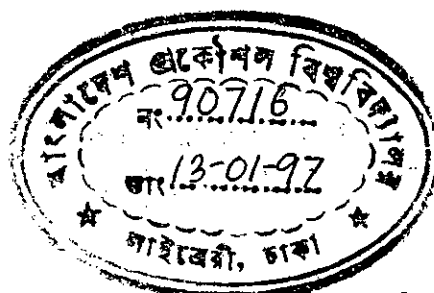


# A STUDY ON THE MORPHOLOGICAL CHARACTERISTICS OF THE GORAI-MADHUMATI RIVER

Submitted by

MD. MONIRUL ISLAM



In Partial fulfillment of the requirements for the degree of  
Master of Science in Engineering (Water Resources)

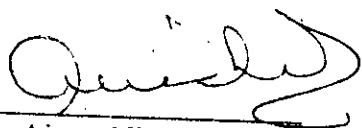
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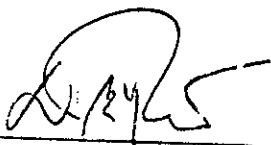
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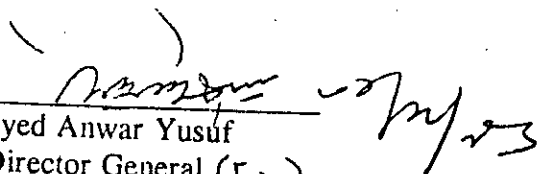
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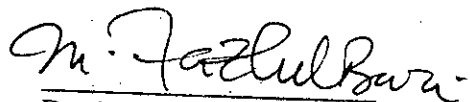
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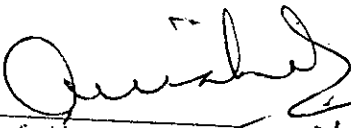
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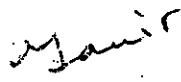
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This is to certify that the research reported in this thesis was performed by me and neither this thesis nor any part thereof has been submitted elsewhere for the award of any degree or diploma.

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## ABSTRACT

Processes governing morphological changes in the Gorai-Madhumati, an off take of the Ganges which is an alluvial meandering river have been studied. A time-series data base on water level and discharge was utilized. In addition sediment transport, rating curves, channel cross-section and channel alignment data were also analyzed. The morphological parameters of the 258.5 km reach of the river were studied on the basis of maps of 1953, 1973 and 1990. Partial maps for lower reach of 1771, 1863, 1944, 1963 and 1980 was also used. Water level, discharge and sediment data of Gorai Railway bridge station and Kamarkhali covering 1946 to 1990 (with a gap in 1955-57 and 1971) was used in the study.

The meander parameters such as meander wave length, meander belt and sinuosity for the river was established by identifying and detail measurements of the partially and fully developed meanders on the plan maps. The relationships between meander parameters and channel width as well as between the representative discharge was developed.

As a first step representative discharge i.e., dominant discharge for the river was estimated. The dominant discharge for this river was found to be 4500 cumecs. The average bankfull discharge, computed utilizing the rating curves for different years was found to be 4813 cumecs. The relations between meander parameters and discharge were found to be  $M_L = 43.05 Q_d^{0.5}$  and  $M_B = 33.01 Q_d^{0.50}$ , where  $M_L$ ,  $M_B$  and  $Q_d$  are meander wave length, meander belt and dominant discharge respectively. From the average value of meander parameters for different years, it was observed that the ratio of meander wave length to meander belt gradually decreased with time. Strong correlations was found with average channel width with meander parameters in some cases. The bend characteristics i.e., the bank erosion rates along the curved sections of channels was studied with the help of landsat

and spot images. Twenty seven (27) clearly distinct bends were analyzed in detail following the techniques suggested by Nanson and Hickin (1984). It was found that bank erosion rates depends on the relative bend curvature value,  $r/w$ , where  $r$  and  $w$  are radius of curvature and average channel width respectively. It was found that maximum erosion rate occurred with the  $r/w$  value approximately 2.5. Bend migration generally starts with  $r/w$  value reaching app. 5.0 and about stop when  $r/w$  value decreases to 1.0. There is also a trend between erosion rate and relative bend curvature. Maximum bend migration rates were found to be 142.6 m/year.

Hydraulic geometry was studied for Gorai Railway bridge and Kamarkhali for 1976 to 1990 hydrological years. Correlations were developed between width, depth, velocity and cross-sectional area with observed discharge and good correlations between these parameters was observed. Specific gauges were analyzed for different discharges for the station Gorai Rly bridge and Kamarkhali. From this analysis it was observed that the water levels gradually rose especially after 1975 for same discharge. To evaluate the change in river bed morphology, BWDB cross-sectional data for different years was used. From this study it was found that the channel was gradually narrowing in the middle part and bed levels at the upper reach gradually rose especially after 1975. But no definite rise/fall tendency was observed in the other parts of the river. Average river bed slope, valley slope, sinuosity and channel longitudinal bed profiles were studied using the cross-section survey data for different years. The average bed and valley slopes were found to be  $4.665 \times 10^{-5}$  and  $7.47 \times 10^{-5}$  respectively. Average sinuosity values observed to be approximately 1.845 in 1953, 1.684 in 1973 and 1.837 in 1990. From sinuosity values it was observed that the river was in highly meandering stage.

Formation of cut-offs, meander development and length changes of the river with time were also studied. Maps were superimposed and

compared to evaluate the direction, patterns and different phases of the meander development and the possibility of cut-off formations. While five (5) numbers of natural cut-offs occurred between 1953 and 1990, no definite conclusion could be drawn on the physical process of actual cut-offs occurring due to absence of continuous data. From the study it was observed that the change in total length of the river in the long term is negligible and the river was in a dynamic equilibrium state.

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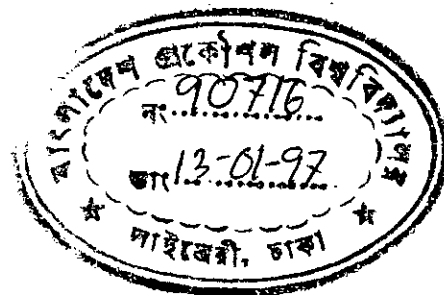
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## LIST OF ABBREVIATIONS

|         |                                                |
|---------|------------------------------------------------|
| BWDB    | Bangladesh Water Development Board             |
| DD      | Dominant Discharge                             |
| PWD     | Public Works Department                        |
| RRI     | River Research Institute                       |
| SOB     | Survey of Bangladesh                           |
| SPARRSO | Space Research and Remote Sensing Organization |
| WARPO   | Water Resources Planning Organization          |

# CHAPTER-I

## INTRODUCTION



### 1.1 General

Rivers have been a focus of human activities throughout the history. While mankind benefits from rivers, and they also suffer from floods and land erosion caused by it. Engineers have to plan, design, operate and maintain projects involving rivers, for regulation, flood mitigation, water supply, navigation, and irrigation. Hence rivers have been subjects of study by engineers and scientists who have been fascinated by the self formed geometric shapes and other responses to changes in nature and human interventions. Understanding of river behaviour is also necessary for environmental enhancement.

Alluvial rivers are migratory. An important characteristics of many rivers in the alluvial flood plains is its flow in sinuous path, called meandering. Meandering stream is one whose channel alignment consists principally of pronounced bends, the shape of which have not been determined predominantly by the varying nature of the terrain through which channel passes. They flow freely from one bank to another. The meandering causes the river to leave their original courses, force them to flow along new courses, and thus devastating vast areas of land and affecting important and valuable nearby structures, such as bridges, railway lines, roads etc. The proper understanding of meander development and channel pattern changes of alluvial rivers is very important for all engineering projects which has linkage with the river. Meandering is one of the means through which rivers tend towards the so-called dynamic or quasi-equilibrium state. River training and construction of training works are also designed to achieve a proper equilibrium. The more the planned channel pattern, channel geometry, slope conditions, etc. correspond to the natural conditions of the river in question,

the better will the river accept the new semi-artificial state. The quasiequilibrium state of a meandering river, or a certain longer reach of it also, includes the dynamic equilibrium of the river length.

The slow successive long term increase in length caused by meander development is compensated by short term, self-shortening process effected by local cut-offs of over developed meanders. One method of training meandering rivers is by the construction of artificial cut-offs. From the view point of equilibrium length, the natural and artificial cut-offs provide similar results. Alluvial river channels are controlled by the amount of discharge. The morphology of an alluvial river, therefore, is a process of fluctuating discharge and sediment balance within a reach. Changes in channel morphology as a result of erosional and depositional process have been well documented in map and diagram form, but only a few studies have measured the actual rate of channel migration and channel morphological change. Meander bend may migrate in lateral and down valley directions and may grow a subside. The process of flow and sediment transport that affect meander bend migration are important. An adequate understanding of such processes, needless to say, is essential for river control and regulation and for mathematical modelling of river channel changes.

Our understanding of alluvial river problems stems largely from two sources : extensive laboratory experiments and studies on the basis of field data. Many engineers and scientists of the world worked in the laboratory as well as with the field data studied the behaviour and characteristics of meandering river. Laboratory data obviously gives the better correlations. However, with necessary field data studies could be made for prototype situation.

An understanding of the direction and magnitude of change in channel characteristics caused by the action of man and nature is

essential for the purpose of planning any activity that will modify the normal behaviour of the river. This understanding can be obtained by : (1) studying the river in a natural condition ; (2) having knowledge of the sediment and water discharge ; and (3) being able to predict the effects and magnitude of man's future activities ; by applying knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers. In the natural rivers, the geological and hydrological features for the different rivers are different and as the behaviour of the river largely depends on the many factors. A given reach of a river, may exhibit both meandering and braiding; alteration of the controlling parameters in a reach can change the character of a given stream from meandering to braided or vice versa. Undisturbed rivers have achieved a state of approximate equilibrium condition (quasi-equilibrium) throughout the long reaches, or reaches without major intervention may also achieved a state of apparent stable condition, but, regardless of the degree of channel stability, man's local activities may produce major changes in river characteristics both locally and throughout an entire reach, e.g., an increase in channel slope, may be either by human intervention or by natural processes, indicates a tendency evolve from a relatively tranquil, case to control meandering pattern to braided pattern that varies rapidly, with time, has high velocities, is subdivided by sand bars, and carries relatively large quantities of sediment.

## 1.2 Specific objectives of the study

The main objectives of the present study are :

1. to study the bank shifting characteristics of the Gorai-Madhumati river reach under study and determine the bend migration pattern.
2. to establish the interrelationships between various geometric parameters ( $M_L$ ,  $M_B$ ,  $S_r$  etc.)
3. to estimate the dominant discharge of Gorai.



## CHAPTER - II .

### STUDIES ON ALLUVIAL RIVERS

#### 2.1 Introduction

River Engineering involves the control and utilization of rivers for the benefit of mankind and the knowledge of river meander is of practical importance in the design and location of river training works and location of such structures as bridges and water intakes where a choice of location may exist (Inglis, 1947). The most important parameter needed by the designer are meander length and radius of curvature. Meander length is needed so that the works can be located in relation to existing or future stable patterns of meanders and the minimum radius of curvature is needed to indicate whether the river channel may be induced to follow a given course to predict the course likely to result from imposed restrictions (Thomas, 1985).

#### 2.2 Sediment transport

The most important of a river is that it prevails a movement of water, and sediment from the land to the oceans consequently the structure and form of the land and its margins are altered through erosion and deposition. Then character of the river changes. Sedimentation includes the processes of erosion, entrainment, transportation, deposition, and the compaction of sediment in the stream channels. The products of erosion may be deposited immediate down stream of the sources, or may be transported considerable distance to be deposited. Quantifications and investigation of the properties of sediment transported through alluvial channels are an important task, because they play a vital role on the stability and behaviour of an alluvial stream. The representative discharge or dominant discharge of a natural stream depends completely upon the

characteristics of the sediment transported through the channel. The theories and empirical relationships for the prediction of sediment movement so far developed are quite large.

There are many authors who have significant contributions in this field namely : Dubois (1879) Mayer-Peter and Muller (1948), Schoklitsch (1935), Einstein-Brown (1950), Laursen (1958), Blench (1966), Engelund-Hansen (1967), Inglis-lacey (1968), Toffaleti, (1969) Ackers-White (1973), Yang (1976), Mantz (1983), and Hossain (1985) etc. The rate of sediment transport in rivers depends upon great many variables out of which the water discharge, average flow depth, flow velocity, energy slope, shear stress, particle size or representative particle size,  $d_{50}$  etc. are important. The sediment transport formulae available have been derived primarily based on laboratory flume and natural stream data. In estimating the dominant discharge sediment transport data is of utmost importance. Two important sediment discharge formula those are best suited (walliuzzaman, 1986) and easy to handle in computing the sediment discharge of Gorai-Madhumati river are discussed below.

#### Engelund and Hansen (1967) formula.

Engelund-Hansen developed a sediment discharge formula based on the shear stress approach. In developing the equation they relied heavily on the data from experiment in a specific series of tests in a large flume. The sediment used in this flume had a median fall diameter of 0.19 mm, 0.27mm, 0.45 mm, and 0.93 mm and the flume dimensions were 150 ft long and 8 ft wide as reported by Guy et al, (1966 after Hossain, 1989).

The equation can be written as :

$$\gamma_s = 0.05 \cdot \gamma_s \cdot v^2 (d_{50} / (g(\gamma_s - \gamma)))^{0.5} \cdot (\tau_0 / ((\gamma_s - \gamma) \cdot d_{50}))^{3/2} \quad (2.1)$$

Where,  $g_s$  = The sediment transport per unit time per unit width,  
 $v$  = the average flow velocity,  
 $d_{50}$  = median particle diameter,  
 $\tau_0$  = bed shear stress,  
 $\gamma_s$  = specific weight of the sediment particles,  
 $\gamma$  = specific wt. of the water,  
 $g$  = acceleration due to gravity.

All the variables were in the F.P.S. system.

Hossain (1987) formula (after Dey, 1995).

