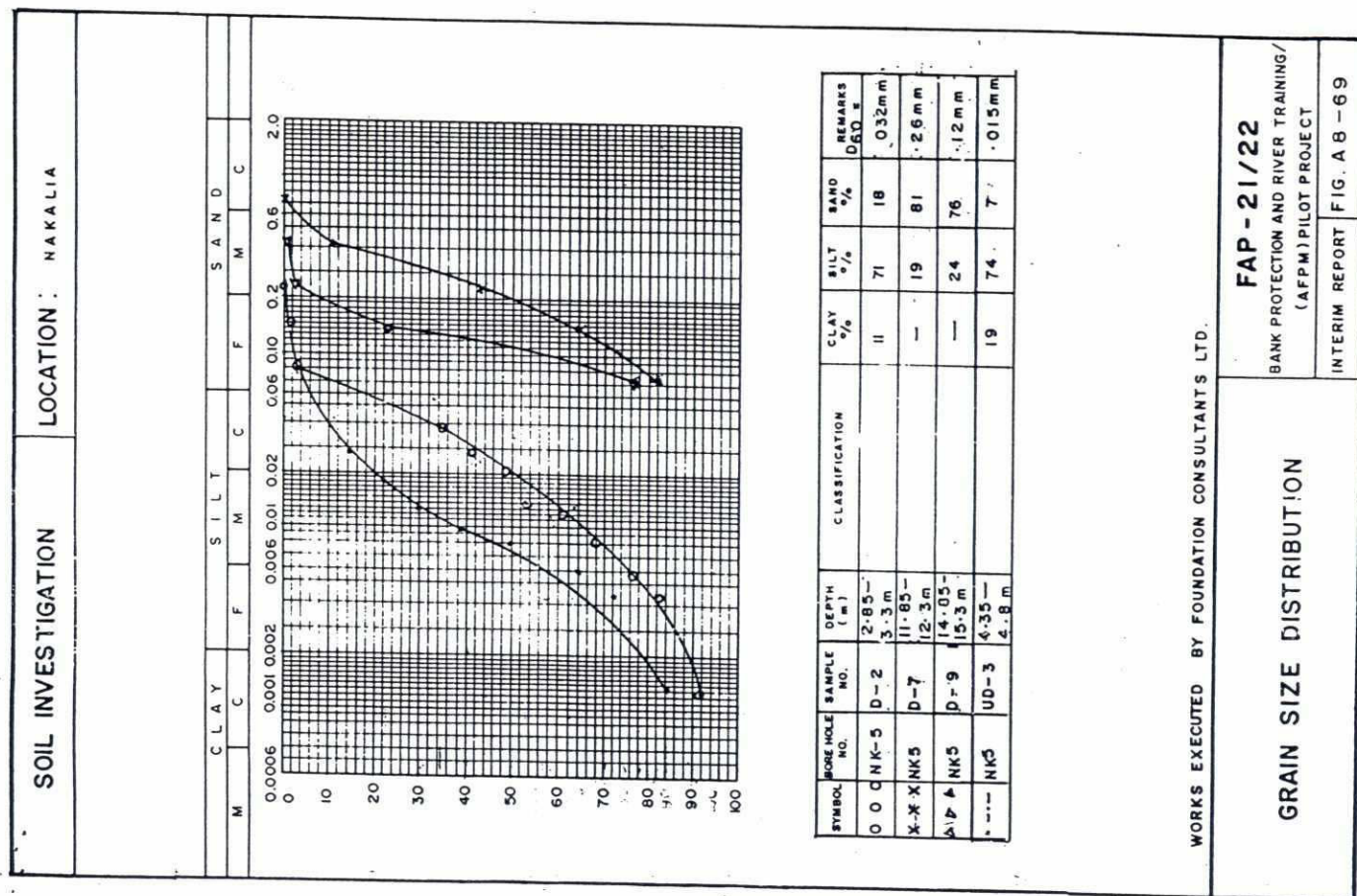
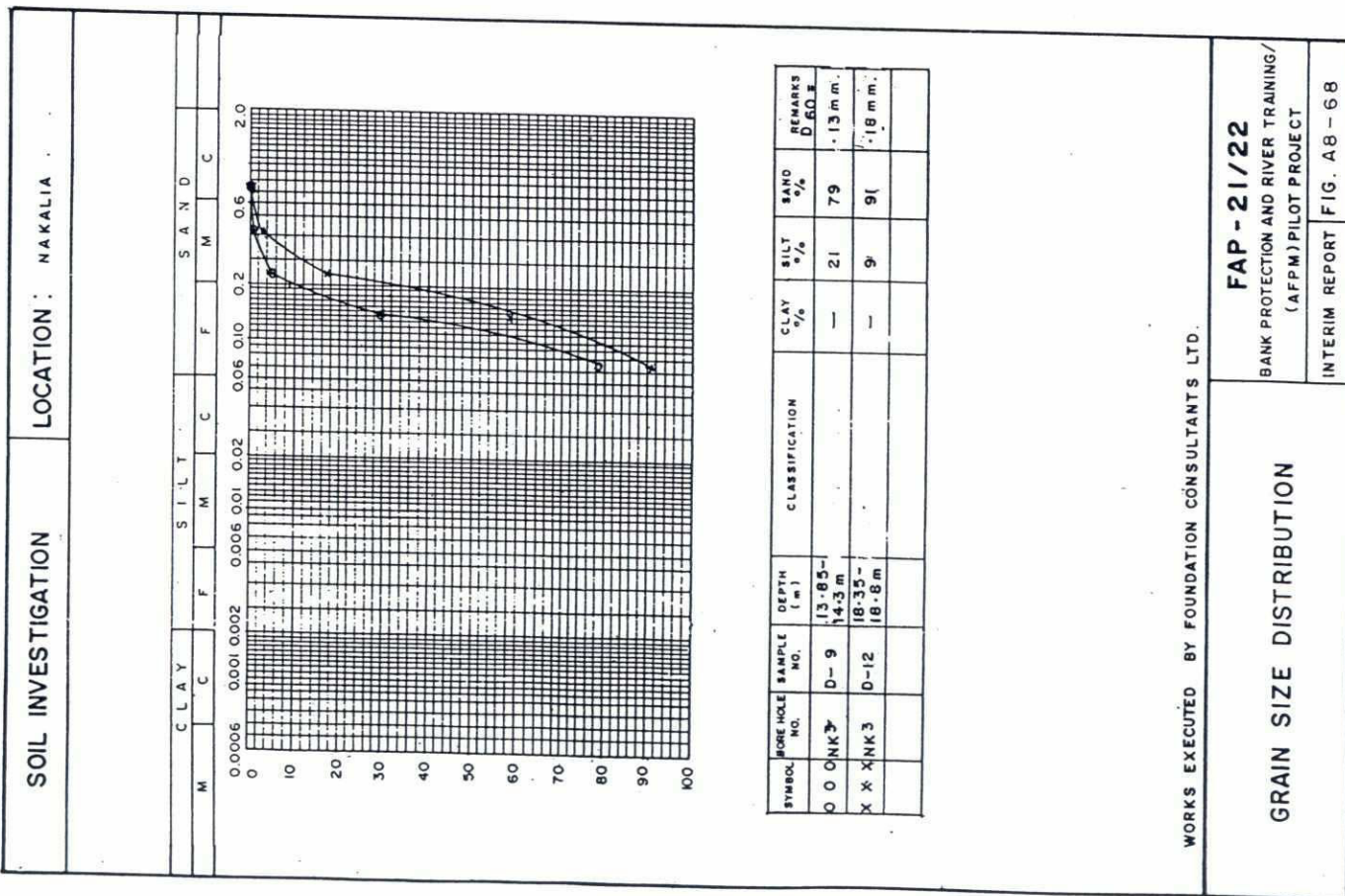
INTERIM REPORT FIG. A8 - 61

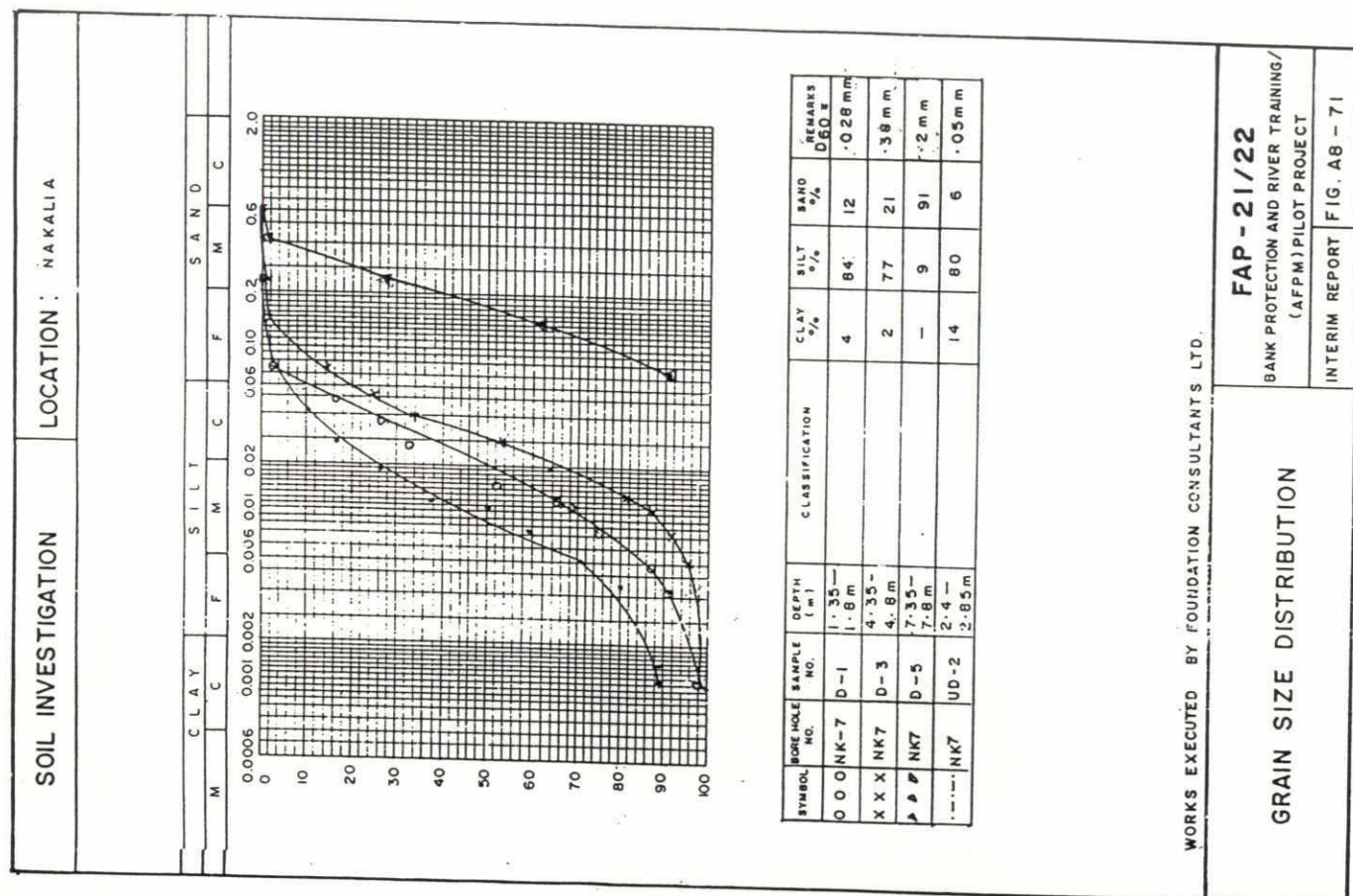
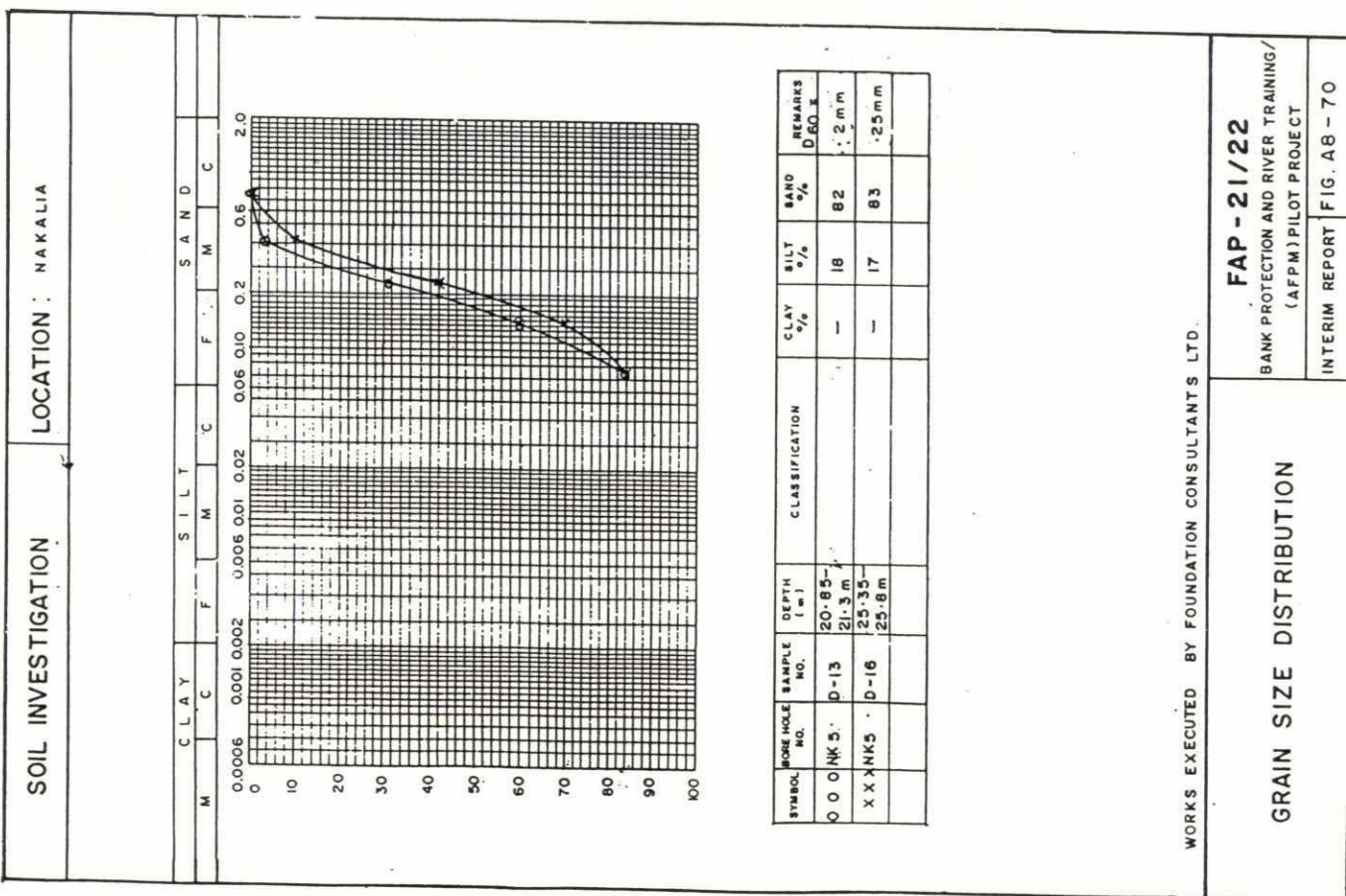