In 1987 Hossain suggested a sediment transport formula based on the concept of dimensional analysis and similitude argument of semi-empirical nature. He proposed that the sediment concentration in a stream of steady water and sediment flow is a power function of : (i) the product of Froude number and slope, (ii) the settling velocity ratio and (iii) the discharge ratio. The equation can be written as :

$$C_t = A(X^a Y^b Z^c) \quad (2.2)$$

Where,  $C_t$  = Sediment concentration in ppm,  
 $A = 0.845 \times 10^5$  for  $Q < 1.0 \text{ m}^3/\text{s}$   
 $= 6.946 \times 10^5$  for  $Q > 1.0 \text{ m}^3/\text{s}$  and  $B/H < 100$   
 $= 6.946 \times 10^6$  for  $Q > 1.0 \text{ m}^3/\text{s}$  and  $100 < B/H < 500$

$$X = v_s / (gh)^{1/2}$$

$$Y = w_r / w$$

$$Z = Q / Q_c$$

and

$$a = 0.745$$

$$b = 0.633$$

$$c = 0.500$$

$w_r$  = Settling velocity for a representative sediment size for which

$d_{50}$  = 0.15 mm at ambient temperature.

$w$  = Settling velocity of the sediment particles.

$Q_c$  = Assessed discharge from equation.

$Q_c = [ (2.15 + K B/H) H (gs)^{1/5} ]^{5/2}$

$Q$  = discharge

$K = 0.205$

But this formula was modified when applied against large river data of Bangladesh, value of  $K$  were found as follows.

$K = 0.055$  when  $Q < 15000 \text{ m}^3/\text{s}$

$K = 0.170$  when  $Q > 15000 \text{ m}^3/\text{s}$

$B$  = average width of channel

$H$  = average depth of channel

$s$  = average water surface slope

$g$  = acceleration due to gravity

## 2.3 River course shifting

River constantly change their position and shape as a result of erosion around the outside of the bend and through deposition on the point bar. More dramatic changes may occur from the formation of cutoffs. Constructing new bar and destroying the old bars the whole channel of wider river system may shift across the flood plain.

### 2.3.1 Causes of river course shifting

The characteristics and direction of flow and the properties of bed and bank material are the main cause of river course shifting. Characteristics of flow implies oncoming sediment load versus sediment carrying capacity of the flow. The bed and bank material properties comprises the erosive resistance of the bank against the bank caving capacity of the flow and the resistance against the bed shear of the flow. Probably the major controlling factor, however, is the amount of sediment and the inability of the river to transport it. The aggradation constantly causes the

main current to seek better gradients, new alignments and paths of least resistance. In this manner the river constantly migrating from one position to another position.

Although the aggradation of the river bed is one of the major factors responsible for the wandering of a river course, it also creates conditions under which other factors manifest their influence.

A number of factors, including the high susceptibility to erosion of bed and bank material, the large variability of the flow, and the abrupt rise and fall of the river stage, have a strong bearing in determining the migration patterns (Coleman, 1969).

## 2.4 Meander geometry

In describing the features of a meandering stream the meander wave length ( $M_L$ ), the meander belt ( $M_B$ ), channel width ( $W$ ), radius of curvature, ( $r_c$ ) and the sinuosity ratio ( $S_r$ ) are important. But traditional meander scale and shape indices are the meander wave length and radius of curvature (Ferguson, 1975). Engineers and research workers have attempted to relate these strictly average characteristics of a natural stream to the flow characteristics and with each other. Axial wave length and radius of curvature is difficult to measure in irregular bends, so that a careful operational definition is a must. Nevertheless the important relationships are based on these indices (Richards, 1982).

Based on experimental analysis, in Poona laboratory (1945) it was found that

$$M_L = 36.5 Q^{0.5}$$

and  $M_B = C_b Q^{0.5}$

Where,  $C_b = 11.5$  to  $17.0$  and

$Q$  = dominant discharge or constant discharge in laboratory flume.

(2.3)

Inglis (1949) correlated the meander wave length and meander belt analyzing the huge amount of field data, with the maximum discharge as,

$$M_L = 29.60 Q^{0.5} = 6.06 W \quad (2.4)$$

Where,  $Q$  = maximum discharge in cfs,  $M_L$ ,  $M_B$  and  $W$  in feet.

Dury (1955) relates the meander wave length to bankfull discharge as

$$M_L = 54.3 Q_b^{0.5} \quad (2.5)$$

Although Ackers and Charlton (1970) showed the role of bankfull discharge on meander wave length, a stronger correlation exists with the channel width (Schumm, 1969).

Zeller (1967) and Leopold and Wolman (1960) found that

$$M_L = 7.32 W^{1.11} \quad (2.6)$$

$$\text{and } M_L = 12.13 W^{1.09} \quad (2.7)$$

Neglecting the exponents, the linear regression

$$M_L = 12.34 W \text{ is quite satisfactory.} \quad (2.8)$$

In the above equation, the constant (co-efficient) is very close to  $4\pi$  (12.57), which suggests a close link with the riffle-pool wavelength ( $2\pi W$ ), because in a regular meander bend there are two riffle-pool cycles to one bend with pools at the apices and riffles in the inflections (Richards, 1982). The strong correlation with width reflects the role of secondary circulation controlled by channel size, channel width is caused by an interaction of discharge and bank material properties. Discharge and sediment jointly control width, which then controls the wavelength of secondary flow (Yalin, 1971) and hence that of the riffle-pool sequence and meander bends. Hence the correlation with the discharge are poorer than that with the channel width.

Leopold and Wolman (1960) also found that the mean radius of meander bends ( $r_m$ ) is, in general, related to width of channel ( $W$ ), so that  $M_L$  can be expressed in terms of  $r_m$

$$M_L = 10.90 W^{1.01} = 4.70 r_m^{0.98}$$

(2.9)

and  $M_B = 2.7 W^{1.10}$

All of them are measured in feet. If the exponent are approximated as unity, then

$$r_c = 2.4 W$$

(2.10)

Which indicates the approximate maximum curvature for meander bends, found by Leopold and Wolman.

Dury (1965) found the relationships with mean annual flood ( $Q_{ma}$  cfs) as follows :

$$M_L = 30 Q_{ma}^{0.5}$$

(2.11)

$$M_B = 18 \text{ to } 20 W$$

and with bankfull discharge as

$$M_L = 54.3 Q_b^{0.5}$$

(2.12)

Where  $W$  = bankfull width in feet.

Carlston (1965) found the following relation between meander wave length and mean annual discharge,  $Q_{ma}$  for the rivers in flood plains,

$$M_L = 106 Q_{ma}^{0.46}$$

(2.13)

Using laboratory data, based on the theory of oscillation of the transverse velocity component, Anderson (1967) obtained the following equation,

$$M_L = 39 Q^{0.39}$$

(2.14)

Ackers and Charlton (1970) established the relationship between the meander wavelength and dominant discharge from the experimental data as

$$M_L = 62 Q^{0.47}$$

(2.15)

Chitale (1970) provides some evidence that axial wavelength also varies with slope and relative roughness.  
Gorycki's (1973) study suggests that the meander wavelength,

$$M_L = 17.2 W \quad (2.16)$$

JSCE task committee (1974) and Yalin (1971) found the following relationships as:

$$M_L = L_1 W n_1 \quad (2.17)$$

and  $M_L = 2 * \pi * W$   
where  $n_1 = 0.98$  to  $1.27$

Yasuda et al. (1974) found the relation as

$$M_L/d = k_2 (Q^2/gd^5)^{n_2} \quad (2.18)$$

where  $n_2 = 0.186$  to  $0.279$

Ferguson's (1975) relationship between meander wavelength and discharge as :

$$M_L = k_3 Q^{n_3} \quad (2.19)$$

where  $n_3 = 0.34$  to  $0.67$

In 1975 Charlton found the relation between the meander wavelength and mean discharge ( $Q_L$ ) as

$$M_L = 166 Q^{0.46} \quad (2.20)$$

Parker (1976) based upon analytical solution found that

$$M_L / (Wd)^{1/2} = k_4 F^{1/2} \quad (2.21)$$

Where  $F$  is the Froude number.



Hawashi (1980) also found analytically the relation for the dominant meander length as

$$M_L = 3.66 (1.F + F^{-1}) \quad (2.22)$$

Where F varies between 0.1 to 1.0

Ikeda et al, (1981) developed a relationship for meander wavelength as

$$M_L/d = 8 (\pi W/fd)^{1/2} \quad (2.23)$$

Where  
 $d$  = depth of flow  
 $W$  = width of the channel at bankfull stage  
 $f$  = Darcy-weisbach friction factor.

Edgar and Rao (1983) based on 61 laboratory channels with constant discharge data found that the relative meander length

$$M_L/A^{1/2} \propto F^{1/2} \quad (2.24)$$

$$\text{and } M_L/A^{1/2} \propto Q^{0.39} \quad (2.25)$$

Where  
 $F$  = Froude number and  
 $A$  = cross sectional area

Chang (1948a) developed a graphical relationships between meander wavelength and width-depth ratio. Wavelength-depth ratio is directly related to channel width-depth ratio and inversely related to channel roughness.

Hossain (1987) analyzed the data of the river Ganges for the period of 1965 to 1976 and found the following relationships :

$$\begin{array}{lll} M_L = 9.76 W & \text{and } M_B = 3.6 W & \text{for 1956} \\ M_L = 7.46 W & \text{and } M_B = 3.0 W & \text{for 1967 (2.26)} \\ M_L = 7.60 W & \text{and } M_B = 3.69 W & \text{for 1976} \end{array}$$

Sinuosity of thalweg line,  $S_r = 1.27$  and  $1.26$  for the year 1967 and 1976 respectively.

Hossain (1989) upon analyzing the field data for the period of 1973 to 1987 of the river Ganges found the following correlations:

$$M_L = 24.27 + 1.23 M_B \quad \text{when } S_r < 1.2$$
$$M_L = 15.7 + 0.96 M_B \quad \text{when } S_r > 1.2 \quad (2.27)$$

The  $M_L$  and  $M_B$  value are governed by the several factors such as sediment size, nature of geologic strata, sediment load, form of hydrographs, etc., and hence the co-efficient in the above equations are expected to depend on them (Blench, 1957).

The study also indicates that experimental results obtained under laboratory conditions are not always transferable to seemingly analogous field conditions (Chang, 1988).

## 2.5 River bend characteristics

A feature typical of meandering rivers is the bend of varying size and shape. In each bend a helical current is produced and causes a variation in bed level. Most bends are the part of a meander or deformed meander system. Bends are normally formed as a result of the natural tendency for sinuous flow in alluvial channel when the slope of the river is less than  $S=0.0017/Q^{1/4}$ . The shape of bends varies from beautifully symmetrical patterns to the deformed bends encountered most frequently in nature. Bends can be designated by the curvature ( $r_c / W$ ). In a sample of 50 rivers differing in size as well as in physiographic province, Leopold et al. (1964) found that 2/3 of the ratios for  $r_c / W$  value were in the range 1.5 to 4.3 with a median value of 2.7. In view of the this striking geometric regularity of winding rivers, they suggested that meanders are no accident and they

appear to be in the form in which a rivers does the least work in turning.

Chang (1984a) analyzed the meander curvature and other geometric features using an approach together with relations for flow continuity, sediment load, resistance to flow, bank stability, and transverse circulation in channel bends. The analysis established the maximum curvature ( $r_c / W$ ) for which a river does the least work in turning and the average value of that curvature was 3. Sinuosity varies among the meandering river, but  $r_c / W$  value remains similar in the different river bends of different sizes and physical settings (Leopold and Langbein, 1966 ; Leopold and Wolman, 1957).

### 2.5.1 Radius of bend curvature

Since bends are not, in general, circular arcs (Ferguson , 1975), difficulties arises in defining and measuring radius of curvature. A sine-generated curve (Leopold and Langbein, 1966) increases in curvature towards the apex, so it requires description of minimum ( apex ) radius and mean radius. The latter is the best defined by replacing the continuous curve of the bend by equally spaced points (about one channel width apart), connecting these by straight segments, and finding the centroid of the polygon defined by their perpendicular bisector. This represents the mean center of curvature, which can subsequently be used to initiate radii in order to find the mean radius (Hickin, 1974 ; Goudie et al., 1990). But in 1983, Hanson and Hickin redefined the radius of curvature in the form  $r_m = (r' + r'') / 2$  for their meander bend migration rate evaluation. In this method, the radius of curvature for the bend is the mean radius of two circles; one passing through  $x_2y_2$ ,  $x_3y_3$ , and  $x_4y_4$  the other passes through  $x_1y_1$ ,  $x_3y_3$  and  $x_5y_5$  (fig. 2.1). Where  $x_3y_3$  are the bend axis (point of maximum curvature). The digitizing interval,  $\Delta x$  for the five sets of coordinates is equal to the mean channel width measured at straight reaches

(usually at planform inflection points). This measures of bend radius reflects both the strong curvature at the bend axis and the broader sweep at the limbs of the bends.

The coordinates for this calculation are located on the convex bank because it represents a bend curvature intermediate to those defined by the concave bank line and the scroll bar marking the beginning of the migration period in question.

### 2.5.2 River bend migration

A river is the authour of its own geometry. It is adjusted in the long term. The adjustments include channel geometry, slope, meandering pattern etc. (Chang 1984b). In this process, the migration at meander bends is a characteristic feature of alluvial rivers and one of the most conspicuous changes affecting fluvial landscapes. River bends shift laterally by erosion of the concave bank and concomitant deoposition at the convex bank. This phenomena is of considerable scientific and engineering significance. An attempt is of great importance to systematically examine the range of migration rates on a variety of river types. At present there is no means of expalining or predicting channel migration rates for selected reaches of variety of rivers where rates have never been actually measured. In general, the rate of channel migration,  $M$ , is likely to depend primarily on the variables as stream power ( $\Omega$ ), opposing force of boundary area ( $y_b$ ) bank height, ( $h$ ), bend radius ( $r_c$ ) & channel width ( $w$ ).

But detailed measurements on the Beaton river, Canada, clearly show that channel bend migration rates are strongly controlled by bend curvature (Nanson & Hickin, 1983).

The basic relation is, the migration rate,

$$M = f ( r_c / W ) \quad (2.28)$$

Sediment transport also influence the channel migration rates. However, maximum curvature in terms of  $r_c / W$  is found to be a function of the channel slope, discharge, sediment size, and the average width-depth ratio (Chang, 1984b). Hickin and Nanson (1975) found in a survey of ten point bar complexes on the Beaton river in British Columbia, Canada, that the rate of channel bend migration reached a maximum value when the value of  $r_c / W$  approximated 3. The rate of channel migration rapidly declined for bends with values of  $r_c / W$  greater or less than 3. The radius of curvature used in this study was mean radius defined in Art. 2.5.1.