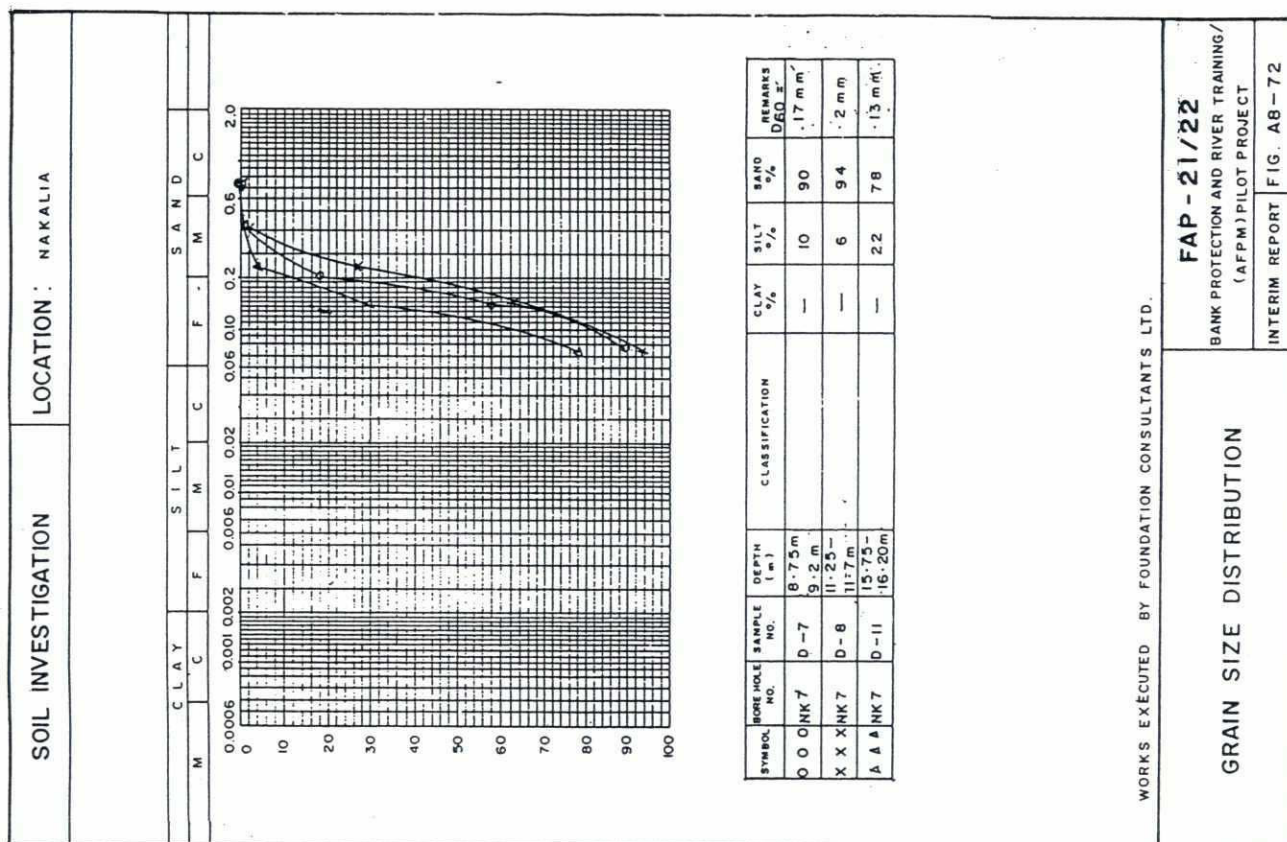
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GRAIN SIZE DISTRIBUTION

ANNEX 9

PRELIMINARY ECONOMIC STUDY

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1 GENERAL

FAP 21/ FAP 22 Project aims at evolving appropriate bank protection techniques and cost-effective methods for works in the Active Flood Plains. As such they are not anticipated to include a full economic analysis of costs and benefits but to provide useful technical and economical data for the preparation of regional development plans and future development projects with a particular attention given to the cost-effectiveness of the recommended measures and construction methods.

Economic aspects are therefore to cover the following subjects:

- i. Analysis of past budgets for bank protection and river training in Bangladesh with due consideration given to their effectiveness;
- ii. Contribution to unit rate and cost analysis for future works possibly involving new construction methods and the use of materials that are not yet familiar to the executing agencies and to local contractors;
- iii. Preliminary inventory of potential benefits to be expected from bank protection and river training works with a rough estimate of their possible order of magnitude and of their economic impact.

In agreement with the terms of reference, the study concentrates mainly on Brahmaputra-Jamuna River although the applicability of the tested works to other rivers in Bangladesh will be also examined.

2 PAST BUDGET ANALYSIS

2.1 AVAILABLE INFORMATION AND SOURCE OF DATA

The past experience of BWDB in bank protection and river training works is reported in a paper published by BWDB in 1984.

The magnitude of the river bank erosion problem in Bangladesh was summarized by BWDB as follows:

RIVER	N° OF LOCATIONS	LENGTH OF EROSION (KM)
Teesta	11	34.9
Brahmaputra-Jamuna	41	162.6
Ganges-Padma	26	94.5
Meghna	8	72.0
Minor rivers	112	92.3
Flashy rivers	75	23.0
Tidal rivers	32	85.8
Overall	305	565.1
Cities & villages	85	

Broadly 22 bank protection or river training schemes were reported in 1988 while others were under execution or at planning stage.

Construction and O & M budgets have not yet been published except for World Bank funded works executed under the Flood Damage Restoration Project. Data on previous cost estimates and actual expenditures since the beginning of works were consequently directly collected from the Executive Engineers, O & M Division, BWDB at Sirajganj, Bogra, Gaibandha, Chandpur and Rajshahi. Government expenditures for bank protection in selected areas were also reported in FAP 9B report. Some discrepancies may be found between the two sources of data due to differences in the definition of funding. Figures presented further are generally those provided by Executive Engineers of BWDB.

Past expenditures in current Taka have been converted into 1991 Constant Taka by using the deflator index published by BBS for the Construction Sub-Sector in years 1972-73 to 1990-1991. The deflator index was not available for previous years and was roughly estimated by the Consultant for years 1969-70 to year 1971-72 using the cost of living index published by the Bangladesh Bureau of Statistics (BBS) as an indicator. The forecast for year 1991-92 has been estimated by extrapolation of the observed trend since 1984-85 (Figure 2.1-1). The correction factors derived from these data and estimates are shown in Table 2.1-1.

2.2 SIRAJGANJ TOWN PROTECTION

Since 1940 bank protection measures were taken, consisting mostly of brick mattressing which revealed effective till 1968 when erosion became very active. After emergency measures taken in 1969, new construction works started, consisting of brick mattressing and of the dumping of loose stones and brick filled wire cages. Later on, steel jacks were dumped and concrete blocks of different sizes were placed without filter due to difficulty in placing khoa; it was thus accepted to spend more for maintenance by dumping additional concrete blocks when required. One groyne was constructed at Sailabari some 12 km upstream of Serajganj in 1978. Another groyne was constructed at Ranigram immediately upstream of the town in 1985-86.

A summary record of expenditures is shown in Table 2.2-1 and Figures 2.2-1 and 2.2-2.

Town protection appears to have been partly effective during 23 years at a total estimated cost of about 1991 Constant Taka 1,180,000,000 including initial construction cost, repairs, improvements, the construction of Sailabari and Ranigram groynes and maintenance.

For town protection itself the repair and maintenance budget has amounted to about 8% of the initial investment. It has been 2 % for Sailabari groyne.

Kazipur groyne was washed away after construction with a total expenditure of 1991 Constant Taka 116,531,000.

2.3 BANK PROTECTION NEAR SARIAKANDI

Protection against river bank erosion has been provided since 1987 to prevent the destruction of trading center and agricultural land. Another major concern is the risk of capture of the

Bangali River by the Jamuna. Protective works included a groyne, cross bars and revetments with brick mattressing and the dumping of concrete blocks over khoa filter. They are presently complemented by revetment with concrete blocks on the left bank of Bangali river.

Past expenditures are shown in Table 2.3-1.

Repairs were required to revetments and apron. Erosion severely increased downstream after the completion of Kalitola groyne.

2.4 BANK PROTECTION NEAR PHULCHARIGHAT

The area has an important rail ferry ghat for goods and passengers and suffered from bank erosion. The Brahmaputra Right Embankment is at risk of being breached in the next years. Protective measures taken included the construction of cross bars and the placing of bamboo porcupines which proved not effective.

Past expenditures are shown in Table 2.4-1.

2.5 BANK PROTECTION WORKS ON RIGHT BANK OF TEESTA RIVER

Bank protection of the right bank of Teesta River has been provided by groynes and cross bars. One groyne and three cross bars were recently washed away while the other structures were considered effective.

Construction and maintenance costs are reported in Table 2.5-1.

2.6 CHANDPUR TOWN PROTECTION WORKS

Chandpur town is located on the left bank of Meghna and has been subject to severe erosion during the last 35 years. Emergency protective works started in 1972 by boulder revetment and the construction of a groyne. Later on, boulder revetment was extended to a total length of about 2,000 m and a regular dumping of boulders is required to maintain the river banks. The groyne was settled down under water in the last years and the town remains threatened by gradual destruction.

Expenditures reported by BWDB since 1972 are shown in Table 2.6-1 and Figure 2.6-1. The detail of annual expenditures is not yet available for years 1972 to 1982 and for years 1979 to 1982. The expenditures in current Taka were supposed evenly distributed within each of the two periods. Their provisional estimate amounts to a total of about Tk 430 Million in 1991 Constant Taka with an O & M budget of about 6% of the initial investment.

2.7 RAJSHAHI TOWN PROTECTION WORKS

The erosion of Ganges-Padma river banks in the vicinity of Rajshahi is reported to have occurred as far back as one century ago or even more. After a shifting of the main channel which stopped erosion for 10 years between 1960 and 1970, erosion started again and required protection measures. Emergency works included the construction or reconstruction

of spurs and groynes. They were followed by a comprehensive programme of works which extended till 1981. Since then, regular maintenance and repairs are provided and make bank protection fairly effective.

Expenditures reported by BWDB since 1971 are shown in Table 2.7-1 and Figure 2.7-1. Only the total of years 1971 to 1981 is presently available. It has been supposed equally distributed in current Taka over the 10 years. Data are still lacking for the six years between 1981 and 1987. The provisional estimate of expenditures amounts to a total of almost Tk 600 Million in 1991 Constant Taka with an O & M budget during the last 5 years of about 3% of the initial investment.

2.8 Overall Protective Works in Bangladesh

As mentioned above, bank erosion is a widespread problem which concerns a number of towns, communication nodes, agricultural schemes and embankment systems. Efforts have already been made with varying success and frequently on emergency circumstances to avoid major disasters.

A comprehensive assessment of past budget allocations for such works would be desirable to fully appreciate the strategical importance of the research undertaken in the field of Bank Protection and River Training. It

requires the cooperation of all BWDB O & M Circles throughout the Country to provide planners with the following information and data:

- i. Inventory and location of all protective works of any kinds undertaken in their area since 1970 and possibly before
- ii. Purpose of construction
- iii. Annual expenditures for design, construction, maintenance and repairs since 1,970 till 1992
- iv. Effectiveness of works
- v. If washed away, when, why and under what circumstances ?

River training expenditures have also been associated with the recent construction of R&H bridge on the Meghna River. They will also be a significant component of Jamuna bridge construction budget.

3 UNIT RATE AND COST ANALYSIS FOR PROTECTION WORKS

3.1 GENERAL

Construction methods presently used in Bangladesh for bank protection and river training are strongly influenced by the availability of suitable materials, equipment and know-how of local contractors. They may result in poor effectiveness and higher final costs with an additional burden with respect of funding for emergency repairs or tentative improvements. The project

is therefore aimed at testing the designs and methods of construction that are the most cost effective. It implies the issue of unit rates corresponding to new technical specifications applicable in Bangladesh. It also requires that economic indicators can be derived from the analysis of construction and recurrent costs with a view to determining and comparing different alternatives.

Suitable indicators can be identified as follows:

- Economic cost determined by applying the proper conversion factor to financial cost
- Respective amount of foreign and local cost
- Use of unskilled labour
- Net Present Value of the economic project costs during the useful life of the project.

Prices will be in 1991 Constant Taka with an exchange rate of US\$1 = Tk.36 to allow easy updating.

3.2 SOURCE OF DATA

For the above elaborations the following sources were used :

- Schedule of rate for the different O & M Circles of BWDB
- Quotations of foreign firms and importers for imported materials
- Other FAP Consultants' estimates
- Interviews with contractors and BWDB field engineers
- Estimation of economic prices of selected commodities for use in FAP planning studies (FPCO, 1992).

3.3 PROPOSED METHOD OF UNIT RATE ANALYSIS

- i. Identification of the items that will be considered in the bills of quantities for preliminary costing and in the call for tenders.
- ii. Technical specifications corresponding to each item.
- iii. For each item, brief study of the method and sequence of work with a view to estimating the requirements of labour, materials, and equipment or machinery per unit of work item.
- iv. Survey of unit cost of labour, materials and equipment at the construction site for the reference year. Unit cost has to be at current market price at the construction site if available , or increased by the transport cost from the most likely procurement area. In the case of special equipment not easily available at site, provision has to be made for its mobilisation in due time possibly at an extra cost.
- v. For each cost element of the item, i.e. labour, material or equipment, the foreign cost component will be given in % and the conversion factor to be applied will be reported. Conversion factors used to convert financial prices into economic prices as requested by FPCO are as follows:

PROJECT INPUTS

CONVERSION FACTORS

1. Skilled labour	0.87
2. Unskilled labour	0.72
3. Transport equipment	0.68
4. Cement	0.79
5. Steel	0.61
6. Machinery (normal duty)	0.62
(concessionary duty)	0.74
7. Others	0.87

- vi. The total of cost elements has to be increased by a certain % to include contractor's overheads, mobilisation, installation at site, taxes and profit to arrive at the final estimate of unit rate. BWDB is recommending to use 12.5 % which seems much too low for works which require highly skilled contractors and imply a certain amount of risk.

The calculation of unit rates and economic indicators is proposed by using the forms shown in Tables 3.3-1 and 3.3-2 as follows:

$$\text{Total unit rate} = \text{Sum}\{(2) \times (3)\}$$

$$\text{Total foreign cost} = \text{Sum}\{(4) \times (5) / 100\}$$

$$\text{Average Conversion factor} = \text{Sum}\{(4) \times (6)\} / \text{Sum}\{(2) \times (3)\}$$

- with
- (2) = unit cost of materials, labour or equipment
 - (3) = quantity of materials, labour or equipment
 - (4) = (2) × (3)
 - (5) = % of foreign cost for materials or equipment
 - (6) = conversion factor of cost component (as above)

3.4 COST ANALYSIS

Following FPCO Guidelines for Project Assessment, base construction costs estimated from bill of quantities and unit rates are to be expressed in mid-1991 Constant Taka. Physical contingencies equal to 15 % of base construction costs will be used to cover unforeseen works, and engineering costs will be added.

The construction costs will be phased by years and further maintenance, repair and improvement costs will be forecast for a useful life of 30 years following the completion of works.

After conversion of financial costs to economic costs, the Net Present Value of expenditures over 30 years will be calculated using a discount rate of 12 %. The comparison between the different construction alternatives proposed and the previous construction methods will be based on the following criteria:

- Net Present Value of expenditure at economic price,
- Net Present Value of expenditure in foreign and local currency,
- Average annual financial expenditure after initial investment,
- Annual use of unskilled labour.



4 POTENTIAL BENEFITS

4.1 SOURCE OF DATA

Technical aspects of bank protection and river training have been covered in BWDB publications but benefit assessment was not deemed necessary till recently and bank protection schemes were generally decided in the past with the conviction that there was an urgent need for such measures and that their justification was obvious.

Population, agriculture and livestock data are found at the scale of Unions in BBS publications for years 1981 and 1983-84. They can be used to roughly identify the areas and the population particularly concerned with river bank erosion.

The most valuable information on potential benefits of protection works has been produced by recent FAP studies:

- FAP 1 : Brahmaputra Right Embankment Strengthening
- FAP 9B: Meghna Left Bank Protection Project.

An economic assessment of river bank protection is provided by a Consultant's report dated December 1991 for 6 selected priority locations on Jamuna Right Bank (River Training Studies of the Brahmaputra River, Report on Priority Works). The economic analysis of town, communication infrastructure and irrigation schemes protection projects is included in the report of FAP 9B.

4.2 ACTIVE FLOOD PLAIN AREA

In the conditions of Bangladesh, where River floods cover very extensive areas and river banks are highly unstable, the boundaries of the Active Flood Plain are neither obvious nor constant. As a result of past events, the Active Flood Plain includes all the islands and new land deposits that are between the present river banks; for planning purposes it has to also include the areas that are at risk of being eroded by the River within the next 30 years. The limits of these latter areas are still more or less unpredictable and therefore have to be assumed as an envelope of the study area. For practical reasons they are tentatively assumed to coincide with the administrative boundaries of Unions bordering the River.

The content of the study area (Brahmaputra-Jamuna) is given in Table 4.2-1.

It covers a total area of 3,670 km² over a strip length of about 213 km and includes uncultivable land and open water areas estimated at about 1720 km². A total of 306,500 households were living there as from the 1981 census; this number has probably increased by 20 to 30 % since then.

The net agricultural income derived from the 163,000 ha of cultivated land is not known but its order of magnitude can be roughly estimated by assuming an average added value of about Tk 10,000 per cropped hectare. It thus amounts to about Tk 3,000,000,000 per year.

Available information could let suppose that annual land losses due to bank erosion along Brahmaputra-Jamuna are around 1,500 ha. It means that approximately the equivalent of the cultivable study area could disappear within the next century and would be very partly replaced by new "char" lands. The economic losses directly due to the erosion of agricultural lands along Brahmaputra-Jamuna cannot be assessed without much more detailed studies, but their order of magnitude in the long term could reach or overpass the billion of Taka per year.

Agricultural income outside the Active Flood Plain is also affected by the erosion of flood protection infrastructure as demonstrated by FAP 1 study.

Other sectors of the economy like industry, trade and services are also present in the Active Flood Plain. At the regional level, total net income may represent 2.5 to 3 times the income of the crop sub-sector. Although these ratio cannot be applied to the study area without more detailed analysis, it is obvious that the impact of the negative river actions on the overall local economy is much higher, possibly 2 times more, than that on the crop sub-sector.

Communications at the national level are strongly affected by the conditions in the study area which is in the same time a communication axis for water transport and an obstacle for land transport. The benefits which could be expected from improved navigability are beyond the scope of the present study but are worth mention. Other aspects are discussed further.

4.3 DIRECT BENEFITS OF BANK PROTECTION

The consequences of river banks erosion are identified as follows:

1. Land losses, though at least partly compensated by the accretion of new lands frequently of low value
2. Destruction of public and private property
3. Destruction of infrastructure with further consequences, especially in the case of flood protection or communication infrastructure
4. Disruption of economic activities
5. More or less adequate investments following emergency decisions
6. Obstacle to the development of communications and consequently to the overall economic development of the Country.

The assessment of bank protection benefits has been made for 5 selected areas on the Right Bank of Jamuna by FAP 1 Consultants. The results are shown in Table 4.3-1. In the selected areas, the expected annual benefits vary between about Tk million 4.6 and 12.6 per km of erosion prone bank. They are mostly agricultural benefits except for the township of Sirajganj. The highest benefits were found for Sariakandi where there is a risk of capture of Jamuna by a relatively small distributary. Except for Sirajganj area, the economic consequences of river bank erosion extend outside the erosion prone area for more or much more than 50%.