In the typical meander bends maximum curvature for the value of  $r_c / W$  ranges from 2.2 to 4.0 ; it varies in direct portion to the channel slope or the width-depth ratio (Chang, 1984 b ). Chang (1984 b ) found analytically that the maximum channel bend curvature occurred when  $r_c / W = 3.2$  and the median value is about 3.0. On the other hand Leopold and Langbein (1966) found it as 2.7 from field measurements. When the width-depth ratio is small, possibility of development of very sinuous channels. Width-depth ratio is found to be an important factor governing meander geometry ( Hey, 1976 ; Lane, 1957 ; Schumm, 1973). While the  $r_c / W$  ratio is remarkably constant, it does indicate the possibility of more acute bends for streams of flatter slopes which are also associated with a smaller width-depth ratio.

In 1983, Nanson and Hickin redefined the radius of curvature in the form  $r_m = (r' + r'')/2$  as shown in Fig.2.1 and analyzed additional field data relating to bend migration rates for the Beaton river which confirm the form of the relationship the writers described in 1975. He also found that the channel migration is a discontinuous process within any single bend and the river is down cutting at a rate of about  $2 \text{ mm/y}_r$ , not at  $10 \text{ mm/y}_r$ , as reported earlier.

In 1984, Hickin and Nanson studied on two different river reaches in Canada and had tried to establish the general relationship between bend migration rates (M) and bend curvature ratio ( $r_c / W$ ) for all field sites. He found that maximum migration rates occurred at  $2.0 < r_c / W < 3.0$  and when  $r_c / W < 2.0$  as the bend curvature tightens, migration rates per unit channel width (M/W) quickly tends to zero at  $r_c / W = 1.0$ . These results are also similar to the results shown in 1983 (Nanson & Hickin).

In 1984, Hickin & Nanson also proposed a quantitative method to relate the bank erosion or bank migration rates to :

- stream power of the river or channel
- bend characteristics
- resistance of the bank

Stream power of the river is  $\Omega = \rho g Q s = \gamma Q s$  (2.29)

|       |          |   |                                                                                                                                                                          |
|-------|----------|---|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
|       | $\Omega$ | = | Stream power (N/s)                                                                                                                                                       |
| Where | $\rho$   | = | density of water ( kg/m <sup>3</sup> )                                                                                                                                   |
|       | $g$      | = | acceleration due to gravity ( m/s <sup>2</sup> )                                                                                                                         |
|       | $Q$      | = | river discharge ( m <sup>3</sup> /s ), 5 year discharge which is slightly larger than bankfull discharge. It is assumed that the bankfull discharge can be taken as well |
|       | $s$      | = | river slope                                                                                                                                                              |
|       | $\gamma$ | = | $\rho g$                                                                                                                                                                 |

The resistance of the bank can be represented by a co-efficient  $Y_b$ , that is shown to be a function of the particle size composition of the bank material (fig.2.2) ( Hickin and Nanson, 1984 and FAP-21/22, Vol. 2, 1992).

They demonstrated on the basis of a number of rivers in Canada that the bank erosion rate can be computed with the following relations :

$$E = 2.5 M_{2.5} / (R/B) \quad (\text{for } R/B > 2.5) \quad (2.30)$$

$$E = (R/B - 1) M_{2.5} / 1.5 \quad (\text{for } R/B < 2.5)$$

Where  $E$  = annual bank erosion (m),  $M_{2.5}$  = annual bank erosion for  $R/B = 2.5$ ,  $R$  = radius of curvature of the bend (m),  $B$  = width of the river (m). Value of  $M_{2.5}$  can be found from the following relation

$$M_{2.5} = \Omega / (Y_b H) \quad (2.31)$$

where  $H$  = outer bank height (m)

The value of  $Y_b$  has to be verified by comparing with the field data. In 1984, Ferguson suggested an alternative conceptually superior and convenient (as his opinion) method for measuring the radius of curvature and an appropriate idea about the bend curvature or simply curvature. But on the conclusion Nanson and Hickin (1984) opined that in measuring the radius of curvature their method is less complicated and comparatively accurate.

Hooke (1987) critically reviewed the Hickin and Nanson's (1984) consideration that bend migration rate is dominantly governed by the bend curvature ( $r_c / W$ ), and investigated further for the controlling variables of the river bend in southern Devon, England. Finally he concluded that they were unaware of any attempt to examine systematically a range of migration rates on a variety of types. He found that, there eight independent variables governing the bend migration rates of which drainage area and silt-clay percentage are the dominant variables, though the curvature of the bend is important in influencing the distribution and magnitude of erosion and, therefore migration of the bend. Klaassen and Vermeer (1988) studied the channel characteristics of the braided Jamuna river in Bangladesh. They found that, the bend characteristics are approximately similar to meandering rivers, which may indicate that individual channels of a braided river may to some extent be comparable to the single channel of meandering rivers. Klaassen and Masselink (1992)

studied the bank erosion rates for Jamuna river and found the erosion rates much faster than that predicted by Nanson and Hickin (1984)(fig.2.3). They also found a negative correlation between the relative bend curvature ( $r_c / W$ ) and erosion rate ( $E/W$ ). There was no definite tendency for very sharp bends ( $r_c / W < 2.5$ ) to have smaller erosion rates as was observed by Nanson and Hickin (1984) for meandering rivers.

## 2.6 Dominant discharge

Major part of the following text were taken from Hossain, 1979. The flow in a river varies widely over the year as well as from year to year. A river carries both water discharge ( $Q$ ) and sediment discharge ( $Q_s$ ) and they together decide upon the shape and characteristics of the stream. The sediment discharge,  $Q_s$ , also varies widely with the variation of water discharge. The ratio of maximum to minimum discharge may be as high as 1000 or more. The flow in the medium sized river varies widely. The ratio of maximum to minimum discharge for both liquid and sediment may be as high as possible because in dry season even no flow occurs. But to maintain an equilibrium condition for alluvial channels, it is widely believed that this ratio for both liquid-sediment flow should be as low as possible.

Problems of river morphology are very complex. For this reason simplification is a must. But, the simplified river regime can never replace the real regime as far as the reproduction of the morphological characteristics of a river reach are concerned. The widely varying flow pattern makes it difficult to choose a representative discharge in studying the alluvial river characteristics.

Different methods have been proposed by different investigators for the choice of representative discharge. This representative discharge is otherwise referred to as "dominant discharge". The dominant discharge concept has widely been used for an efforts to define a river regime into one single discharge.



Originally the dominant discharge concept was introduced in combinations with the regime theory in order to extend the regime theory to those rivers in which discharge change and vary throughout the year (Inglis, 1968). At this discharge, equilibrium is most closely approached and the tendency to change is the least.

Inglis (1947), defined the dominant discharge as that hypothetical steady discharge which would produce the same result as the actual varying discharge. Inglis suggested that for most of the Indian rivers the dominant discharge was nearly the same as bankfull discharge and further recommended that the dominant discharge may be taken to be half to two thirds of the maximum discharge.

Schafernok (1950), proposed based on the reasoning that the dominant discharge should be the discharge at which most of the formative works is done and should "therefore" correspond to that stage at which the bulk of the bed load is carried. But it is difficult to decide at which stage the bulk of the bed load is carried.

Leopold and Maddock (1953), preferred to use a discharge of particular frequency of occurrences for comparing the hydraulic geometry of streams. According to them, the mean annual discharge which was found to be equalled or exceeded about 25 percent of time was most suitable for studying the variation of hydraulic geometry along the river.

Blench (1957), designated the dominant discharge as that discharge which is equalled or exceeded 50 percent of time. This definition is valid for American rivers.

Torrel and Borland (1958), defined the dominant discharge as the discharge that will carry the greatest amount of sediment load

coarser than 0.0025 mm and found that this discharge is slightly greater than the median discharge.

Nixon (1959), preferred to use bankfull discharge for studying the alluvial channel geometry in England and Wales. He found that the average frequency of this discharge is about 0.6 percent. As this frequency is a function of climatic condition, drainage characteristics and river order which are highly variable, departures from the above values are likely in other rivers.

Ackers and Charlton (1970), defined dominant discharge as the steady flow that would produce the same meander wave length as the observed range of flows within which that steady flow lies. This definition relates the dominant discharge with a new parameter, the meander length.

Wolman and Gerson (1978), extended the Wolman's original contribution on dominant discharge in arguing that the effectiveness of flow event reflects the morphological changes it causes through erosion and deposition, as well as the associated sediment transport. These are, however, complementary definitions in that it should be expected from basic principles that the flow doing most of the work as the channel would be responsible for forming and scaling the salient parameters of its geometry, size and sedimentary features.

NEDECO (1985), has tried to use a method based on a more physical reasoning to determine the dominant discharge. NEDECO recommends finding the depth  $h_0$  corresponding to dominant discharge using the equation.

$$h_0 \int_0^T Q_b dt = \int_0^T Q_b h dt \quad (2.32)$$

$$\text{or, } h_0 = \frac{\int_0^T Q_b h dt}{\int_0^T Q_b dt} \quad (2.33)$$

in which  $Q_b$  is the bed load transport rate.

In the JMBA (1990) study of the Brahmaputra, the dominant discharge is defined as "that steady discharge which had it operated continuously for the period of record, would have transported the same amount of sediment as the range of flows which actually occurred".

The analysis would be done according to the approach suggested by Schafernok (1950) and the result may be compared with the Blench (1957) flow duration concept.

#### 2.6.1 Bankfull discharge and bankfull stage

The river discharge which exactly fills the river channel to the bankfull level without spilling on to the flood plain. Bankfull discharge has been often assumed to be a significant or critical channel forming flow which is important in determining the size and shape of the river channel and it has therefore sometimes been regarded as equivalent to dominant discharge (Goudie, 1985). But in terms of sediment transport specifically of bed load transport studies from different rivers have demonstrated that the dominant discharge for bed load transport is not the same as the bankfull discharge. Therefore, the discharge at the bankfull stage should not be thought of as the formative discharge (Partin, 1983). Always the bankfull discharge approximated the discharge

slightly higher than the actual dominant discharge estimated from the bed load transport considerations.

Nevertheless, it does provide a working approximation of the dominant formative discharges.

The frequency of occurrence of bankfull discharge varies considerably and has been equalled as ranging from 4 months to 5 years in Scotland, 1.3 to 14 years in lowland England and 1 to 30 years in 28 basins in the Western USA (Nixon, 1959).

On hydrological grounds there is a significance of the channel bankfull stage. Flow resistance decreases as water depth within the channel increases; reaches a minimum at bankfull stage; and increases again as the water flows out of the channel and on to the adjacent flood plain. Various possible definitions of the bankfull stage (at least 16 different methods) have been used (Williams, 1978a) referring to : the heights of the valley flat, the active flood plain, the benches within the channel, the highest channel bars, the lower limit of perennial vegetation, the upper limit of the sand sized particles in the boundary sediment, the elevation at which the width-depth ratio becomes a maximum, where there is a first maximum of the bench index (Riley, 1972 after Goudie, 1985), or the relation of cross-sectional area to top width changes. Williams (1978a) concludes that the bankfull level to the active flood plain level is the most useful of the fluvial geomorphologists, whereas the banks of the valley flat are the most important to engineers.

## **2.7 The morphology of river cross-sections**

Alluvial rivers show certain consistencies in terms of flow and morphology. In accommodating a change in discharge rivers can change width, depth, slopes and velocities. These changes appear to be as uniformly distributed among these components as is permitted by the river mechanics. In developing their profiles,

rivers in humid regions tend toward equal power per unit of area and toward equal power per unit of length. In the context of river cross-sections, the adjustments of river morphology to discharge variations is manifested in the "hydraulic geometry" concept which is concerned with changes in flow geometry.

### 2.7.1 Hydraulic geometry

Stream channels vary in form and experience a wide range of flows: the mean form has been described in the literature under the term "Hydraulic geometry" and at different discharges the observed mean velocity, cross-sectional area, width and mean depth of flowing water reflect the hydraulic characteristics of the channel cross-section. Channel width is considered as a primary morphological variable (Leopold and Wolman, 1957).

Hydraulic geometry is an empirical model devised by Leopold and Maddock (1953) to provide a quantitative description of stream behaviour either at a particular cross-section or along a particular stream. Leopold and Maddock (1953) have provided, extending regime concept of alluvial channels to the rivers in the U.S.A. an empirical frame work for the expression of consistent relationships between flow and morphology and defined the hydraulic geometry of alluvial stream channels as: (1) the at-a-station adjustment of flow characteristics such as width ( $w$ ), depth ( $d$ ) and velocity ( $v$ ) within a section as discharge ( $Q$ ) varies; and (2) the general downstream adjustment of these flow properties as discharge increases at a constant flow frequency. Plotting channel top-bank width  $w$ , cross-sectionally averaged depth  $d$ , and velocity  $v$  at mean annual water discharge  $Q$  versus that discharge as it varies down stream for various streams. They established power-law for  $d/s$  hydraulic relations of the form

$$w = aQ^b ; \quad d = cQ^f ; \quad v = kQ^m \quad (2.34)$$

Both at-a-station and down-stream, these adjustments are commonly described by the similar power functions :

$$w = aQ^b ; \quad d = cQ^f ; \quad v = kQ^m \quad (2.35)$$

and since  $wdv = Q$ , it follows that  $ack = 1$  and  $b+f+m = 1$

In defining the term hydraulic geometry, Leopold and Maddock (1953) considered the water and sediment discharges in a river as independent variables and the characteristics of the channels as dependent variables which are evolved such that approximate equilibrium is achieved between the channel and the water sediment loads that it must transport.

In the development of quantitative relationships for the hydraulic geometry of river cross-sections significant contributions have been made by numerous authors namely : Leopold and Maddock (1953), Leopold and Wolman (1957), Blench (1961), Langbein (1964), Engelund and Hansen (1967), Williams (1978b) and Chang (1979) etc.