The results of benefit assessment by FAP 9B are summarized in Table 4.3-2. They concern 7 selected locations on the Left Bank of Meghna. Three of them are agricultural project areas

while the others are aimed at protecting active urban centers and communication nodes. The method of assessment is fairly different of that used in FAP 1 and makes difficult to compare the results between the two studies. The order of magnitude of expected benefits is however roughly the same. Communication and industrial infrastructure appear to be key factors in the identification of priority areas for river training and bank protection.

4.4 OTHER POTENTIAL BENEFITS

Available benefit assessments concern selected locations where river bank erosion threatens existing economic resources, properties and infrastructure. They are based on the present development of activities and eventually on present development trends. This approach is fully justified for projects that are planned in the short term.

When planning for the medium and long term, it appears necessary to envisage the interactions between river training and communications. The improvement of physical links between the different regions of Bangladesh is probably one of the main requirements for the accelerated development of industry and services which is needed in most areas. It is presently restricted and made more costly by the morphology of the main rivers. River training thus appears as a component of any comprehensive national development plan with benefits which can hardly be considered in isolation and without prospective views.

A good illustration of the scale of future river training requirements is provided by the Jamuna bridge project which is under short term planning. The cost of river training within this project has been estimated on behalf of UNDP/WB at 1987 Constant US\$ 94 Million equivalent to 1991 Constant Tk 3.6 billion or approximately 25% of total project costs.

TABLE 2.1-1
CORRECTION FACTOR FROM
CURRENT TAKA TO
1991 CONSTANT TAKA

Year	Constr. deflator	Correct. factor
1 1969-70	12.00	13.022
2 1970-71	15.00	10.417
3 1971-72	18.00	8.681
4 1972-73	18.18	8.595
5 1973-74	28.25	5.531
6 1974-75	50.33	3.105
7 1975-76	57.58	2.714
8 1976-77	51.30	3.046
9 1977-78	52.49	2.977
10 1978-79	55.09	2.836
11 1979-80	66.77	2.340
12 1980-81	78.35	1.994
13 1981-82	91.67	1.705
14 1982-83	89.93	1.738
15 1983-84	90.15	1.733
16 1984-85	100.00	1.563
17 1985-86	113.75	1.374
18 1986-87	117.86	1.326
19 1987-88	125.94	1.241
20 1988-89	136.25	1.147
21 1989-90	144.91	1.078
22 1990-91	156.26	1.000
23 1991-92	168.00	0.930

SOURCE: BBS for Years 1972-73
to 1990-91
Consultant's estimate
for Years 1969-70 to
1971-72 and 1991-92

TABLE 2.2-1
BANK PROTECTION WORKS NEAR SIRAJGANJ

Year	Serajganj town		Sailabari groyne		Beara cross bar		Kazipur groyne	
	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000
1969-70	28791	374905						
1970-71	3963	41284						
1971-72	3712	32226						
1972-73	13158	113091						
1973-74	3718	20567						
1974-75	3029	9403						
1975-76	2998	8136						
1976-77	5054	15395						
1977-78	12948	38546	35000	104193				
1978-79	1902	5394	1429	4053				
1979-80	8230	19260	1429	3344				
1980-81	32759	65335	1429	2850				
1981-82	4445	7577	1429	2436				
1982-83	2535	4406	1429	2483				
1983-84	7166	12421	1429	2477				
1984-85	3020	4719	1429	2233				
1985-86	68175	93653	1429	1963				
1986-87	36880	48896	1429	1895				
1987-88	49565	61497	1429	1773	11900	14765	45000	55834
1988-89	29697	34058	1429	1639	6637	7612	32685	37485
1989-90	16390	17674	1429	1541	0	0	4868	5249
1990-91	10280	10280	1429	1429	5997	5997	17963	17963
1991-92	5995	5576	1429	1329	0	0	Washed away	
Total		1044300		135638		28374		116531
Number of years		23		15		5		4
NPV (12%)		565977		108252		23062		94887
Annual average after 1st investm.		30427		2246		3402		20232

. Net Present Value calculated for the year preceding construction

. Maintenance cost of Sailabari groyne reported at about Current Tk20,000,000
since construction without annual detail

TABLE 2.3-1
BANK PROTECTION WORKS NEAR SARIAKANDI

Year	Kalitola groyne		Antarpur x bar		Hasnapur x bar		Pakulla x bar		Chalkandi x bar	
	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000
1969-70										
1970-71										
1971-72										
1972-73										
1973-74										
1974-75										
1975-76										
1976-77										
1977-78										
1978-79										
1979-80										
1980-81										
1981-82										
1982-83										
1983-84										
1984-85										
1985-86										
1986-87										
1987-88	38025	47180							10908	13534
1988-89	28945	33196			656	752			0	0
1989-90					93	100	358	386	0	0
1990-91	4989	4989			1537	1537	992	992	0	0
1991-92									0	0
Total		85364				2390		1378		13534
Number of years		5				4		3		5
NPV		71759				1846		1135		12084
Annual average after 1st investm.										
to be complemented										

TABLE 2.4-1
BANK PROTECTION WORKS NEAR PHULCHARIGHAT

Year	Porcup. at Pulch.		Phulchari x bars		Gazaria x bar		Phulchari x dam	
	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000
1969-70								
1970-71								
1971-72								
1972-73								
1973-74								
1974-75								
1975-76								
1976-77								
1977-78								
1978-79								
1979-80								
1980-81								
1981-82								
1982-83								
1983-84								
1984-85								
1985-86								
1986-87								
1987-88								
1988-89	32785	37600						
1989-90	484	522						
	10622	10622						
1991-92								
Total		48744						
Number of years		4						
NPV		41548						
Annual average after 1st investm.								
to be complemented								

TABLE 2.5-1
BANK PROTECTION WORKS ON RIGHT BANK OF TEESTA RIVER

Year	Belka x bars		Tarapur x bar I		T. x bar II & III		Tambulpur groyne		Pinalghat groyne	
	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000	Current Tk,000	1991 Constant Tk,000
1969-70										
1970-71										
1971-72										
1972-73										
1973-74										
1974-75										
1975-76										
1976-77										
1977-78										
1978-79										
1979-80										
1980-81										
1981-82										
1982-83										
1983-84										
1984-85										
1985-86										
1986-87										
1987-88	21072	26145	6340	7866	12581	15610	3674	4559	15227	18893
1988-89	2579	2958	698	801			12952	14854	2992	3431
1989-90	0	0					12209	13165	13651	14720
1990-91									70	
1991-92										
Total		29103		8667		15610		32578		37045
Number of years										
NPV		25702		7662		13937		25283		30082
Annual average after 1st investm.										
to be complemented										

TABLE 2.6-1
CHANDPUR TOWN PROTECTION WORKS

Year	Bank revetment and groyne	
	Current Tk,000	1991 Constant Tk,000
1969-70		
1970-71		
1971-72		
1972-73)	9470	81398
1973-74)	9470	52383
1974-75)	9470	29402
1975-76)	9470	25700
1976-77)	9470	28846
1977-78)	9470	28192
1978-79	1481	4201
1979-80)	783	1833
1980-81)	783	1562
1981-82)	783	1335
1982-83	2300	3996
1983-84	1240	2149
1984-85	1661	2595
1985-86	2267	3114
1986-87	7811	10356
1987-88	13600	16874
1988-89	14487	16615
1989-90	103913	112052
1990-91	5666	5666
1991-92	10	9
Total		428280
Number of years		20
NPV		211208
Annual average after 1st investm.		14027
. Groyne was washed away in the last years		

TABLE 2.7-1
RAJSHAHI TOWN PROTECTION WORKS

Year	Bank revetment and groynes	
	Current Tk,000	1991 Constant Tk,000
1969-70		
1970-71		
1971-72)	12638	109708
1972-73)	12638	108622
1973-74)	12638	69903
1974-75)	12638	39236
1975-76)	12638	34296
1976-77)	12638	38494
1977-78)	12638	37621
1978-79)	12638	35846
1979-80)	12638	29575
1980-81)	12638	25204
1981-82		
1982-83		
1983-84		
1984-85		
1985-86		
1986-87		
1987-88	6382	7919
1988-89	16833	19305
1989-90	347	374
1990-91	21801	21801
1991-92	17156	15957
Total		593863
Number of years		21
NPV		288922
Annual average after 1st investm.		13071
to be complemented		

TABLE 3.3-1

COST ANALYSIS

Item N°

LOCATION :

DESCRIPTION :

1. Construction method (Sequence of works)

Description		Unit	Unit cost Tk	Quantity per Unit	Unit rate Tk	Foreign cost (%)	Convers. factor	Unsk.lab. requir. (mandays)
		(1)	(2)	(3)	(4)	(5)	(6)	(7)
2. Materials								
	2.1				0			
	2.2				0			
	2.3				0			
	2.4				0			
	2.5				0			
	2.6				0			
	2.7				0			
	2.8				0			
3. Labour requirements								
	3.1 Unskilled labour				0			0
	3.2				0			
	3.3				0			
	3.4				0			
	3.5				0			
	3.6				0			
	3.7				0			
	3.8				0			
4. Machinery and equipment requirements								
	4.1				0			
	4.2				0			
	4.3				0			
	4.4				0			
	4.5				0			
	4.6				0			
	4.7				0			
	4.8				0			
5.	Sub-total				0			
6.	Contractor's overheads & profit		%		0		0.87	
7.	Total rate (Taka/unit)				0	ERR	ERR	0

TABLE 3.3-2 (example)

COST ANALYSIS Item N° LOCATION : Kazipur UNIT : m3

DESCRIPTION : Supply of 1 m3 of CC Blocks 55cm size

1. Construction method (Sequence of works)

Preparation of steel forms and casting yard for casting CC cubes
 Supply of materials, mixing equipment, vibrator and water pump
 Shuttering for CC cubes
 Casting, curing and stocking of CC blocks

Description		Unit	Unit cost Tk	Quantity per Unit	Unit rate Tk	Foreign cost (%)	Convers. factor	Unsk.lab. requir. (mandays)
		(1)	(2)	(3)		(5)	(6)	(7)
2. Materials								
2.1	Cement	bag	220	4.25	935	90	0.79	
2.2	Bricks	No	2.1	295	620	0	0.87	
2.3	Sand	m3	212	0.47	100	0	0.87	
2.4	Polythene sheet	m ²	2.2	11.3	25	0	0.75	
2.5	Fuel	l	15	9.6	144	90	0.63	
2.6	Lubricant	l	39	0.1	4	90	0.63	
2.7					0			
2.8					0			
3. Labour requirements								
3.1	Unskilled labour	day	60	4.25	255	0	0.72	4.25
3.2	Helper	day	80	0.16	13	0	0.72	0.16
3.3	Mason	day	100	0.2	20	0	0.87	
3.4	Mechanic/Operator	day	120	0.2	24	0	0.87	
3.5	Head mason	day	140	0.035	5	0	0.87	
3.6	Welder	day	120	0.045	5	0	0.87	
3.7					0			
3.8					0			
4. Machinery and equipment requirements								
4.1	Mixture machine 0.5 m3 capacity	hr	125	0.5	63	60	0.62	
4.2	Electric vibrators	hr	8	1	8	60	0.62	
4.3	Motor pump with appliances	hr	4	0.15	1	60	0.62	
4.4					0			
4.5					0			
4.6					0			
4.7					0			
4.8					0			
5.	Sub-total	m3			2220			
6.	Contractor's overheads & profit	25 %	m3		555		0.87	
7.	Total rate (Taka/unit)	m3			2775	1017	2243	4.41