#### 2.7.1.1 At-a-station hydraulic geometry

The discharge "at-a-station" or at a given cross-section varies synoptically and seasonally from low flow to flood. The variation is accompanied by changes of hydraulic factors. The water area within a cross-section is itself the product of the width and mean depth of flow. So that the continuity equation may be re-written as :

$$Q = WdV \quad (2.36)$$

Again differences in any one parameter between cross-sections must be matched by opposing changes of one or both of the other two variables. The way in which these components (width, depth and velocity) vary at a cross-section as discharge changes has

been described as the at-a-station hydraulic geometry (Leopold & Maddock, 1953). Different channel shapes will produce different changes of the width, depth and velocity components in order to accommodate variations of discharge. For example, a narrow deep channel with vertical banks will accommodate changes of discharge primarily by : changes of water depth, a triangular channel will adjust all three components  $w$ ,  $d$ , and  $v$ . Whilst in wide shallow channels with gently sloping banks, width will be the most important components (Petts, 1983). In the development of quantitative relationships between different hydraulic geometric parameters of river cross-section. Significant contributions have been made by numerous authors as mentioned earlier. Lacey (1947) originated a regime concept and found based on his regime theory for irrigation canal with the data obtained in a two-year period, the following relations :

$$\begin{aligned} W &\propto Q^{0.5} \\ d &\propto Q^{0.333} \\ V &\propto Q^{0.167} \end{aligned} \quad (2.37)$$

Leopold and Maddock (1953) as a result of additional investigations in different geographical regions (Midwest USA) found the average relationships for at-a-station hydraulic geometry as :

$$\begin{aligned} W &\propto Q^{0.26} \\ d &\propto Q^{0.40} \\ V &\propto Q^{0.34} \end{aligned} \quad (2.38)$$

for midwestern rivers,

$$\begin{aligned} W &\propto Q^{0.12} \\ d &\propto Q^{0.45} \\ V &\propto Q^{0.43} \end{aligned} \quad (2.39)$$

for the average of 158 gauging stations in United States.

And

$$W \propto Q^{0.13}$$

$$d \propto Q^{0.41}$$

$$V \propto Q^{0.43}$$

(2.40)

for ten gauging stations on Rhine river. Engineering problems often require the variations of channel parameters with discharge at-a-station. The regime equations do not explain how the hydraulic geometry varies in development of a scouring pilot cut in a river.

At-a-station flow geometry reflects the general cross-sectional shape, which itself dependent on material properties. The exponent set can be predicted using minimum variance theorem.

Langbein (1964) on the basis of his laboratory experiments conforming the rivers in humid region that there is a liberty to adjust its profile, velocities, depths, and widths to accommodate the down stream increase in discharge. That is there were three degree of freedom. He analyzed the data with the theory of minimum variance and found that

$$W \propto Q^{0.23}$$

$$d \propto Q^{0.42}$$

$$V \propto Q^{0.35}$$

(2.41)

These results compare quite favourably with those reported for river cross-sections by Leopold and Maddock (1953). The above result is for at-a-station hydraulic geometry.

Leopold, Wolman and Miller (1964) found for American rivers the following at-a-station relationships :

$$W \propto Q^{0.18}$$



$$d \propto Q^{0.52}$$

$$\text{and } V \propto Q^{0.30}$$

(2.42)

Most natural river cross-sections display more complex behavior than conventional hydraulic geometry suggests. Lewis (1966) noted that flow at low discharge is often confined to a secondary channel in the stream bed and the width-discharge curve displays a break of slope when its capacity is exceeded. Richards (1976) extends this idea to the consideration of the whole cross-section.

Park (1977) had tried to stipulate the world wide variations of at-a-station hydraulic geometry and found the following relationships as :

$$W \propto Q^{0.13}$$

$$d \propto Q^{0.40}$$

$$V \propto Q^{0.47}$$

(2.43)

for the Langedalen river, Norway, with steep banks.

$$W \propto Q^{0.32}$$

$$d \propto Q^{0.40}$$

$$V \propto Q^{0.28}$$

(2.44)

for the Langedalen river, Norway, with low angle banks.

$$W \propto Q^{0.08}$$

$$d \propto Q^{0.52}$$

$$V \propto Q^{0.40}$$

(2.45)

for the Hoober river, U.K., with very steep banks.

$$W \propto Q^{0.12}$$

$$d \propto Q^{0.37}$$

$$V \propto Q^{0.51}$$

(2.46)

for the small, high gradient stream with boulder bed, Dartmoor, U.K.

Goudie (1985) pointed out that at-a-station flow geometry varies between rectangular sections with steep cohesive banks in humid areas, and parabolic sections in sandy sediments in semi arid regions. In the former,  $b = 0.05$ ;  $f = 0.45$  and  $m = 0.50$  and in the latter  $b = f = m = 0.33$ .

Chang (1985) analyzed the regime geometry and channel patterns of sand bed river analytically (using an energy approach) and found the result consistent with the field data. The quasi-equilibrium channel geometry is usually related to the slope, discharge, and sediment properties (Schumm, 1968). Such relationships are by no means continuous because there exists several apparent thresholds or discontinuities. Therefore, regime adjustment involving a small change in slope may lead to a large change in channel pattern (Schumm, 1977). Das (1992) also analysed the data for the river Ganges and found the at-a-station relationships as :

$$w \propto Q^{0.172}$$

$$d \propto Q^{0.246}$$

$$v \propto Q^{0.575}$$

(2.47)

## CHAPTER III

### DESCRIPTION OF THE RIVER & DATA COLLECTION

#### 3.1 The Gorai - Madhumati river

The Gorai - Madhumati river is one of the main distributaries of the Ganges flowing through the deltaic plains (fig.3.1). The Gorai takes off from the right bank of the Ganges near Talbaria, Kushtia in the name Gorai. From where it flows south-eastwards to the Bay of Bengal and falls through Haringhata estuary. The total length of the river, upto the down of patgati, is about 258.5 km (161.5 miles). The river has different names at different parts of its course. Towards the upstream part above kamarkhali ghat, it is known as the Gorai, down stream of kamarkhali ghat it flows by the name Madhumati and further down near patgati, it takes the name Baleswar and falls into the Bay through Haringhata estuary. This is a moderately old river. The history of the river is more than 230 years old. In tracing the changes in the rivers in the Jessore and Fariapur districts Major James Rennell's (a surveyor on British Govt.) found the Kushtia creek (Gorai) in June 1764. The river was 130 to 200 yards wide and 15 to 60 ft. deep near off take. The depth had been reduced very sharply inside the country. Rennell (1767 after Williams, 1919) mentioned the name of the Nabaganga and Barassia but no mention was made of the Madhumati. The lower reaches of the Barassia appeared from the Rennell's map (1767) to have followed much the same course as the the Madhumati does now (Williams, 1919).

In Westland's (1870 after Williams, 1919) history of Jessore it was stated that the Madhumati probably was the out come of the severe floods which swept over the district between 1795 and 1801. On the other hand Fergusson (1863 after Williams, 1919) stated that the Madhumati formed some time between 1818-20 and

1830-33. The statement in Westland's history may incorrect as Fergusson possessed a close knowledge of these river at the time the changes took place (Williams, 1919). Before 1911-12 probably the rivers Gorai and Madhumati were the different rivers. Location of the Madhumati was in the east and that of Gorai was on the West (Williams, 1919).

The evidence of the recent years shows that the flow into the Gorai-Madhumati from the west has been gradually stopped and the current is now reversed. So that there is a tendency for both the rivers to change their courses westwards : they have built a ridge which means higher water levels and they are now endeavouring to seek a new course in lower land. The Gorai has considerably improved since Rennell's time (Williams 1919).

This activity probably depends on the position of its off take relatively to the main stream of the Ganges (parent) river. Brahmaputra river changed its old course from Mymensingh-Gafargaon, the change appears to have been a more or less gradual process, occurring about the end of the 18th century. The combined flow of Teesta and Brahmaputra met the Ganges near Goalundo and heading back of the Ganges flow by the Brahmaputra considerably influenced the flow of Gorai. Before the changes of the course of the Teesta and the Brahmaputra it was an insignificant outlet, but after the changes had been affected, this river assumed to be a larger river. The width of the Gorai near off take was 183 m in 1828 and had gradually widened to 582m and the spill was also increased (Williams, 1919).

The flow of Gorai how much would be affected due to the heading back of the Ganges by the Brahmaputra can be estimated as follows the off take of the Gorai is at a distance 72 km above Goalundo, if the water level of the Ganges at Goalundo raised 1.83m taking the average surface slope of the Ganges at 5.23cm per kilometer and assuming that during the high flood there is a depth of 9.14m at Goalundo then the effect would be felt at the Gorai off take

where the level would be raised about 0.686m and if the depth of the Ganges was 12.20m at Goalundo the raise would be about 0.85m (Williams, 1919).

It is therefore clear that the Gorai might have been considerably influenced by the change in the course of the Brahmaputra. From this stand point it was assumed that the Ganges may be diverted through the Gorai but this avulsion ultimately did not take place. The possible explanation of this phenomena (irruption) not to happen would be that the changes in the course of the Brahmaputra together with Teesta, reinforced the Ganges flow and robbed the Meghna. So that the fall across the intervening neck of land was considerably increased with the result that the Ganges (the Padma) took the easiest course via the Kirtinasa (Callyganga on Rennell's map) which gave it a lead i.e., the callyganga in Rennell's map appears to be the Kirtinasa through which the Padma broke to join the Meghna at Chandpur : it is not clear when this change took place but in 1840 the new river was 3 to 4 miles wide and in 1871-72 Rajnagore was swept away by the Kirtinasa. This change may likewise have relieved the Padma sufficiently of its additional volume to prevent the Ganges breaking out above Goalundo through the Gorai as it threatened to do in 1828-1863 (Williams 1919).

The Kumar - Nabaganga join the Madhumati river near Bardia through a Halifax cut and a river, Atharabanki, takes off from the Madhumati near Mollahat. At the down of Patgati the Madhumati bifurcates into the Baleswar and Kalinganga. The river Bhairab meets the Baleswar flowing through Bagerhat and the lower part of the Kalinganga meets the Swarupkati river near Kaukhali and then renamed as Kocha river.

The Kocha and Baleswar again join together near Umedpur and flows down in the name Baleswar (some times it is called as Kocha). Receiving the flow of Panguchi river on the right bank the Baleswar in turn take the name of Haringhata from the Bogi forest

out post of the Sunderbans (fig.3.2). The estuarine part is very deep and broad. The Gorai Madhumati river is now an important distributary of the Ganges which brings fertility to the mid part of delta region and provides direct navigational facilities to country boats from the sea to the Ganges. The drainage area upstream of the Bardia is approximately 2443 sq-kms. and the drainage plans for Kamarkhali transit is built of sandy clay loam soil with intense cultivation. The vegetal cover is moderate to thick. Banks of the stream are highly erodible.

### 3.2 Present flow conditions of Gorai-Madhumati river

The flow of Gorai-Madhumati river largely depends upon the flow and morphological conditions of the Ganges at the off-take of Gorai. During the last three decades, diversion of Ganges water to the Gorai has been highly variable due to moving sandbars which periodically seriously obstruct the flow into Gorai. The average annual discharge at Gorai Rly bridge and Kamarkhali transit are 1409 and 1255 cumecs respectively. The average minimum flows at the Gorai Railway bridge and at Kamarkhali transit before 1975 were 87.32 and 108.50 cumecs respectively and those after 1975 are 28.83 and 25 cumecs respectively. The minimum flows are 0.057 cumecs and 0.764 cumecs at Gorai Railway bridge and at Kamarkhali transit respectively. The average discharge in winter at Gorai-Railway bridge was 475.42 cumecs and in rainy season 6035.35 cumecs in the year 1969 and the maximum discharge was 7568 cumecs in August 28, 1969. In 1988, the year of devastating flood in Bangladesh, the highest discharge was 8490 cumecs on 02.09.1988 at Gorai Rly bridge site. On the basis of the data available from 1946 to 1990 in surface water Hydrology of BWDB, the recorded highest discharge at Gorai Railway bridge is 8490 cumecs in 1988 and at present even no flow occurs during the lean period. Actually in winter no water of the Ganges falls into the Gorai and hence the upstream parts remain almost dry. But in the rainy season huge amount of water from Ganges river pass through the Gorai river, the maximum daily

average silt content measured at Gorai-Rly bridge site is 550 ppm. The average median sediment size  $d_{50}$  at Gorai-Railway bridge was 0.13 mm (Surface Water Modelling Center 1991) and 0.048 mm (Walliuzzaman, 1986).

Due to serious siltations and withdrawal of Ganges water at the upstream, Water level in the river Gorai is falling gradually and bed level rising up. Siltation problems arises at the intake and insufficient flow rates cause saline water intrusion at the downstream. This saline water is harmful for the industrial processes etc.

There are many important projects on either sides of Gorai-Madhumati river are on going such as baliakandi Irrigation Project (Rajbari), Gorai-Irrigation project (Magura), Old river Resuscitation Project (Magura), Barasia Irrigation Project (Gopalgonj), Magura-I Irrigation Project (Magura) and Chenchuri beel Irrigation and Drainage Project (Narail) (fig.3.3). The performance and efficiency of the above projects (net benefitted area of about 191,240 hectares are tremendously affected by the hydraulic and morphological characteristics of this river (Walliuzzaman, 1986).

It is estimated by the special studies, BWDB (1981 after Walliuzzaman, 1986) that the minimum flow in Gorai for extreme low water condition would be approximately 190 cumecs and the corresponding minimum discharge at Ganges at Hardinge bridge measurement station would be 629 cumecs to save the aforesaid projects. It is observed that water discharge d/s of the Gorai-Rly bridge gradually decreases. In the years 1967, 1969, 1974, and 1987 the maximum water discharges at Gorai-Railway bridge were 6887 cumecs, 7568 cumecs, 8460 cumecs and 7500 cumecs and at Kamarkhali ghat the corresponding maximum discharges were 4704 cumecs, 5299 cumecs 7958 cumecs and 6360 cumecs respectively.

On the other hand the minimum discharges at Gorai Railway bridge were 111 cumecs, 41 cumecs, 117 cumecs and 7.90 cumecs and at Kamarkhali ghat the corresponding minimum discharges were 147 cumecs, 58 cumecs, 100 cumecs and 5 cumecs in the above mentioned years respectively.

The minimum discharge at Gorai-Railway bridge in 1988 was only 0.286 cumec on the contrary the maximum discharge in the same year was app. 8490 cumecs, i.e., the ratio of maximum discharge to minimum discharge was approximately 30,000, which is too high. That is, discharge variation throughout the year is as high as possible.

It also appeared that after 1975, the flow of water in the lean period have been seriously affected and decreased to about zero flow.

### 3.3 Past studies

In Bangladesh, studies on the morphological characteristics of the river are limited. However, numerous studies on the Ganges had been undertaken and also there some studies on the Brahmaputra-Jamuna and Meghna, but studies on the medium sized river are very limited.