Source : FAP 1, BRTS, 2nd Interim Report, Annex 5

TABLE 4.2-1
CHARACTERISTICS OF THE ENVELOPPE STUDY AREA (BRAHMAPUTRA-JAMUNA)

District	Upazila	Union	Gross area (ha)	Cultivated area (ha)	Cropped area (ha)	Cropping intensity	Number of households	Net crop income (Tk,000)	
KURIGRAM	NAGESWARI	Narayanpur	5494	1539	2658	1.73	2073	32	
		Nookhawa	3564	1185	2832	2.39	1714	34	
	KURIGRAM	Jatrapur	6883	1761	3425	1.94	2105	41	
		Ghogadaha	2714	1574	3044	1.93	3051	37	
		Panchgachhi	2413	1975	3686	1.87	2663	44	
	ULIPUR	Begumganj	14691	3350	6027	1.80	3997	72	
		Buraburi	3979	2021	4028	1.99	3680	48	
		Hatia	3064	1783	3441	1.93	3938	41	
	CHILMARI	Nayarhat	5198	1260	2120	1.68	1212	25	
		Chilmari	2693	375	688	1.84	791	8	
		Ashtamir Char	7498	2217	3551	1.60	2433	43	
	RAUMARI	Dantbhanga	4296	3165	6026	1.90	4101	72	
		Bandaber	5941	4389	7212	1.64	4997	87	
		Raumari	3639	2869	5542	1.93	4331	67	
	CHAR RAJIBPUR	Jadurchar	3509	2609	4480	1.72	3246	54	
		Kodail Kati	2509	1605	3129	1.95	2035	38	
		Char Rajibpur	3132	2302	4377	1.90	3690	53	
		Mohanganj	5463	1873	3624	1.94	3117	43	
		Kamarjani	3130				1435	0	
District sub-total			89811	37853	69892	1.85	54609	839	
GHAIBANDA	SUNDARGANJ	Kapasia	5986	1455	1907	1.31	2211	23	
	GHAIBANDA	Kamarjani	3211	1505	2948	1.96	2140	35	
		Erendabari	8884	3486	5615	1.61	3697	67	
	PHULCHARI	Fazlupur	6888	2002	2122	1.06	2364	25	
		Uria	2361	1487	2565	1.72	2334	31	
		Gazaria	2878	1156	2094	1.81	3042	25	
		Phulchari	4955	1952	2895	1.48	1966	35	
		SUGHATTA	Haldia	4681	1494	2004	1.34	2725	24
	District sub-total			39845	14538	22151	1.52	20479	266

(continued)

TABLE 4.2-1 (Continued)
CHARACTERISTICS OF THE ENVELOPPE STUDY AREA (BRAHMAPUTRA-JAMUNA)

District	Upazila	Union	Gross area (ha)	Cultivated area (ha)	Cropped area (ha)	Cropping intensity	Number of households	Net crop income (Tk,000)
JAMALPUR	DEWANGANJ	Danghdara	4228	2428	4430	1.82	3073	53
		Char Amkhada	3711	2648	5054	1.91	3518	61
		Hatibhanga	1869	1444	1774	1.23	2042	21
		Bahadurabad	4336	2571	4459	1.73	3817	54
		Chikajani	3471	1493	2431	1.63	3043	29
		Chukaibari	2779	822	1213	1.48	2788	15
	ISLAMPUR	Kulkandi	2843	970	1517	1.56	2133	18
		Belgachha	3540	1578	2300	1.46	2801	28
		Sapdhari	3068	1670	2675	1.60	2633	32
		Noapara	3871	2283	3506	1.54	3804	42
	MADARGANJ	Char Pakerdaha	3365	2479	4009	1.62	3882	48
		Balijuri	3977	2266	4083	1.80	5871	49
	Satpoa	4679	3636	6415	1.76	5928	77	
	Pogaldigha	4000	2505	4310	1.72	6027	52	
	Aona	3057	1771	3370	1.90	4245	40	
	Pingna	2915	1709	3180	1.86	3706	38	
	District sub-total			59963	34772	59399	1.71	62924
BOGRA	SARIAKANDI	Chaluabari	5850				1563	0
		Hatsherpur	2757	1272	2021	1.59	2750	24
		Kazla	6681	1798	3011	1.67	2048	36
		Sariakandi	2561	1446	2845	1.97	3722	34
		Karnibari	5229	1355	2205	1.63	3349	26
		Bohall	6943	1229	2252	1.83	2910	27
	District sub-total			30020	7100	12334	1.74	16342

(continued)



TABLE 4.2-1 (Continued)
CHARACTERISTICS OF THE ENVELOPPE STUDY AREA (BRAHMAPUTRA-JAMUNA)

District	Upazila	Union	Gross area (ha)	Cultivated area (ha)	Cropped area (ha)	Cropping intensity	Number of households	Net crop income (Tk,000)
SIRAJGANJ	KAZIPUR	Khas Rajbari	3659	1223	2280	1.86	1428	27
		Char Girish	6828	2038	3387	1.66	2753	41
		Maizbari	1972	1513	2074	1.37	4207	25
		Natuarpara	2728	1275	2457	1.93	2532	29
		Nischintapur	3635	1196	2042	1.71	1435	25
	SIRAJGANJ	Tekani	3303	1001	1758	1.76	2215	21
		Mechhra	4496	1666	3028	1.82	4096	36
		Kaoakola	3403	1101	2103	1.91	1789	25
		City	1956				17313	0
		Kaliaharipur	2726	1533	3009	1.96	4603	36
	BELKUCHI	Saidabad	2793	1401	2745	1.96	4089	33
		Rajapur	2811	2030	4055	2.00	5507	49
		Belkuchi	2652	970	1913	1.97	6412	23
	CHAUHALI	Baradhul	3128	730	1473	2.02	1985	18
		Sadiachandpur	5176	764	1258	1.65	4162	15
		Sthal	4185	1248	2551	2.04	2047	31
		Ghorjan	2994	914	1916	2.10	1798	23
		Mirkutia	4870	3735	6924	1.85	5725	83
		Umarpur	5975	1726	3249	1.88	4263	39
District sub-total			69292	26065	48222	1.85	78359	579
PABNA	BERA	Haturia Nakalia	3053	1713	3277	1.91	4123	39
		Nutan Bharenga	4197	2563	5049	1.97	4804	61
		Puran Bharenga	2643	1120	2084	1.86	1843	25
		Ruppur	2977	1619	2738	1.69	2755	33
		Masundia	2819	1333	2152	1.61	2274	26
		Dhalar Char	3875	1370	2900	2.12	1674	35
District sub-total			19563	9719	18201	1.87	17473	218

(continued)

TABLE 4.2-1 (Continued)
CHARACTERISTICS OF THE ENVELOPPE STUDY AREA (BRAHMAPUTRA-JAMUNA)

District	Upazila	Union	Gross area (ha)	Cultivated area (ha)	Cropped area (ha)	Cropping intensity	Number of households	Net crop income (Tk,000)
TANGAIL	BHUPUR	Arjuna	3975	2057	4306	2.09	3837	52
		Gabsara	6903	1842	3570	1.94	3650	43
		Nikrail	4439	1715	3664	2.14	2885	44
	KALIHATI	Durgapur	5172	3164	6730	2.13	4684	81
		Salla	2747	1944	3698	1.90	4152	44
	TANGAIL	Kakua	3796	2478	5187	2.09	4072	62
		Hugra	3452	2419	4939	2.04	3247	59
		Katuli	4943	3097	5512	1.78	4843	66
	NAGARPUR	Bharra	3077	2645	5569	2.11	3467	67
		Gayhata	2568	1810	4301	2.38	3758	52
		Salimbad	2407	1533	3140	2.05	3527	38
District sub-total			43479	24704	50617	2.05	42122	607
MANIKGANJ	DAULATPUR	Char Katari	1901	1992	2776	1.39	2530	33
		Bachamara	3939	1406	2684	1.91	3197	32
		Baghutia	4502	2926	6308	2.16	4125	76
	SHIVALAYA	Teota	5047	1981	3120	1.57	4358	37
		District sub-total			15388	8305	14887	1.79
Total Jamuna River			367361	163056	295702	1.81	306518	3548

Total length of River is about 213 km

Water surface and uncultivable area is estimated at 1720 km²

The order of magnitude of annual land losses may be estimated at about 1500 ha not including Char lands

Source : Bangladesh Bureau of Statistics

TABLE 4.3-1
RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER (2nd Interim Report)
REPORTED DISBENEFITS IN SELECTED AREAS (Annual average in Tk Million)

	Fulcharighat	Sariakandi	Kazipur	Sirajganj	Betil
Length of bank erosion (km)	8	16	8	8	8
Erosion rate (m/year)	60	100	60	100	60
Consequence of erosion					
Land losses	7.640	25.225	8.035	14.063	10.500
Property losses	3.813	6.181	2.480	30.422	5.070
Infrastructure losses	1.014	1.682	0.816	2.708	1.419
Subtotal	12.467	33.089	11.331	47.193	16.989
Consequence of embankment breaches					
Crop losses	16.039	5.759	23.332	5.774	6.319
Livestock losses	0.321	0.034	0.314	0.595	0.332
Property losses	1.864	0.731	3.469	8.331	7.626
Infrastructure losses	1.567	0.380	1.942	1.705	2.150
Subtotal	19.791	6.904	29.057	16.405	16.427
Consequence of embankment retirement					
Reduced crop production	0.413	160.208	0.351	0.296	0.306
Reduced fish production	0.000	1.128	0.000	0.000	0.000
Reconstruction and O & M costs	4.060	0.000	4.060	4.060	4.475
Subtotal	4.473	161.336	4.411	4.356	4.781
Total average annual disbenefits	36.731	201.328	44.799	67.954	38.197
		+		+	
Average per km	4.591	12.583	5.600	8.494	4.775

+ High EIRR

TABLE 4.3-2
MEGHNA RIVER BANK PROTECTION/SHORT TERM STUDY
AVERAGE EXPECTED ANNUAL BENEFITS (in Tk Million)

Bhairab Bazar	210.2	+ (Inland port and industrial township)
Munshiganj	54.6	+ (Cold storage and industrial center)
Chandpur	238.6	(Inland port and industrial-trading center)
Eklashpur	56.2	(Irrigation project, but agricultural benefits represent less than 10% of the river training project benefits)
Haimchar	33.0	(Irrigation project, but agricultural benefits represent less than 10% of the river training project benefits)
Meghna R&H Bridge	380.4	+ (Communication node)
Maniknagar	14.9	(Agricultural project, but agricultural benefits represent only 20% of the river training project benefits)
+ High EIRR		

Figure 2.1-1

CONSTRUCTION DEFLATOR (1985 = 100)

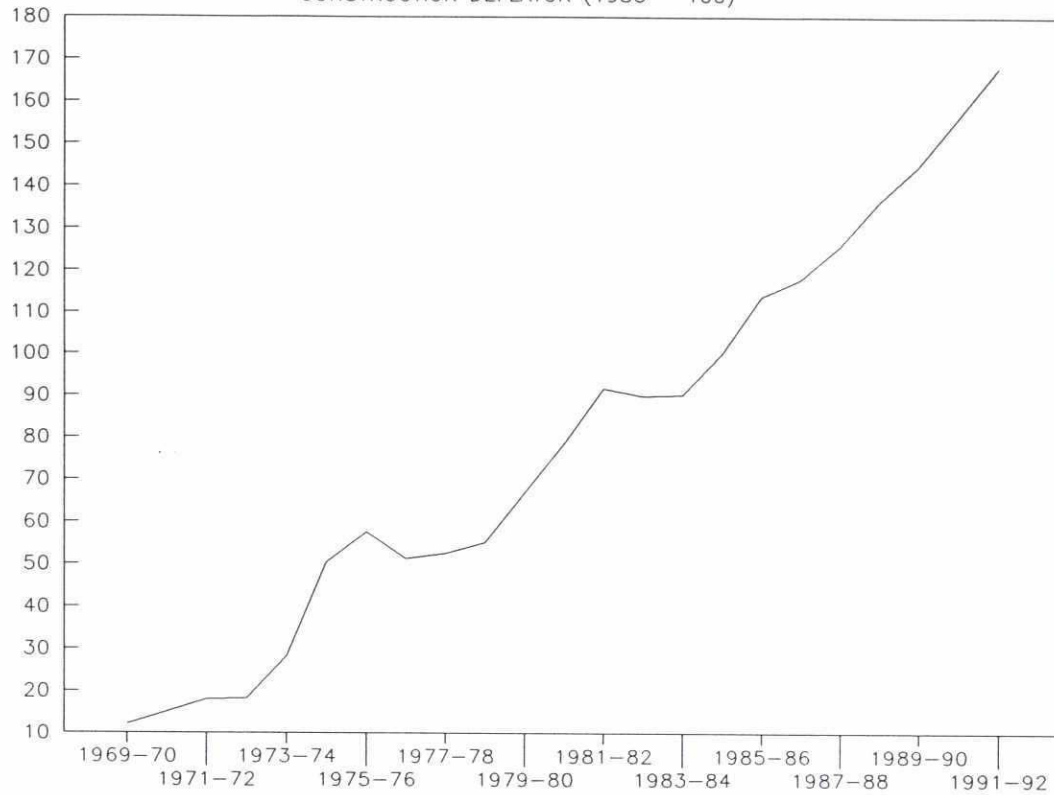


Figure 2.2-1

SIRAJGANJ PROTECTION BUDGET (M. Tk.)