IECO (1964), the consultant of EFWAPDA had compiled the hydrological data for master plan (after Hossain, 1989). Chowdhury (1966 after Hossain, 1989) studied on the gradual shifting of the Ganges from west to east in delta building operations using historical maps. Coleman (1969) studied the channel process and sedimentation of Brahmaputra-Jamuna river. Dad (1977) studied the shifting pattern of the Ganges based on the plan map of the bank lines. Rahman (1978) had studied the erosion of Padma river from Goalundo to confluence of Padma-Meghna near chandpur and reported on lateral movement of banks. SPARRSO has been collecting the aerial maps of the rivers using



remote sensing techniques to study river morphology. Bangladesh Water Development Board (1979) investigated the bank line movement of the river channel shifting of large rivers of Bangladesh. NEDECO (1983 after Hossain, 1989) investigated the feasibility of Ganges-Kobadak Rehabilitation and Improvement project. Ahsan and Nishat (1985) prepared a list of reports where sediment data may be available for various rivers of Bangladesh.

Bari (1978) has analyzed the sediment data of the Ganges river and concluded that Engelund-Hansen and Colby formula yield good results.

Waliuzzaman (1986) analyzed the sediment data of the river Gorai-Madhumati at Gorai-Rly bridge and found that Mantz and Engelund-Hansen eqn, yields the better result. The average daily minimum and maximum sediment concentration in the Ganges was reported to be about 67 ppm and 2344 respectively. On the other hand, the average daily maximum sediment concentration in the Gorai was reported to be 550 ppm.

NEDECO (1983 after Hossain, 1989) analyzed the grain size curves for the Ganges at Hardinge Bridge and found that the median size i.e.,  $d_{50}$  of the sediment load was about 0.14 mm and  $d_{50}$  for the wash load was about 0.04 mm.

The value of  $d_{50}$  for the upper Meghna is 0.14 mm (Haskoning & DHI, 1992). Bangladesh Water Development Board analyzed the grain size curves for the Gorai-Madhumati river at Gorai-Rly bridge and reported the value of  $d_{50}$  for the sediment to be 0.048 mm (Waliuzzaman 1986). On the other hand Surface Water Modeling Center (1991) analyzed the grain size curves for Gorai-Madhumati river at Gorai-Rly bridge and found the value of  $d_{50}$  for the sediment to be 0.13 mm. RPT-NEDECO/BCL (1990) the consultants of Jamuna Multipurpose Bridge Authority (JMBA) also investigated the hydrological and morphological characteristics of the river Jamuna for bank protection and river training for the Jamuna

Multipurpose Bridge Project. Hossain (1989) studied the geomorphic characteristics of the river Ganges upto Brahmaputra confluence.

Halcrow and Partners Ltd. (1991) investigated the geomorphological characteristics of the river Brahmaputra-Jamuna for its bank stabilization and river training purposes. Consultants of BWDB Haskoning & DHI (1992) Jointly carried out a study on Hydrology and river morphology and geomorphology for bank protection works at critical locations along the Meghna river. Consulting consortium FAP-21/22 (1993) carried out detail morphological and hydrological study on the Brahmaputra-Jamuna for the bank protection and river training (AFPM) purposes at the selected locations. Significant hydrological and morphological studies to understand the characteristics and behaviour of medium sized rivers like Gorai-Madhumati etc. have not yet been done.

### 3.4 Data collection

For the present morphological study different types of data were obtained from different sources. The type of data can be listed as follows :

- hydrological data (water level, discharge and slope)
- maps ;
- satellite images ;
- cross sectional maps ;
- sediment transport data ;

### Hydrological data

Time series hydrological data were needed for the river morphological study, in particular for the determination of the river characteristics. The quality of the study depends upon the availability and accuracy of the sufficient hydrological data.

The data were collected from directorate of surface water hydrology, BWDB for the stations Gorai-Railway Bridge and Kamarkhali Ghat (Transit) for the period of 1946 to 1990 with a gap of 1971.

### Maps

Old maps can be used to determine the location and the planform of a river, and provided that they are made during low flow. Old maps of the Gorai-Madhumati river were obtained for the year 1953 from Survey of Bangladesh (SOB) in the scale 1 : 50,000 and for the year 1956 from SWMC in the scale of 1 : 15,840.

### Satellite images

Satellite images are the useful instrument to get insight in the changes in channel pattern. It is preferable to use images with approximately the same stage as only then the bank line changes will not be mixed up with changes due to a different stage. The prints that were made by SPARRSO were not at exactly the same magnification which induced some error. Satellite images used were LANDSAT images. Six different series were required to cover the entire Gorai-Madhumati river. The Satellite images (LANDSAT images) were obtained from SPARRSO for the years 1973, 1980 and 1987. The scales of these maps were 1 : 50,000.

In addition to the above mentioned LANDSAT images, also spot images for the year 1990 were available at Surface Water Modelling Center. These images were at a scale of 1 : 50,000. These were used for the analysis of the bend characteristics (bend migration rates etc.) and meander geometry. The quality of the spot images were much higher than the LANDSAT imageries.

### Cross-sectional data

The cross-sections sampled by EWDB are usually approximately 4 miles apart and these are measured once every four years. The cross-sections are connected to bench marks which are located on both sides of the cross-sections. These bench marks are known in location and level. For the Gorai-Madhumati river cross-sectional data were obtained from directorate of morphology, EWDB for 32 cross-sections for the year 1968 to 1990. The data were obtained in graphical form.

### Sediment transportation data

Only a limited number of results of bed load and suspended load measurements in the Gorai-Madhumati river were available. The measurements have been carried out at Gorai-Railway bridge. Only, the sediment data used by Waliuzzaman (1986) for the investigation of sediment transport characteristics of this river have been reused in the present study. But one exception is that Waliuzzaman used the  $d_{50}$  value 0.048 mm in computing the sediment load but in the present study the value of  $d_{50}$  also used as 0.13 mm as per SWMC (19991) data.

## CHAPTER - IV

### DATA ANALYSIS AND DISCUSSION OF THE RESULTS

#### 4.1 Dominant discharge

##### 4.1.1 Dominant discharge for the Gorai-Madhumati river

In estimating the dominant discharge according to the Schafernok (1950) concept sediment rating curves (fig.4.1 & 4.2) and flow frequency distribution curves (fig.4.3 & 4.4) were constructed first. Then with the help of these curves sediment transport Vs. discharge curves (fig.4.5 to 4.8) for different discharge classes were constructed for Gorai-Railway bridge site. A flow duration curve (fig.4.9) was also constructed to observe percentage of time the dominant discharge equalled or exceeded. The peak of the figs.4.5, 4.6, 4.7 and 4.8 defines the flow doing most of the work on the channel through the transportation of sediment : that is the dominant discharge.

From Figs. 4.5 to 4.8 it is appeared that the dominant discharge according to this concept to be  $4500 \text{ m}^3/\text{s}$ . The different modes of analysis, using different discharge classes, different sediment rating curves etc. gives approximately the same result. And this is quite a good result which is insensitive to the precise nature of the sediment rating curve used to derive it.

Table 4.1 reveals that the average discharge of different years (1965-88) corresponding to the bank-full stage (12.19m PWD) to be about  $4813 \text{ m}^3/\text{s}$ . Which is slightly higher (only 6.9 % ) than the discharge found earlier. The highest yearly reported discharges (peak discharges) at Gorai-Railway bridge site for the period of 1965-88 as per BWDB records were also noted. There is a concept

that the dominant discharge varies between the ranges of half to two-third of the peak discharge. .

It appears from Table 4.1 that the average value of half of the peak discharges was  $3214 \text{ m}^3/\text{s}$  and an average value of two-third of peak discharges was found to be  $4730 \text{ m}^3/\text{s}$ . An average of them to be 3972 cumecs which is about 62% of the average peak discharge and some what less (by 11.73 %) than the discharge found earlier by Schafernok concetp.

From flow duration curve (developed utilizing the data for the period of 1967 to 1988 for Gorai-Rail Way bridge site) it appears that the discharge  $4500 \text{ m}^3/\text{s}$  equalled or exceeded about 11.25 % of the time and the discharge 3972 cumecs equalled or exceeded approximately 15% of the time. It is appeared that the frequency of occurrence of the representative discharge (D.D.) were appreciably low in the Gorai-Madhumati river which is doing most of the work on the channel, i.e., development of meander, adjustment of meander length, meander belt etc. It is therefore, concluded that the dominant discharge of the Gorai-Madhumati river is about  $4500 \text{ m}^3/\text{s}$ , which is equalled or exceeded approximately 11.25 % of the time. The return period for this discharge is approximately one year. But dominant discharge found for Gorai app.  $4250 \text{ m}^3/\text{s}$  by the almost same procedure for South West area water resources management project study (Halcrow & Partners, 1991).

#### 4.1.2 Comparison with the dominant discharges of other rivers in Bangladesh

Halcrow and Partners (1991) computed the dominant discharge for the Brahmaputra river with the data at Bahadurabad and found the dominant discharge to be  $38,000 \text{ m}^3/\text{s}$  which is little less than the bankfull discharge assumed as  $44,000 \text{ m}^3/\text{s}$ .

Flow duration curve indicates that this discharge is equalled or exceeded 18 percent of the time.

China-Bangladesh joint Expert Team (CBJET), (1991) mentioned in their study report that the dominant discharge (D.D.) for Brahmaputra river is  $37,500 \text{ m}^3/\text{s}$  which is also less than the bankfull discharge.

Benson and Thomas (1966) found that the dominant flow in several streams with sediment transported predominantly as suspended load, had exceedance frequencies in the range of 7.6 to 18.5 percent of the time, and that dominant flow was less than the bankfull flow (after Halcrow & Partners, 1991).

Utilizing the definition for the dominant discharge as the discharge in a river that carries most of the sediment. It was found that the dominant discharge of the upper Meghna river was about  $11,000 \text{ m}^3/\text{s}$ . And the dominant discharge of the lower Meghna river was about  $80,000 \text{ m}^3/\text{s}$ . This slightly lower than the bankfull discharge of  $82,5000 \text{ m}^3/\text{s}$  assumed (Haskoning et al, 1992).

Hossain (1992) computed the dominant discharge for the Ganges and Brahmaputra-Jamuna river using the Schafernok bed generative concept as well as Inglis bankfull and peak flow concepts. On the basis of these study he found an average value of dominant discharge for the Ganges to be about  $30,500 \text{ m}^3/\text{s}$  and for the Brahmaputra-Jamuna river to be about  $36,000 \text{ m}^3/\text{s}$  with the data at Harding bridge and at Bahadurabad ghat respectively. The bed generative discharge at formative stage for the Ganges he found to be as  $28,500 \text{ m}^3/\text{s}$  and an average value of bankfull discharge was found to be about  $30,500 \text{ m}^3/\text{s}$ .

Hossain found an average value of bank-full discharge for the Brahmaputra-Jamuna at Bahadurabad ghat to be approximately  $36,500 \text{ m}^3/\text{s}$ . Whereas schafernok's bed generative concept gave the value of dominant discharge to be  $36,000 \text{ m}^3/\text{s}$ . A rough estimate for the

dominant discharge for the Teesta river was made by Hossain and found the dominant discharge to be approximately  $2267 \text{ m}^3/\text{s}$ .

#### 4.2 The meander parameters ( $M_L$ , $M_B$ , $S_r$ ) and their interrelationships

From the bank line maps (compiled from the spot imageries, landsat images and SOB maps) of the river Gorai-Madhumati for the year 1953, 1973, 1980 and 1990, computations of meander length ( $M_L$ ), meander belt ( $M_B$ ) and sinuosity ( $S_r$ ) of bank lines were made.

In this river the number of complete bends were 10 in 1953 and 13 in 1973, 1980 and 1990. Relationships were established between meander length ( $M_L$ ), and channel width ( $W$ ) for each meander.

From the analysis average relationships were as

$$\begin{aligned} M_L &= 13.60 W && \text{in } 1953 \\ M_L &= 12.87 W && \text{in } 1973 \\ M_L &= 14.32 W && \text{in } 1980 \\ \text{and } M_L &= 13.26 W && \text{in } 1990 \end{aligned} \quad (4.1)$$

Power relationships between meander wave length and channel width were also established and found to be

$$\begin{aligned} M_L &= 14.89 W^{1.106} && \text{for } 1953 \\ M_L &= 5.94 W^{0.164} && \text{for } 1973 \\ M_L &= 8.51 W^{0.51} && \text{for } 1980 \\ \text{and } M_L &= 6.38 W^{0.297} && \text{for } 1990 \end{aligned} \quad (4.2)$$

Which compare quite favourably with the Leopold & Wolman (1960) and Zeller (1967) relationships. General relationships between meander length and channel width for the entire period were also established and found to be



$M_L = 0.33 + 11.90W$  through regression analysis and

$$M_L = 5.613w^{1.10} \text{ with the average value of } M_L \& w \quad (4.3)$$

Effort has also been made to found the relationships between meander wave length and discharge. The relationships (through regression analysis) with the average past annual maximum discharge were as

$$M_L = 191.31 Q_{ma}^{-0.41} \quad (4.4)$$

with Gorai Rly bridge data and

$$M_L = 7.86 Q_{ma}^{0.47} \quad (4.5)$$

with the Kamarkhali transit data. The average relationships have also been established using simple algebraic method between meander wave length (average value) and discharge (average of maximum and dominant discharge). The relationships were as

$$M_L = 36.10 Q_{max}^{0.50} \quad (4.6)$$

$$\text{and } M_L = 43.05 Q_d^{0.50}$$

The units were in F.P.S. system.

Relationships were also developed between meander belt ( $M_B$ ) and channel width ( $W$ ) for the river Gorai-Madhumati, using the meander bends considered. The relationships were

|           |   |         |          |       |
|-----------|---|---------|----------|-------|
| $M_B$     | = | 8.62 W  | for 1953 |       |
| $M_B$     | = | 10.09 W | for 1973 |       |
| $M_B$     | = | 11.75 W | for 1980 |       |
| and $M_B$ | = | 11.68 W | for 1990 | (4.7) |

Meander belt vs.  $Q_{ma}$  (annual maximum discharge) relationships were also established and found to be

$$M_B = 0.244 Q_{ma}^{0.32} \quad (4.8)$$

with Gorai Rly bridge data and

$$M_B = 6.66 Q_{ma}^{-0.057} \quad (4.9)$$

with Kamarkhelli transit data. The average relationships between meander belt (average value) and discharge (average of maximum and dominant discharge) had also been established using simple algebraic operations. The relationships found to be

$$M_B = 27.69 Q_{max}^{0.50} \quad (4.10)$$

and  $M_B = 33.01 Q_d^{0.50}$

The units were in F.F.S. system.