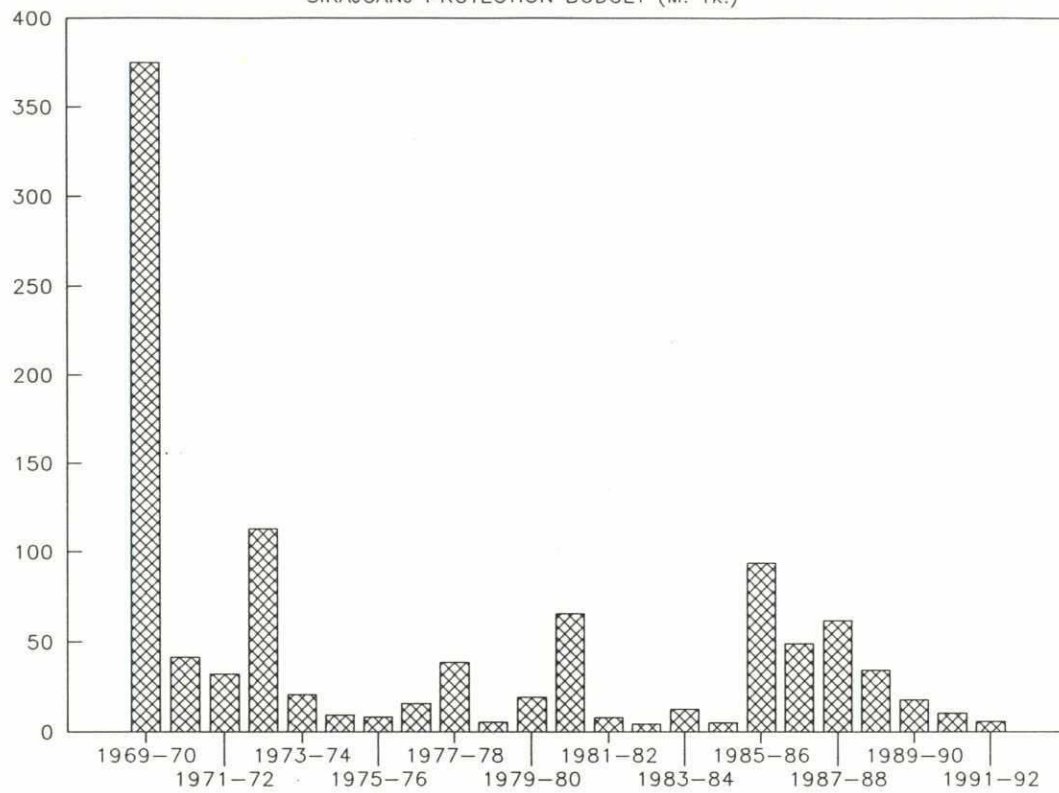


Figure 2.2-3

SAILABARI GROUYNE BUDGET (M.Tk.)

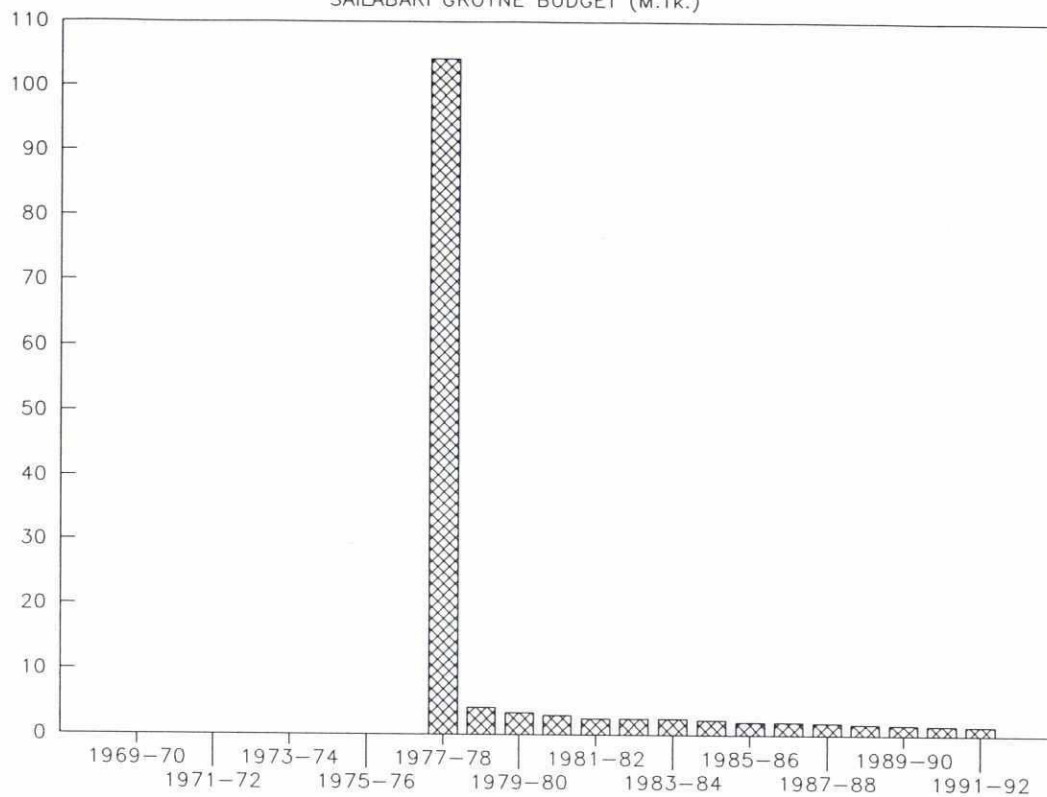


Figure 2.6-1

CHANDPUR PROTECTION BUDGET (M.Tk.)

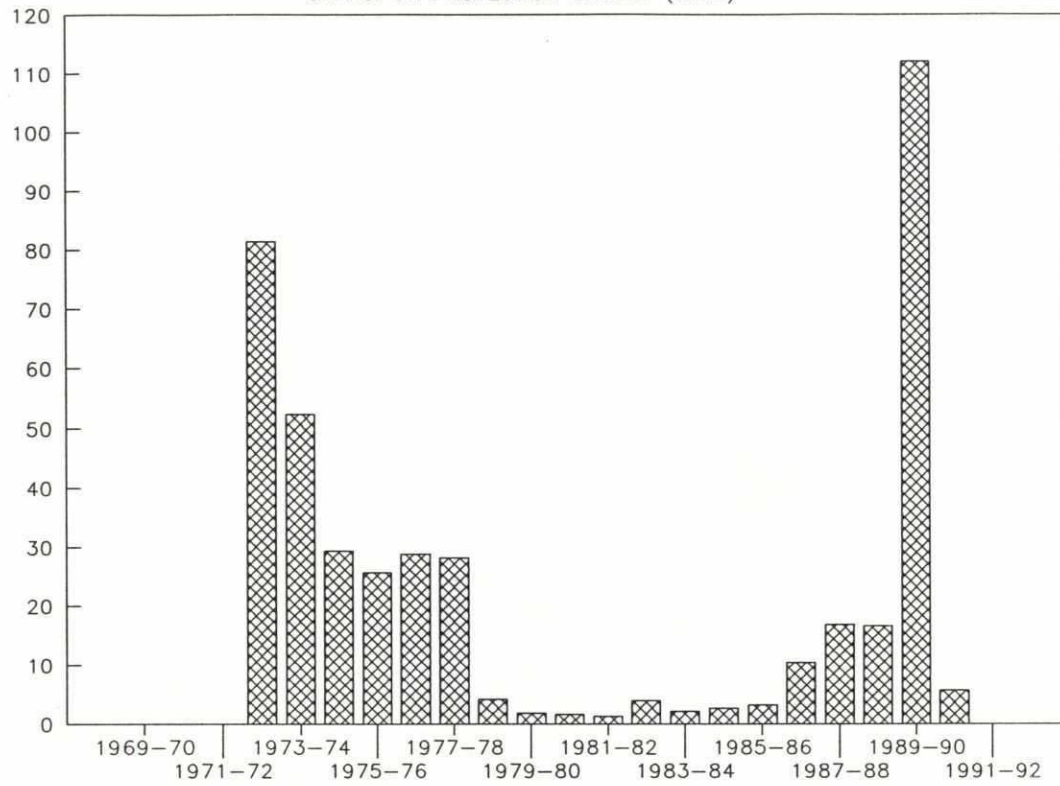
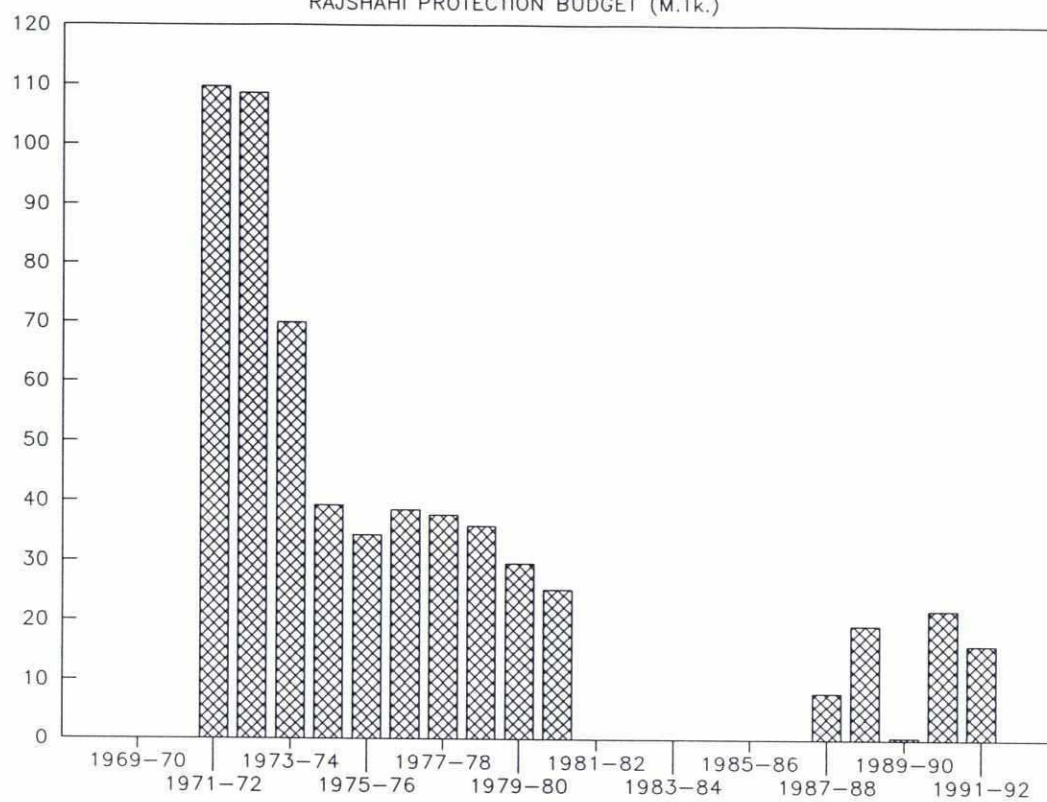


Figure 2.7-1

RAJSHAHI PROTECTION BUDGET (M.Tk.)



ANNEX 10

1-DIMENSIONAL MODELLING

ANNEX 10



1-DIMENSIONAL MODEL

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1 INTRODUCTION

For the study of the effectivity of recurrent measures for active flood plain management 1-dimensional hydrodynamic computations and 1-dimensional morphologic computations have been used for comparison and for calibration purposes. These mathematical models have been made with the MIKE 11 software package.

Two modules of the MIKE 11 package have been used in this study so far: the hydrodynamic (HD) module which is capable of simulating unsteady flows in a network of open channels and the non-cohesive sediment transport (NST) module of which the morphological mode has been used.

Some local models of a few branches, which represent the channels in the braided Jamuna river, have been developed and in these models the hydrodynamic and morphological effects of two river training measures, dredging and surface vanes at a bifurcation have been simulated.

The approach to simulate a river training measure was first to establish a morphologic equilibrium, this means that during a period of about 2 years with constant boundary conditions and a constant discharge no substantial erosion or sedimentation is calculated in the cross sections of the model.

In a restart computation the effect of the river training measure is superimposed on this equilibrium situation.

In the development of these local models special attention has been paid to the accuracy of these models. In this annex some aspects of the development of these local models are described in more detail. These aspects are the schematization of the cross sections, the influence of the space centering coefficient, the influence of a nodal point on the morphologic computations and the schematization of dredging activities in the 1-dimensional model. The results of the computations with respect to the effectivity of the simulated river training measures are not discussed.

2 GENERAL DESCRIPTION OF MIKE 11 MODEL

The core of the Mike 11 system consists of the hydrodynamic (HD) module which is capable of simulating unsteady flows in a network of open channels. The results of an HD simulation consist of time series of water levels and discharges. Associated with the HD module is the rainfall-runoff model NAM which may be used to generate inflows to the HD module. The non-cohesive sediment transport (NST) module can be run in two modes: explicit and morphological. In the explicit mode the sediment transport computations are based on the results of the hydrodynamic computation using characteristic transport parameters. There is no feedback from the sediment transport

calculations to the HD module. Results are in the form of volume transport rates and accumulated volumes of deposition or erosion.

In the morphological mode sediment transport is calculated in parallel with the HD module. The feedback between the sediment transport calculations and the HD computation is achieved through solution of the sediment continuity equation and through the updating of the bed resistance and the following sediment transport. Results are in the form of sediment transport rates, bed level changes, resistance number and dune dimensions (depending on the transport relationship adopted). The NST module is equipped with four different formulas for the calculation of sediment transport rate and alluvial roughness. These are the Engelund and Hansen model and the Ackers and White model for the calculation of total load, and the Engelund and Fredsoe and van Rijn formulas for the calculation of bed load and suspended load separately.

The model provides a choice among three different flow descriptions: dynamic wave approach, diffusive wave approach and the kinematic wave approach and in version 3 especially for morphologic computations a quasi-steady state approach. For a morphologic computation in a river reach the quasi-steady state model is often more appropriate than the full hydrodynamic model, which often requires relatively small time steps. During the first months of this project version 3 was not available and all computations have been made with the full hydrodynamic mode of version 2 during that period.

3 ASPECTS OF THE DEVELOPMENT OF LOCAL MODELS

In this study some local models have been developed to simulate the effects of some river training measures. These models are used also to be acquainted with the Mike 11 model and to explore the capabilities of this mathematical model.

In general the results with the hydrodynamic module are satisfactory. However the morphological module is obviously less developed than the hydrodynamic module. In the version 2.10 of Mike 11 the sediment distribution at a bifurcation point is set equal to the ratio of the outflowing discharges in the branches, which are connected with this bifurcation point. In a recently released test version 3.0 of Mike 11 different options are available to select this sediment distribution. For a correct simulation of river training measures it is important that this sediment distribution can be selected in a flexible way.

Mike 11 model includes descriptions for a wide range of structures which act as control points. In a morphological computation with Mike 11 no structures should be placed in the model.

3.1 SCHEMATIZATION OF A CROSS SECTION

The natural river cross-sections of the Jamuna River differ widely in shape. Therefore in a morphological model one or more representative cross-sections are placed according to the elevation of the natural channel slope of the river to represent the river channel. It reduces the rapid changes in flow velocity between adjacent cross-sections in case natural cross-sections are applied and in this way it prevents mathematical instability during the computation. Equivalent cross-sections are schematized in the model in two ways. First an equivalent smoothed cross-section, based on a type of regime equations for the averaged width and depth of the cross-section of the Jamuna river is schematized in the model, see figure 3.1-1. In a second schematization an equivalent rectangular cross-section based on the same regime equations is used to represent the river channel in the model, see figure 3.1-1. If the total width of the flood plain is reduced to 20% by introducing relative resistance of 5, as is recommended in other studies, a rather good similarity in the reproduction of the water levels of the first schematization is obtained, see figure 3.1-2.

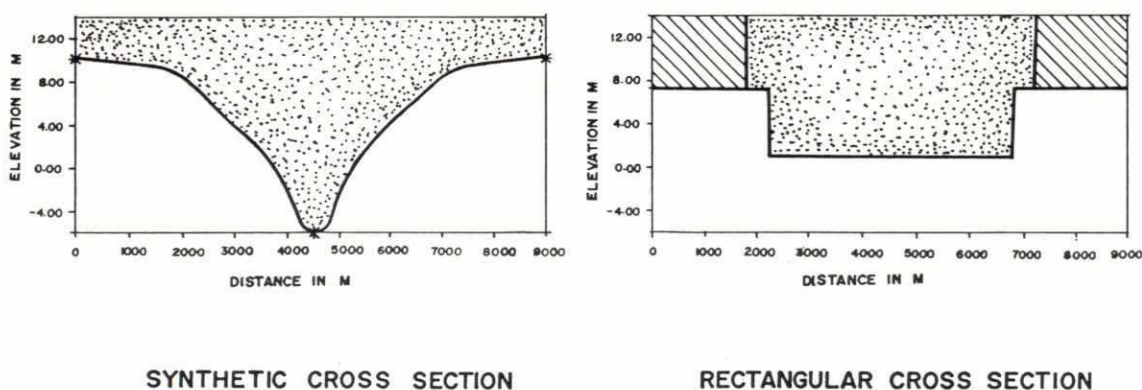


Fig. 3.1-1

It is mentioned that for a computation with the full width of the flood plain a more gradual transition from the main channel to an inclined flood plain is recommended to avoid instability during the first stage of overtopping of the char. The conclusion is that in this study a schematized equivalent rectangular cross-section for the main channel and the flood plain can be applied without introducing serious deviations in the water level, if the discharge is given. This simple schematization will be very useful in the continuation of the study.

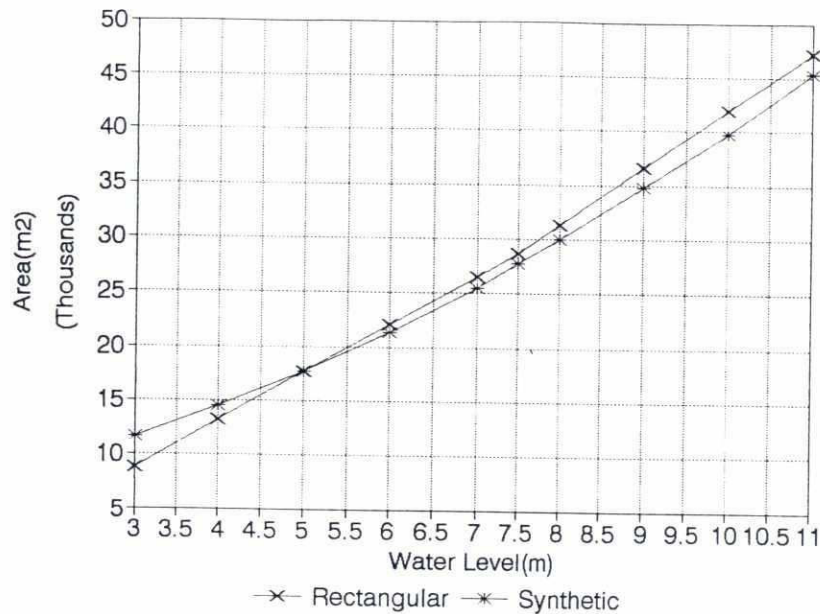


Fig. 3.1-2

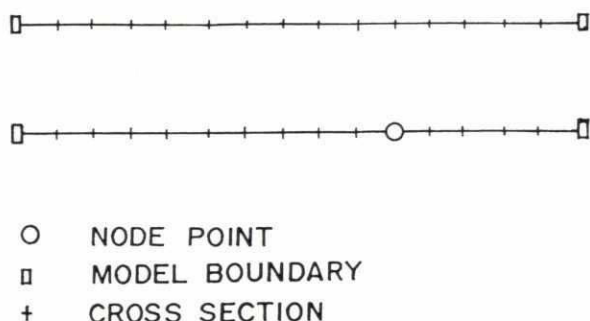
3.2 INFLUENCE OF SPACE CENTERING COEFFICIENT

During the initial development of a simple single channel model in which the cross-sections are spaced at 1 km interval, it turned out that a model run with a time step higher than 30 minutes and a Courant number of about 24 shows instability in model results. To overcome this instability the space centering coefficient, Δ , has been increased. A forward centering of the $\delta h / \delta x$ -term in the numerical scheme has a dissipative influence. By increasing Δ from 0.50 to 0.90, the time step could be increased to 4 hours without introducing any instability. To assess the influence of space centering coefficient, a rather short single channel model with a rectangular cross-section of 4500 m width at 1000 m interval is developed. At the upstream model boundary a water level hydrograph with a variation of water level of 1 m per day was defined and a rating curve ($h = 0.56 Q^{0.23}$) is used as downstream boundary. Next some hydrodynamic model runs are made with a time step of 60 minutes, Courant number of 45 and a Δ coefficient of 0.55, 0.70, 0.80, 0.90, 0.95. Model results are compared with that of having a time step of 10 minutes and a Δ coefficient of 0.50. In these computations the maximum difference between the water levels is only 1 to 4 mm. The conclusion is that an increase of this coefficient Δ from the default value 0.5 to 0.95 is acceptable and does not reduce the accuracy of the local models for the Jamuna River.

3.3 INFLUENCE OF A NODAL POINT.

In order to assess the influence of a nodal point in a model on the sediment transport, bed level and Manning Number, a single channel model has been made without a nodal point

and with a nodal point (see figure 3.3-1). The computations are made with the same boundary conditions and geometric, hydraulic and morphological parameters. The length of the long channel is 15 km and in the other model two channels with a length of 10 and 5 km are connected by a nodal point. In both models the channels have a rectangular cross-section with a width of 4600 m. The upstream boundary condition is a constant discharge of 35000 m³/s and at the downstream boundary the water level is kept constant at 6.54 m. The sand of the channel bed has a global grain diameter of 0.20 mm and the channel roughness is characterised by a global Chézy number of 65 m^{1/2}/s. The sediment transport is calculated with the Engelund-Hansen formulas for a morphological computation. In some runs a period of 1 year is simulated with time steps of 60 minutes. The distance between the equidistant cross-sections is 1000 m, so the Courant number is 32.