General relationships between meander and channel width were also established and found to be

$$M_B = 0.18 + 12.32w \text{ through regression analysis and} \quad (4.11)$$

$$M_B = 5.07 w^{1.10} \text{ with the average value of } M_B \text{ and } W \text{ through simple algebraic operations.} \quad (4.12)$$

An interesting relationship between average meander wave length and meander belt was observed from the study. The relationships were as follows :

|           |   |         |    |      |        |
|-----------|---|---------|----|------|--------|
| $M_L$     | = | 1.576 W | in | 1953 |        |
| $M_L$     | = | 1.275 W | in | 1973 |        |
| $M_L$     | = | 1.218 W | in | 1980 | (4.13) |
| and $M_L$ | = | 1.135 W | in | 1990 |        |

For the 27 selected bends of Gorai-Madhumati river, the width, the bend migration and the radius of curvatures were obtained from the classified images [plan maps (SOB), spot and landsat imageries (SWMC and SPARRSO)] of the year 1953, 1973, and 1990. These data were plotted in several graphs to investigate the possible relationships. Results are presented in the fig 4.14 to 4.18.

The observed bend migration rates were analyzed within a frame work similar to the previous work by Hanson and Hickin (1984) for meandering river by establishing the relation between the bend migration rates and the relative bend curvature of the channels.

In fig. (4.14) & (4.15) the migration rate i.e., migration per year (m/y) have been plotted against the relative radius of curvature ( $r/w$ ) for the widths ranges from  $w=0-0.35$  Km and  $w=0.36-0.5$  Km.

The movement of the apexes i.e., the bend migration rates were ranges from 5-142 m/year. The maximum bend migration rate observed was 142.6 m/year and the average bend migration rate was about 40 m/year.

In fig(4.16), the bend migration rate (m/y) has been plotted vs. relative bend curvature ( $r/w$ ). This is a general relationship and in fig.(4.17) to (4.19) the relative bend migration (m/w) vs. relative bend curvature ( $r/w$ ) and channel width Vs. channel radius of curvature have been plotted respectively. Upon inspection of the figures the following observations can be made:

(a) There is a negative correlation exists between the relative bend curvature ( $r/w$ ) and the bend migration rate (m/y). Low relative bend curvatures lead to relatively fast migration rates to the certain extent and vice versa. But beyond the certain values of  $r/w$  ( $>3$ ), the migration rates again decreases.

There was a tendency for relatively very sharp bends ( $r/w < 2.5$ ) to have smaller migration rates, as was observed for meandering rivers by Nanson and Hickin (1984). Klaassen and Masselink (1992) found slightly different tendency i.e., increase in relative erosion with the increase in relative curvature to the certain extent in the studies carried out for FAP-21/22 in the braided river Brahmaputra-Jamuna in Bangladesh.

An interesting relationship have been observed in all the figures is that, the bend migration rates gradually increases for  $r/w < 2.5$  and is maximum for  $r/w = 2.5$  (app.) and then gradually decreases for  $r/w > 2.5$ . That is the bend migration rate was maximum for the relative curvature value ( $r/w$ ) of approximately 2.5.

The same relations were also found by Nanson and Hickin (1984) and Klaassen & Masselink (1992). The reason for the maximum bend migration due to erosion at  $r/w = 2.5$  is that for this relative curvature the velocities along the bank (outer) are largest. A further decrease leads to the main flow going through the inner bank, thus reducing the outer bank socur. Klaassen and Masselink (1992) analyzed the data from the Jamuna river and found a similar shape of the bank erosion rates.

b) Klaassen & Masselink (1992) found for the Jamuna river that the relative erosion rates by large channels was relatively small. But in the present study it was observed that there was no definite trend that the bend migration rates for the relatively large channels was small. In a few cases it was observed from figs. 4.14 & 4.15 that the bend migration rates for large channels are relatively more than the narrow channels.

The trend found in the present study that maximum bend migration rates occurred at the relative bend curvature,  $r/w =$  approximately 2.5 which comply with the Bagnold's (1960) inference. He made a theoretical evaluation indicating that an optimum channel curvature should exist with a ratio of bend

radius,  $r$  to channel width,  $w$  between two and three; i.e.  $r/w = 2$  to 3.

An attempt has also been made to relate the bank erosion rates with the stream power of the river, bend characteristics and resistance of the bank following the method proposed by Nanson & Hickin (1984).

For the Gorai-Madhumati river it was found that the value of  $Y_b$ , the bank resistance co-efficient may assumed to be approximately  $45 \text{ N/m}^2$  and  $M_{2.5} = 35 \text{ m/yr}$  to  $53 \text{ m/yr}$ .

For the upper reaches  $M_{2.5} = 35 \text{ m/yr}$  and for the lower reaches  $M_{2.5} = 53 \text{ m/yr}$  yields quite good results.

There some exceptions, as for the 1st & 3rd bend of upper reach the co-efficient,  $M_{2.5}$  should be computed from equations  $M_{2.5} = \Omega / Y_b$  i.e.,  $M_{2.5} = 35$  or  $53 \text{ m/yr}$  no one is valid.

And for the bends of 21, 22, and 27 of lower reach the co-efficient should be  $M_{2.5} = 35 \text{ m/yr}$  instead of  $M_{2.5} = 53 \text{ m/yr}$ .

An interesting relationships can be established between the width of river ( $W$ ) and radius of curvature ( $r$ ) at bends. The relationship has been shown in fig. 4.19 Leopold et al., (1964) proposed the following relationships between the meander length and the channel width and between the meander length and the radius of curvature :

$$M_L = 6.6 W^{0.99}$$

$$M_L = 4.7 r^{0.98}$$

(4.16)

All the dimensions were in meter. From these two equations the meander length ( $M_L$ ) can be eliminated which results

$$W = 0.71 r^{0.99}$$

(4.17)

For the Gorai-madhumati river the width-radius relation was found to be

$$W = 1.22 r^{0.849} \quad (4.18)$$

It appears that in comparison with the Leopold et al., (1964) relationship most channels seem to have too large radius of curvature as shown in fig. 4.19 This may however be due to the fact that here the width during the low flow conditions was used, while it should be the width during bankfull discharge.

River bank erosion is a complex phenomenon in which many factors play a role. The rate of bank erosion is determined by flow, sediment transport, bank properties and water quality (Mosselman, 1992).

The flow in eroding outer bends is affected by the channel characteristics like discharge, dimensions and slope. In bends, also the bend characteristics, in particular the radius of the bend is important, together with the overall planform of the river. In the bend a helical flow pattern is generated that is of great importance for the resulting bed topography. The bank properties include bank material weight and texture, shear strength and cohesive strength, physicochemical properties, bank heights and cross sectional shape, ground water levels and permeability, stratigraphy, tension cracks, ~~vegetation~~ and constructions. Many of these factors are only approximately known. The influence of water quality is assumed to be of minor importance for the rivers in Bangladesh.

#### 4.4 Hydraulic characteristics at a cross-section

##### 4.4.1 At-a-station hydraulic geometry for Gorai- Madhumati river.

At-a-station hydraulic characteristics were studied for the Gorai-Madhumati river at Gorai Railway bridge section and at the

Kamarkhali transit section, utilizing the observed data for the period of 1976 to 1990. The relationships between the width (W), mean depth (d), maximum depth ( $d_{max}$ ), velocity (V) and cross-sectional area (A) with the discharge have been shown in figs.4.20 to 4.24 for Gorai Railway bridge and in figs.4.25 to 4.28 for Kamarkhali transit.

The regression equations for W,d, $d_{max}$ ,V and A had also been derived using power regression technique. For the Gorai Railway bridge section, these were as follows :

$$\begin{aligned}
 W &= 40.65 Q^{0.283} & r &= 0.920 \text{ (for lower part)} \\
 W &= 63.98 Q^{0.24} & r &= 0.910 \text{ (for upper part)} \\
 d &= 0.075 Q^{0.54} & r &= 0.980 \\
 d_{max} &= 0.2438 Q^{0.485} & r &= 0.658 \\
 V &= 0.205 Q^{0.218} & r &= 0.876 \\
 \text{and } A &= 4.85 Q^{0.78} & r &= 0.988
 \end{aligned} \tag{4.19}$$

For Kamarkhali transit, the regression equations were

$$\begin{aligned}
 W &= 125.25 Q^{0.164} & r &= 0.844 \\
 d &= 1.211 Q^{0.206} & r &= 0.770 \\
 V &= 0.0082 Q^{0.63} & r &= 0.983 \\
 \text{and } A &= 132.62 Q^{0.376} & r &= 0.928
 \end{aligned} \tag{4.20}$$

It is found from the above equations that at Gorai railway bridge section discharge was primarily associated with the depth of flow and less effect on velocity. This velocity was average velocity, but it would be the observed velocity. On the other hand at Kamarkhali transit section reverse phenomenon occurred. At this section, discharge was primarily associated with the flow average velocity and less effect on width and depth.

The exponents essentially described both the geometry of the channel and the resistance to erosion. From the results obtained

for Gorai Rly bridge site it revealed that the channel would have a straight steep banks in cohesive materials.

The results of Kamarkhali transit section indicated that some flows were below the threshold of erosion, the channel form was set by the higher discharges, which were capable of eroding bed and banks. The banks were comprised with cohesive material, thus width changes very little with discharge and adjustments occurred in velocity and depth. Discharge were primarily associated with the velocity of flow.

From fig. (4.20) it was appeared that the channel width increased gradually with the increase in discharge upto around 120 cumecs. Beyond this discharge level sharp increase in width was observed and then it follows a gradual increasing trend. This may be due to the change in flow resistance value or due to the change in flow regimes. It may be worth mentioned that the flow Froude number at the Gorai Rly bridge site varies between 0.170 to 0.29. For laboratory situation the lower flow regime Froude number varies between 0.17 to 0.44. But for natural situation to which there more degrees of freedom this range may be different. That is why flow resistance due to change of flow regime may be responsible for such abrupt change in width.

When the flow remains within the main channel, width increases with the increase in discharge and resistance to flow decreases with the ripple bed form upto bankfull level. And when the flow just overtops the banks and starts to flow over the flood plain, resistance to flow increases rapidly as wetted perimeter increases and excess increase in width occurs. After reaching to a certain stage no significant increasing trend in width with the increase in discharge occurs. Gorai Rly bridge site is a restricted or confined section due to protection works, on the other hand Kamarkali ghat section is not so restricted or confined.



#### 4.4.2 Comparison with the other river

Comparison of the value of exponents (b, f & m) obtained by different investigators is shown below :

| Investigator                    | b     | f     | m     |
|---------------------------------|-------|-------|-------|
| Leopold & Maddock (1953)        | 0.26  | 0.40  | 0.34  |
| Leopold, Wolman & Miller (1964) | 0.18  | 0.52  | 0.30  |
| Das (1992)                      | 0.172 | 0.246 | 0.575 |
| Present study                   | 0.24  | 0.54  | 0.218 |

The results obtained in the present study for Gorai-Rly bridge site compare quite favourably with those reported by Leopold and Maddock (1953) as well as by Leopold, Wolman and Miller (1964).

#### 4.5 Specific gauge analysis

When analyzing the medium to long term behavior of a river specific gauge analysis can be used to determine if there any trends with time in the elevation of water surface corresponding to a given discharge. The analysis must be based on sound historical stage-discharge records for a gauging station. In this study, the records for Gorai Rly bridge between 1946/47 and 1989/90 and for Kamarkhali transit between 1965/66 and 1989/90 were used. Gorai Rly bridge is the gauging station on the Gorai-Madhumati river where specific gauge analysis can correctly be performed. Kamarkhali transit is affected by tide so no distinctly representative result is possible from the specific gauge analysis for this station. The rating curves of few years were not available.

The method of analysis was based on the discharges of 500, 1500, 4000 and 6000 cumecs for Gorai Rly bridge and of 500, 1500, 3000,

5000 & 6000 cumecs for Kamarkhali transit and the corresponding water stages from the rating curves for the available years. Water stages were then plotted against the year of observation on an arithmetic plots (figs. 4.29 to 4.32).

There were many limitations in the gauging records but with this stipulation the following inferences can be drawn.

The results suggested that there was a significant trend of water level rising at Gorai Rly bridge at low flow (500 & 1500 cumecs) but the increasing trend was very minor at higher discharges (4000 & 6000 cumecs) but as a whole increasing trend in water level was predominant. From figures, before 1975 the trend of water level rising was not so significant. After 1975, it is appeared that there is a clear tendency to increase in water level in higher discharge and more distinct trend occurred in low flow (500 & 1500 cumecs). It may be due to excessive siltation during low flow which causes the mean bed level to rise. A similar study was carried out for Kamarkhali transit and the results have been shown in figs. 4.31 to 4.32 which indicates that there approximately similar trend in the change in water level for a given discharge and therefore probably there were no sustained aggradation or degradation of the section before 1975 but after 1975 it indicates clearly the possibility of gradual aggradation. Same result was found in South West area water management project study. They found strong upward trend in stages for low discharges that was not so apparent in high stages (Halcrow & Partners, 1993).

From the analysis, although not exclusive, the river appeared to be in a dynamic equilibrium state. The passage of macro-scale bed forms such as sand waves, and chars can radically altered the resistance characteristics and water surface topography, so altering the stage-discharge relationships.

Any persistent trend in the stage-discharge relationships over the about forty years period of record could be indicative of net aggradation or degradation of the channel. At the present study least square regression study techniques were not used to identify any trend.

When analyzing the records to identify any trend it would be inappropriate to use least square regression techniques because of high degree of noise in the data. Instead, application of a robust assessment of trend utilizing own judgement was undertaken.

#### 4.6 Morphology of the channel cross-sections

Cross sectional maps for the selected 5 sections were prepared from BWDB survey data using MIKE-11, a computer software for mathematical modelling and then superimposed for comparison. The pertinent cross-sectional parameters such as area, hydraulic radius, width etc. were computed for comparison using MIKE-11 which were proved to be much helpful and effective for such computations. Typical cross-sectional maps for the year 1968 to 1989 have been shown in figs. 4.33 & 4.34.

The corresponding cross-sectional area, hydraulic radius and width have been shown in table 4.10 for 30 cross-sections out of total 32 sections (fig.4.35). It may be observed from the figs. (4.33) & (4.34) and table (4.10) that the channel sections were gradually widening with time throughout the reach. But in general, sectional area of the reach gradually decreasing except at the upper most and lower sections. The cross-sectional area of the lower reach were gradually increasing though not exclusive. Very sharp increase in cross-sectional area observed at the sections 29 & 30 near Lohagara. It was also observed that the deepest point (thalweg) moved to and fro within the same section with time, that is, wandering of thalweg occurred.

From table 4.10 it is appeared that the change in hydraulic radius was almost consistent with the change in cross-sectional area. But at a few locations the changing trend in hydraulic radius was not consistent with the change in cross-sectional area as well as channel width. This may-due to change in cross-sectional shape. The cross-sectional map shows that the channel near the off take was very wide (about 524.4m at section #1-1 and more than 1250m at section #3-3). On the other hand the average width of the channel immediately down stream of off take was about 457.31m. Again the channel tends to be widened from section #17, near Kamarkhali. The width increased to approximately 1372m at section #19, down stream of Kamarkhali transit. But at further down the channel was narrow and the average width was approximately 762m. It should be pointed out that the width gradually decreased from upstream to down stream and conversely the depth gradually increased towards the down stream. But it is observed from the plan maps of the river that the extreme down stream i.e., the estuarine part of the river was very wide as well as deep. The change in cross-sectional area from 1968 to 1978 was not so appreciable in most of the locations except at locations #1 & #3, at the upper most reach which is clearly visible from the plot and also from the values as shown in table 4.10. The bed levels of the upper most reach (near offtake) gradually rose from 1976 but there was no definite trend of rising or lowering of bed levels at the lower reach. The bed level changing trend found completely erratic in nature. Alluvial rivers show certain consistencies in terms of flow and cross-sectional morphology. In accommodating a change in discharge rivers can change the width, depth, slope and cross-sectional area. These changes appeared to be as uniformly distributed among these components as is permitted by the river mechanics.

#### 4.7 Rivers bed slope, valley slope, sinuosity and channel longitudinal bed profiles.

The longitudinal bed profile of the Gorai-Madhumati river in the study reach had been investigated using the surveyed cross-sectional data of BWDB and BIWTA.

The cross-sectional data were available, for the years 1968 to 1990. From the cross-sectional data the deepest points (thalweg) first investigated and then plotted against the distance (chainage) from upstream to the down stream direction.

In figs. 4.36 & 4.37 the plotted bed profiles (thalwegs) for different years have been shown. Upon investigating the longitudinal bed profiles figs. 4.36 & 4.37, an approximate average bed slope had been estimated and it was  $S_{av} = 4.665 \times 10^{-5}$  m/m.

At the upper reach, the average bed slope was found to be  $6.16 \times 10^{-5}$  m/m and at lower reach the average bed slope was found to be  $3.2 \times 10^{-5}$  m/m. The maximum water surface slope in 1983 was  $6.76 \times 10^{-5}$  m/m and the average bed slope may be considered as  $S = 4.665 \times 10^{-5}$  m/m. From figs. 4.36 & 4.37 it also appeared that the bed level gradually rose at the upper reach, especially near the offtake. But this trend was not so significant at the lower reach. The bed level rising tendency near offtake predominantly occurred from the 1976. Critically reviewing the overall longitudinal bed profiles (fig. 4.36), it was observed that the average shape of the overall longitudinal bed profile was approximately an exponential curve; Which comply with the Haas (1969).

Haas (1969) gave an expression  $S = 0.0020 \exp(-5.8 \times 10^{-3} x)$  in metric units for the Rio Grande river.

Logically, since  $S = -dz_b/dx$ , the bed level may in general show an exponential curve. Hack (1957) showed that this shape frequently

exists. It is clear that these exponential functions can only give a very general idea of the longitudinal bed profile. The general curve is disturbed by the presence of bends. Hence the longitudinal slope of the thalweg will never completely bellow a simple functional relationship.

The bank line elevation along the study reach has been shown in fig. 4.38 and from this figure the valley slope had been approximated and found to  $S_{valley} = 5.48 \times 10^{-5}$  m/m. Valley slope had also been approximated from contour map of Bangladesh as shown in fig. 4.39 and found to be  $S_{valley} = 7.47 \times 10^{-5}$

Ratio of Valley slope and bed slope

$$\frac{S_{valley}}{S_{bed}} = \frac{7.47 \times 10^{-5}}{4.665 \times 10^{-5}} = 1.6012 \quad (4.21)$$

$\therefore$  Ratio = 1.6012 > 1.2 or even 1.50  
i.e., the Gorai-Madhumati is a highly meandering river.

Maximum observed discharge in Gorai-Madhumati river was 8490 cumecs in 1988 and the dominant discharge computed in the present study was 4500 cumecs.

Therefore, slope discharge relationship (Lane, 1957) with dominant discharge (4500 cumecs) was  $SQ^{1/4} = 4.665 \times 10^{-5} \times (4500 \times 35.28)^{1/4} = 0.000931 < 0.0017$

That is, the river with dominant discharge = 4500 cumecs (158760 cusecs) and with bed slope  $= 4.665 \times 10^{-5}$  falls under the condition for meandering stream (Lane, 1957).

As per the Leopold & Wolman (1957) predictor, the river Gorai-Madhumati ( $SQ^{0.44} = 4.665 \times 10^{-5} \times (4500 \times 35.28)^{0.44} = 0.00906 < 0.06$ ) is clearly a mandering river. The Ganges ( $SQ^{0.44} = 3.769 \times 10^{-5} \times 15,24,096^{0.44} = 0.0198 < 0.06$ ) and the river Brahmaputra-Jamuna ( $SQ^{0.44} = 5.2 \times 10^{-5} \times 23,99,040^{0.44} = 0.0333 < 0.06$ ) are the meandering rivers (Hossain, 1989 & BRTS, 1991).

Average sinuosity of the Gorai-madhumati river was found to be

$$\begin{aligned} & \text{Sr} = 1.845 \quad \text{in} \quad 1953 \\ & \text{Sr} = 1.684 \quad \text{in} \quad 1973 \\ & \text{and Sr} = 1.837 \quad \text{in} \quad 1990 \end{aligned} \quad (4.23)$$

Sinuosity at the upper reach was always less than that at the lower reach. For example in 1953, the sinuosity at the upper reach was 1.69 and at the lower reach was 2.00. In 1990, the sinuosity at the upper reach was 1.695 and at the lower reach was 1.979. As the sinuosities found always more than 1.5 that is the river may be called as meandering river.

#### 4.8 Flow Characteristics of the Gorai-Madhumati river

The Gorai-Madhumati river takes off from the Ganges near Talbaria, Kushtia and flows towards South-Easterly direction in the different names and finally falls into the Bay of Bengal through Haringhata estuary. Gauge records and discharge measurements were initiated prior to 1946. There are many water level gauging stations along the river such as Talbaria, Gorai Rail Way bridge, Kamarkhali transit, Bardia, Pirojpur etc. Discharges are measured at Gorai Rail Way bridge and at Kamarkhali transit site. Three typical observed discharge hydrographs for the years 1967, 1974 and 1987 for the stations Gorai Rly bridge and Kamarkhali transit have been shown in figs. 4.40 to 4.42 which were constructed utilizing the hydrological data collected by Surface Water Hydrology, BWDB. Reviewing the available hydro-metrological records, it was concluded that the recorded highest discharge at Gorai Rly bridge is 8490 cumecs being recorded in September 3, 1988. The corresponding water level was 13.638 m PWD. Average annual discharge at Gorai Rly bridge and at Kamarkhali transit were approximately 1409 cumecs and 1255 cumecs respectively. The average annual minimum discharge at Kamarkhali before 1975 was 108.8 cumecs and after 1975 it becomes statistically 28.83 cumecs. But if we exclude

certain exceptional years then the average minimum will be very negligible (fraction of cumecs).

The discharge hydrographs, as shown in figs. 4.40 to 4.42 quite commonly displays two or three major flood peaks. The first flood peak generally occurs during early August and is characterized by an extremely rapid increase in discharge. Within a few days the discharge may increase from around 2000 cumecs to more than 6500 cumecs. After this rapid initial rise, the river level generally falls slightly, and then in the latter part of August or in the first part of September the major flood peak occurs. This flood period will generally lasts for nearly a month, then the river will again drop slightly. During the last part of September, a third flood peak may occur.

It is appeared from the flood hydrographs for Gorai Railway bridge and Kamarkhali transit for the same year, the discharges at Kamarkhali observed always less than that at Gorai Railway bridge site. The flood peak at Gorai Railway bridge should appear first then at Kamarkhali transit, because the Gorai Railway bridge is at the upstream. From fig. 4.42 it is appeared that flood peak occurred at the Gorai Railway bridge on 23-09-67 and at Kamarkhali transit on 19-09-67 which is quite strange. From figs. 4.40 & 4.41 it also appeared that the flood peaks at Gorai Railway bridge occurred on 19-09-87 and 14-08-74, on the other hand the peaks occurred at Kamarkhali on 20-08-87 and 13-08-74.

That is, the peaks always occurred at Kamarkhali first then at Gorai Rly bridge. It is customary that the discharge gradually increased at the down stream and the flood peaks gradually moved from upstream to down stream. But in this river opposite trend occurred. The possible explanation of this is that, the Gorai-Madhumati is a tidal river, especially its down stream part is highly affected by tide (up to Kamarkhali) and hence reduced the discharge at the down stream and changed the usual flood peak period.



#### 4.9 Planform changes of the Gorai-Madhumati river

Planform of the river studied with the help of maps of different years. Maps of different years were superimposed and then compared. The total length of the river Talbaria to Patgati was found about 258.5 km (161.5 miles). The catchment area above the Bardia was approximately 2443 sq. kms. It appeared from the old maps (1771, 1918, etc.) that the river meanders extensively from the early stage. The down stream part of the river was more meandering because the sediment transported from the upper reaches, especially the bed load during high floods, was deposited in various parts of the channel. Consequently the flow was deflected and heavily eroded the loose banks. The mean particles size,  $d_{50}$  of bed load was 0.13 mm. The river flows in its valley which is about 10 to 25 km in width. Above the confluence with Nabaganga (near Bardia) which joins the river through a Halifax cut, the Gorai-Madhumati flows for about 198 km. Where the average bed and valley slopes were approximately  $1.43 \times 10^{-4}$  and  $5.48 \times 10^{-5}$  respectively gradually declining in the down stream direction. According to the maps available since 1953, the meanders have shifted covering a 4 to 5 km wide belt mainly to the right hand side of the valley. The well developed meanders have an average meander length of 5 to 6 km, and an average bend radius of 0.8 to 0.9 km. The channel is generally 0.36 to 0.4 km wide and 6 to 8 m deep in the crossings, and 18 to 21 m deep in the apexes of the bends (pool). One important railway and a highway bridge are situated over the river at Ch. 11.8 km (Gorai Railway bridge) and 86 km (Kamarkhali bridge) respectively. The bridges fixing the two points of the river, cause definite change in the meander patterns. From field visit it was observed that in order to protect the Kushtia town from its severe bank erosion, the river along some 3 km of its length was partially trained by two T-head groynes. And to protect the newly constructed highway bridge at Kamarkhali (81.5 km down from Kushtia town) attempt has been made to protect the right bank of the river (upstream from the bridge site) from severe erosion

with the help of a series of permeable groynes as well as revetment with geotextiles as filter materials. On the other hand, at the Gorai Railway bridge site (7 km down from the Kushtia town) severe erosion occurs at the right bank which threatened the bridge from the possibility of out-flanking. The reach of the river above Gorai Railway bridge is now situated on the (very close) moderate bend. The slope of the valley as well as river bed, upstream of the bridge was fairly steep. So surface runoff drains the river immediately at the upstream and there exists appreciable flow velocity near the bridge during monsoon. Which causes the severe bank erosion (right bank). There is a tendency to shift the river towards the right valley.

### **Patterns of meander development**

Using the maps (old plan maps, Landsat images, spot imageries) of the Gorai-Madhumati river in 1918, 1953, 1973 and 1990, various patterns and phases of meander development including natural cut-offs can be investigated. A special case of meander development can be seen in fig. 4.48 which is at the immediate down stream of Kamarkhali transit. The meander of more than 14 km in length is fully over developed yet unchanged for several decades (since 1918), may be for some local as well as geological reasons. The neck had been protected as there were two Ferry ghats at both sides of the neck (namely Kamarkhali Ferry ghat and Salamatpur Ferry ghat). This situation is a counter example to the case of river training with artificial cut-offs. Without the bank protection or if there were no geological constraint, the neck should have been cut-off long ago causing more than 14 km local shortening of the river. The meanders ( $M_{a1}$ ,  $M_{a2}$ ,  $M_{a3}$ ,  $M_{a4}$  and  $M_{a5}$ ) in figs. 4.49 & 4.50 shows the examples of accelerated meander development. The apices of the first two and fourth bends ( $M_{a1}$ ,  $M_{a2}$  and  $M_{a4}$ ) moved towards the right valley some 1.08 km, 1km and 0.7km during 1st 20 years, then 1.19 km, 1 km and 1.09 km during next 17 years respectively. The apex of the third bend ( $M_{a3}$ ) moved towards the left valley some 0.9 km during 1st 20

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years, then 1.30 km during last 17 years respectively. Near Bardia, immediate down stream of the confluence with Nabaganga, a meander  $M_{25}$  is developing with high acceleration. The apex of the bend moved North-Eastwards some 200 m during 1st 20 years, then 800 during last 17 years, which is now nearly a fully developed meander. There are many locations (at least 5) where natural cut-offs occurred at different times. Near Boalmari, Atiadanga and at the down of Bhatiapara natural cut-offs developed between 1953 and 1973, 1918 and 1953 and close to the time 1973 respectively. Cut-offs at the upstream of Baridia and near Patgati developed between 1918 and 1953. From figures it is appeared that the over development of the bends and most of the cut-offs occurred between the period 1953 to 1973. In fig. 4.46 superimposed maps of the years 1771, 1863, 1944 and 1963 for a short reach of Madhumati river (extreme down stream part) have been shown. From this figure it is appeared that the Barni cut-off at the down of Gopalgonj developed through the neck in the time close to 1918 and subsequently, the shortening caused secondary bank erosion to some 4 miles in the down stream direction consequently a small meander developed at faster rate and a small cut-off occurred on the opposite side of the river between 1953 and 1973. Now the abandoned part of the river formed the ox-bow lake. There are few more ox-bow lakes appeared at the upstream part of the river. In figs. 4.46 to 4.51 thirteen developing apexes can be seen. These bends are migrating laterally as well as towards the down stream direction i.e., skew meanders occurred. At some locations once the cut-off formed on the overdeveloped meanders between 1953 and 1973, again the river starts bank erosion and new meander developed.