SCHEMATIZATION SINGLE CHANNEL MODEL

Fig. 3.3-1

The procedure to determine the bed slope of the channels starts with a horizontal bed in the first computation, then in the next computation the bed slope is made parallel to the computed water surface slope. This is repeated until the water level slope is parallel to the bed slope. Initially a model run is made without upstream sediment boundary for 1 month to prevent any disturbance by the hydrodynamic computation, which might be not in a full equilibrium directly from the start of the computation. After one month a constant sediment discharge of 0.90 m³/s is used at upstream boundary. This sediment transport is estimated as the equilibrium sediment transport.

Some results of the computations are presented in Table 3.3-1. From this table it can be seen that after a simulation time of 1 year the maximum difference in the bed level close to the nodal point between single channel model without a node and the model with a node is 7 mm. But by reducing the distances between the cross sections adjacent to the nodal point from 1000 m to 300 m this difference can be reduced to 2 mm, however the computing time has increased. These computations have been made with version 2.10 of Mike 11 model. The new version 3.0 gives different results, but still differences in bed level in computations between with and without a node are observed.

Table 3.3-1: Difference in bed level (m) between single channel model without a nodal point and with a nodal point.

Chainage Branch1/ Branch2	Model Description				
	FAP002	FAP003 ¹	Diff.in mm.	FAP003 ²	Diff. in mm.
0	0.353	0.353	0.00	0.353	0.00
1	0.312	0.311	1.00	0.312	0.00
2	0.275	0.273	2.00	0.273	2.00
3	0.243	0.239	4.00	0.240	3.00
4	0.214	0.212	2.00	0.212	2.00
5	0.188	0.187	1.00	0.188	0.00
6	0.166	0.162	4.00	0.163	3.00
7	0.134	0.134	0.00	0.135	-1.00
8	0.107	0.106	1.00	0.107	0.00
9	0.079	0.077	2.00	0.078	1.00
10/0	0.051	0.058	-7.00	0.053	-2.00
11/1	0.022	0.029	-7.00	0.025	-3.00
12/2	-0.006	0.000	-6.00	-0.004	-2.00
13/3	-0.034	-0.030	-4.00	-0.033	-1.00
14/4	-0.063	-0.061	-2.00	-0.062	-1.00
15/5	-0.091	-0.092	1.00	-0.091	0.00

Notes:

FAP002: Single channel model without a node

FAP003¹: Single channel model with a node (Δx near node 1000 m)

FAP003²: Single channel model with a node (Δx near node 300 m)

From these computations it is shown that a nodal point reduces the accuracy of the computed bed level, sediment transport and Manning number. The accuracy of the computed bed levels after many time steps and a simulation of several years in a morphological model with several nodes is probably less than expected. But a considerable improvement of the model accuracy can be obtained by inserting more cross-sections near a node. This can be considered as a general recommendation.

3.4 COMPARISON OF RESISTANCE RADIUS AND HYDRAULIC RADIUS

In the Mike 11 model a choice can be made between the hydraulic radius and the so called resistance radius. These parameters have some influence on the variation of the hydraulic roughness as a function of a variable discharge. A simple comparison between these parameters has been made for the characteristic conditions in the Jamuna river.

In the Mike 11 model the resistance radius R_* is calculated as:

$$\sqrt{R_*} = \frac{1}{A} \int_0^B y^{\frac{3}{2}} db$$

where A = cross section area (m^2), y = the local water depth (m) and B = the water width (m) at the same elevation. The advantage of this formulation is that the resistance radius allows a smooth transition in the conveyance of the channel as the char region becomes flooded, while the hydraulic radius causes a sharp, incorrect decrease in the conveyance. A disadvantage of the resistance radius is an over estimation of the conveyance in the deep and narrow cross sections as its formulation does not fully take into account the friction from the cross-section sides as does the hydraulic radius formulation.

The hydraulic radius formulation is based on a parallel channel analysis where the total conveyance K of the section at a given elevation is equal to the sum of the conveyances of the parallel channels. The hydraulic resistance is calculated as the well known form of $R_h = A/P$, in which P = wetted perimeter (m).

To assess the effect of both parameters, hydraulic resistance formulation and resistance radius, on model results, the single channel model with a rectangular cross-section of 4500 m wide main channel and a 4500 m wide flood plain is used. A relative resistance of 5 for the flood plain is applied. The length of the channel is 15 km and the distance between the grid points is 1000 m. A constant discharge of 76000 m^3/s and a constant water level of 10.88 m are the upstream and downstream boundary respectively. Hydrodynamic model runs are made with a time step of 120 minutes for both hydraulic and resistance radius formulations. Model results show that computed water level in case of hydraulic radius formulation is 0.08 m higher than that of in case of resistance radius formulation at upstream boundary. Model results also show that if a relative resistance of 1 is used for flood plain then computed water level in case of hydraulic radius formulation is 0.21 m higher than that of in case of resistance radius formulation at upstream boundary.

These differences are relatively small and since in most previous studies concerning the Jamuna river the hydraulic radius is used, the Consultant has chosen the hydraulic radius in all other computations.

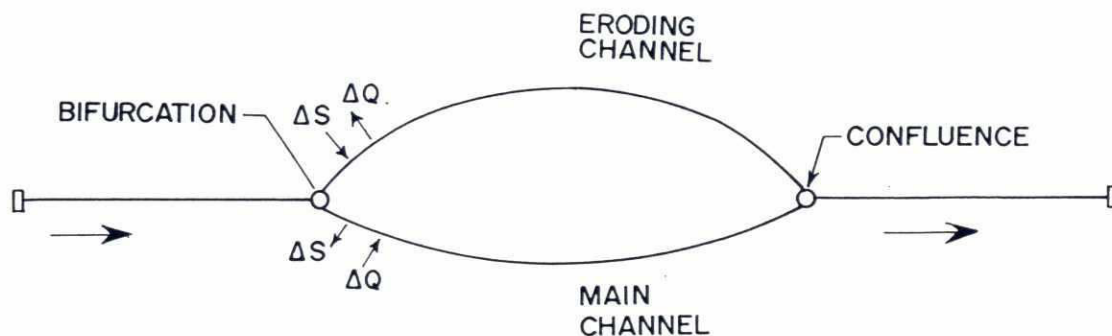


Fig. 3.5-1

3.5 SIMULATION OF DREDGING IN MIKE 11 MODEL

One of the identified river training measures is dredging in one branch and to dump the spoil in a parallel branch to stimulate sedimentation in that parallel branch. This measure can be simulated in a looped channel model, as sketched in figure 3.5-1. Initially this measure was schematized by a lateral sediment outflow and sediment inflow at a single grid point in two parallel branches. To make a simulation of one year first a simulation of 6 months is made, and after this run the bottom is reduced with the total dredged volume during in that period and a restart computation for the next 6 months has been made. Next this procedure is repeated more precisely with a simulation of one year in four computations of each 3 months run time, see the bed profiles in figure 3.5-2. The model results are compared with that in which dredging is schematized by lateral sediment outflow at five grid points in the dredged channel. From this comparison it seems that the porosity of the sand should be considered in calculating the dredged volume. When the lateral sediment outflow boundary is multiplied by (1-porosity) and distributed at five grid points of the dredged channel, it is concluded that the model results match well with the results of that model in which dredging is schematized by stepwise changes in the bottom level of the dredged channel. The conclusion is that dredging can be schematized accurately in the Mike 11 model by lateral sediment outflow at the grid points in the dredged channel.

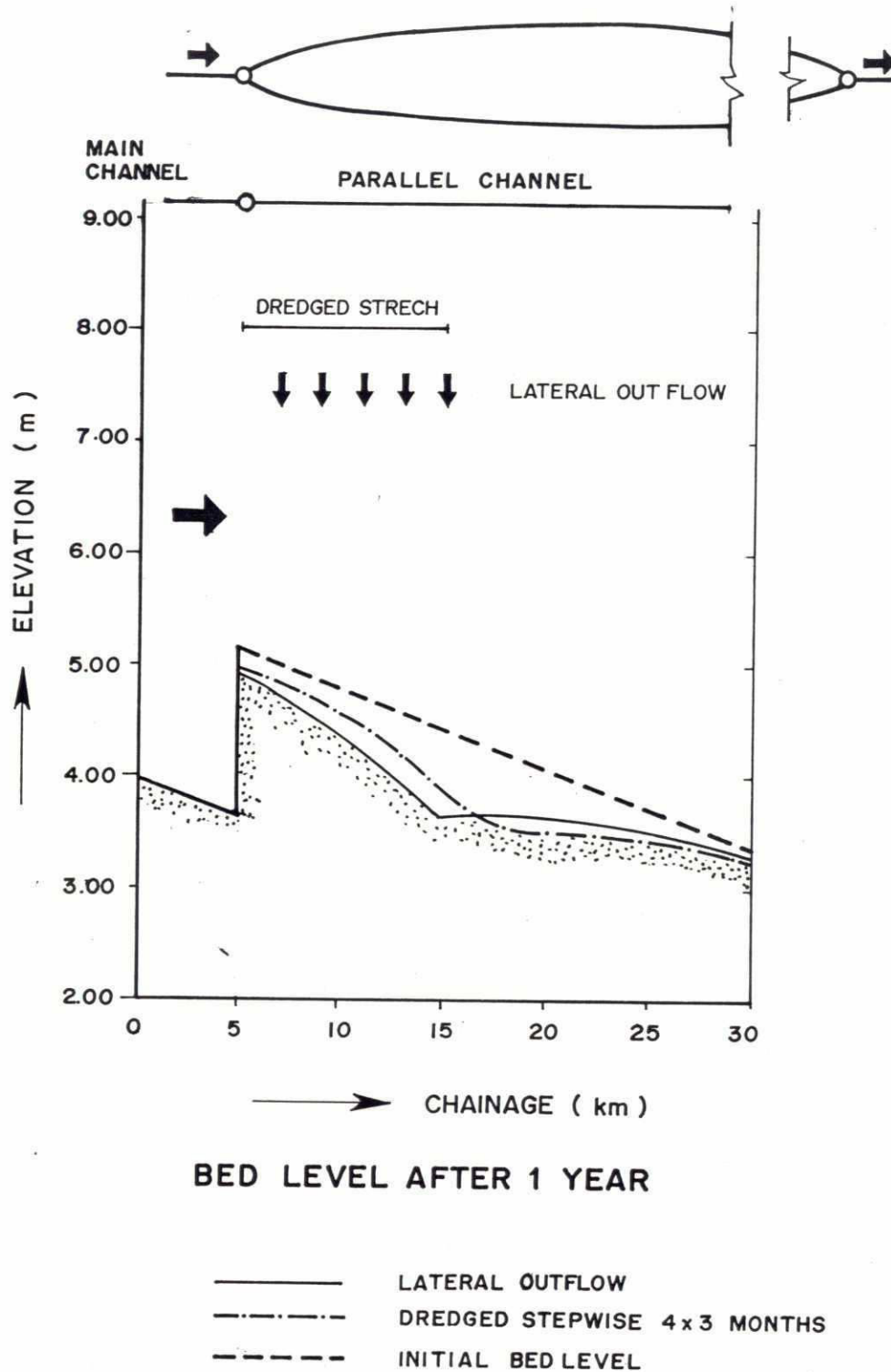


Fig. 3.5-2