Concerning meander development on the Gorai-Madhumati river a number of practical conclusions can be drawn. Firstly, the river reaches are not appreciably affected by the river training. The river is meandering freely and in a fully meandering state. The movement of the apices, i.e., the bend migration rates, is about 5-142 m/year. The average bend migration rate is about 40m/year

and the maximum is about 142.6 m/year. Secondly, meander development finally leads to natural cut-offs, occurred usually during floods. The neck width may not be narrower than the channel width, through which the cut-offs formed. After the primary cut-off, the meander to be abandoned can develop further (fig. 4.56) until the new channel achieves a sufficient width and then down stream bank erosion causes a new meander to develop. Thirdly, due to human activity and geological constraints, the meander pattern and development are rather irregular on most reaches and some reaches remaining unchanged for long times.

### **Cut-offs.**

Using maps of the river from 1863, 1918, 1953, 1963 (partial), 1973 and 1990, the formation of cut-offs can be predicated. A big natural cut-off formed between 1863 and 1918 and four small cut-offs occurred between 1953 and 1973. In the absence of river training interferences and geological constraints, the development of further cut-offs can be predicted. In fig. 4.48 the neck of the two consecutive meanders at Kamarkhali is as narrow as the channel width, therefore, the formation of a natural cut-off can be expected. From figs. 4.49 & 4.50 it is appeared that there are seven locations where the meanders are developing at a faster rates and hence cut-offs can be formed in near future. A sharply developing meander bend is observed from fig. 4.50 near the confluence with Nabaganga. The bend is migrating at the rate of 45 m/year.

### **Length-changes and equilibrium length**

Using the river maps of the different years (1953 to 1990) the length changes of the channel were determined between successive fixed points. The data are summarized in table 4.11. It can be seen that reasonable shortening of length (about 13 km) occurred on the subreaches between Bhatiapara to Bardia in the time between 1953 to 1973 where two cut-offs took place between this

period. Another two subreaches between Kamarkhali to the point (23°-15', 89°-40') and offtake of Atharabanki to bifurcation point (down stream of Patgati) shortened the lengths by 2 km and 2.25 km in the time between 1953 to 1973 respectively. Where two small cut-offs occurred. It is observed from table that the value of total shortening (11.6 km) differs from the value of cumulative shortening ( $13+2+2.25 = 17.25$  km) of different reaches. The difference means that on all subreaches the length increase caused by meander development which is generally the dominant factor. The overall length shortening on the 258.35 km long Gorai-Madhumati river was 11.6 km during 20 years. This shortening is equivalent to 44.87 m/km and 580 m/year. The length of the channel increased further after 1973 and it was increased to 258.5 km from 246.75 km between 1973 to 1990. That is the change in length of the river in long term (1953 to 1990) is equal to only 0.15 km (increase). That is the resultant length changes would be practically zero. Therefore the length of the river seemed to be in a dynamic equilibrium state as a result of the long term general increase and the local shortenings. The major length changes occurred shown in table 4.11 naturally on those subreaches where the river was less trained or not trained at all. Practically no resultant length changes occurred on the subreaches between Talbaria to Gorai Railway bridge and Gorai Rly bridge to the point (23°-45', 89°-15'). Minor changes in length occurred in the subreaches between the point (23°-45', 89°-15') to Kamarkhali and Kamarkhali to the point (23°-15', 89°-40'). Most of the length changes occurred on the down stream part of the river. These results are important in planning a proper horizontal plan and length for the particular river reach to be trained. The practical river training design requires a great deal of experience and knowledge of the behavior of the river in question. Rivers are perhaps the most difficult systems of nature within the natural environment. General relationships which have been derived have to be checked against every river.

## CHAPTER V

### CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

#### 5.1 Conclusion

Based on the present study, the following conclusions can be drawn :

1. The dominant discharge of the Gorai-Madhumati river was 4500 cumecs and bankfull discharge was 4813 cumecs.
2. Meander wave length and meander belt was strongly correlated with the average channel width in few cases. There also a good correlation between dominant discharge and meander parameters. It was also observed that the ratio of average meander wave length to average meander belt was gradually decreasing with time. The Gorai-Madhumati has a tendency to reduce her sinuosity in the short term. But in long term no appreciable change in sinuosity observed.
3. The bends were migrating in an unpredictable manner. The migration rates were ranges from 5-142 m/year and the average migration rate was about 40 m/year. The bend migration rate depends on the relative bend curvature ( $r/w$ ). The maximum migration rate occurred with the relative bend curvature value  $r/w = 2.5$ .
4. Water level of the river had observed risen especially for low discharge. The rising tendency before 1975 was not so significant, but after 1975, the rising tendency was clear. It may be due to excessive siltation during low flow which caused the mean bed level to rose.

5. The bed levels of the river at the upper most reach were gradually rising up from 1976 but there was no definite trend of rising or lowering of bed levels at the lower reach. The bed level changing trend was completely erratic in nature.
6. The average bed slope of the river were gradually decreasing along the down stream direction. The average bed and valley slope of the river were  $4.664 \times 10^{-5}$  and  $7.47 \times 10^{-5}$  respectively. From which it may be concluded that the Gorai-Madhumati is a highly meandering river. The overall sinuosity of the river was found to be in the range of 1.684 to 1.845 which is greater than 1.5 which indicates clearly a meandering river. Sinuosity at the lower reach was found more than that at upper reach i.e., lower part is more meandering as observed also from maps.
7. The Gorai-Madhumati is a highly meandering river from its early stage. Its down stream part is more meandering than its upstream part. There are certain reaches in this river which is remaining unchanged for long time due to human intervention or geological constraints. Between the time period of 1953 to 1973 several natural cut-offs formed along the river reach. The total length of the river in the long term remain unchanged. So the river is in dynamic equilibrium state.

## 5.2 Limitations of the present study

There were many limitations in the present study. The quality of the landsat maps were not so good and detailed old maps (continuous, at least in two years interval) were not available. The cross-section data also were not exclusive. The main limitation was the time constraints. To evaluate the bend characteristics exhaustively detailed field investigation was of utmost importance. But there were little opportunity of field investigations.

### 5.3 Recommendations for the future study

In view of the limitations cited in the previous articles, the following recommendations were made for future study :

1. More good quality maps (preferably spot imageries) should be collected with the close time intervals to study the formation of cut-offs and direction and phases of meander development.
2. Frequent and detailed field investigations are required to study the bend characteristics accurately and exhaustively.
3. Correct and detail cross-sectional data should be collected with regular intervals to investigate the river bed morphology.
4. More (at least for 15 stations) hydro-metrological data (water level and discharge) should be collected from down stream reach to investigate the down stream hydraulic geometry.
5. Good quality water level, discharge and sediment data should be collected to investigate the water level rising / falling trend and to compute the dominant discharge more accurately.



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# TABLE

Table 4.1 Discharges and their components for Gorai-Madhumati river at Gorai Rly bridge station between 1965 and 1988.  
Bankfull level considered is 40 ft or 12.19 m (PWD).

| Year | $Q_{24h}$<br>m <sup>3</sup> /s | $Q_{75\%}$ of $Q_{24h}$<br>m <sup>3</sup> /s | $Q_{50\%}$ of $Q_{24h}$<br>m <sup>3</sup> /s | $Q_{25\%}$ of $Q_{24h}$<br>m <sup>3</sup> /s | Bankfull discharge from<br>rating curve (m <sup>3</sup> /s) |
|------|--------------------------------|----------------------------------------------|----------------------------------------------|----------------------------------------------|-------------------------------------------------------------|
| 1965 | 4770                           | 3577                                         | 2385                                         | 2862                                         | 4100                                                        |
| 1966 | 5850                           | 4387                                         | 2925                                         | 3510                                         | 4590                                                        |
| 1967 | 6880                           | 5160                                         | 3440                                         | 4128                                         | 4800                                                        |
| 1968 | 6280                           | 4710                                         | 3140                                         | 3768                                         | 4900                                                        |
| 1969 | 7560                           | 5670                                         | 3780                                         | 4536                                         | 5050                                                        |
| 1970 | 5070                           | 3802                                         | 2535                                         | 3042                                         | 4530                                                        |
| 1971 |                                |                                              |                                              |                                              |                                                             |
| 1972 | 5040                           | 3780                                         | 2520                                         | 3024                                         | 5040                                                        |
| 1973 | 7190                           | 5392                                         | 3595                                         | 4314                                         | 5900                                                        |
| 1974 | 8460                           | 6345                                         | 4230                                         | 5076                                         | 6000                                                        |
| 1975 | 5970                           | 4477                                         | 2985                                         | 3582                                         | 5090                                                        |
| 1976 | 6620                           | 4965                                         | 3310                                         | 3972                                         | 5020                                                        |
| 1977 | 5890                           | 4417                                         | 2945                                         | 3534                                         | 4950                                                        |
| 1978 | 6960                           | 5220                                         | 3480                                         | 4176                                         | 4650                                                        |
| 1979 | 4250                           | 3187                                         | 2125                                         | 2550                                         | 4250                                                        |
| 1980 | 6570                           | 4927                                         | 3285                                         | 3942                                         | 4500                                                        |
| 1981 | 6630                           | 4972                                         | 3315                                         | 3978                                         | 5390                                                        |
| 1982 | 6810                           | 5907                                         | 3405                                         | 4086                                         | 4900                                                        |
| 1983 | 7480                           | 5610                                         | 3740                                         | 4488                                         | 4500                                                        |
| 1984 | 6050                           | 4537                                         | 3025                                         | 3630                                         | 4100                                                        |
| 1985 | 6290                           | 4717                                         | 3145                                         | 3774                                         | 4950                                                        |
| 1986 | 5240                           | 3930                                         | 2620                                         | 3144                                         | 4450                                                        |
| 1987 | 7500                           | 5626                                         | 3750                                         | 4500                                         | 4650                                                        |
| 1988 | 8490                           | 6367                                         | 4245                                         | 5094                                         | 4400                                                        |
| Avg. | 6428                           | 4821                                         | 3214                                         | 3857                                         | 4813                                                        |

Table 4.2 sediment load (observed) for different discharge classes  
as 400-600,800-1000,1400-1600,1800-2000 etc.

| Discharge, Q<br>(Hundred, $M^3/S$ ) | Frequency, f<br>(Days) | Observed load, Qst<br>(Tons/day) | Total observed load<br>Qst * f (Tons) |
|-------------------------------------|------------------------|----------------------------------|---------------------------------------|
| 4-6                                 | 631                    | 4294                             | 2709514                               |
| 8-10                                | 269                    | 10006                            | 2691614                               |
| 14-16                               | 138                    | 20869                            | 2879922                               |
| 18-20                               | 86                     | 29325                            | 2521950                               |
| 24-26                               | 108                    | 43526                            | 4700808                               |
| 28-30                               | 91                     | 53890                            | 4903990                               |
| 34-36                               | 94                     | 70636                            | 6639784                               |
| 38-40                               | 111                    | 82539                            | 9161829                               |
| 44-46                               | 103                    | 101412                           | 10445436                              |
| 48-50                               | 88                     | 114633                           | 10087704                              |
| 54-56                               | 72                     | 135362                           | 9746064                               |
| 58-60                               | 61                     | 149752                           | 9134872                               |
| 64-66                               | 41                     | 172147                           | 7058027                               |
| 68-70                               | 23                     | 187595                           | 4314685                               |
| 74-76                               | 16                     | 211509                           | 3384144                               |

Table 4.3 Sediment load (calculated bed load) for discharge classes as 500-1000, 1000-1500, 1500-2000 etc. with  $d_{50} = 0.13 \text{ mm}$

| Discharge, Q<br>(Hundred, $\text{M}^3/\text{S}$ ) | Frequency, f<br>(Days) | Bed load, $Q_{sb}$<br>(Tons/day) | Total bed load<br>$Q_{sb} \times f$<br>(Tons) |
|---------------------------------------------------|------------------------|----------------------------------|-----------------------------------------------|
| 00-05                                             | 4685                   | 435                              | 2037975                                       |
| 05-10                                             | 792                    | 2991                             | 2368872                                       |
| 10-15                                             | 421                    | 7328                             | 3085088                                       |
| 15-20                                             | 272                    | 13224                            | 3596928                                       |
| 20-25                                             | 278                    | 20552                            | 5713456                                       |
| 25-30                                             | 247                    | 29225                            | 7218575                                       |
| 30-35                                             | 216                    | 39178                            | 8462448                                       |
| 35-40                                             | 277                    | 50359                            | 13949443                                      |
| 40-45                                             | 316                    | 62725                            | 19821100                                      |
| 45-50                                             | 253                    | 76241                            | 19288973                                      |
| 50-55                                             | 218                    | 90875                            | 19810750                                      |
| 55-60                                             | 164                    | 106600                           | 17482400                                      |
| 60-65                                             | 104                    | 123392                           | 12832768                                      |
| 65-70                                             | 80                     | 141229                           | 11298320                                      |
| 70-75                                             | 41                     | 160093                           | 6563813                                       |
| 75-80                                             | 15                     | 179964                           | 2699460                                       |
| 80-85                                             | 14                     | 200827                           | 2811578                                       |

Table 4.4 Sediment load (calculated total load) for discharge classes as 500-1000, 1000-1500, 1500-2000 etc. with  $d_{50} = 0.048$  mm

| Discharge                | Frequency   | Sediment load     | Total sediment load |
|--------------------------|-------------|-------------------|---------------------|
| Q<br>(Hundred, $M^3/S$ ) | f<br>(Days) | Qst<br>(Tons/day) | Qst x f<br>(Tons)   |
| 00-05                    | 4685        | 1144              | 5359640             |
| 05-10                    | 792         | 7480              | 5924160             |
| 10-15                    | 421         | 17909             | 7539689             |
| 15-20                    | 272         | 31829             | 8657488             |
| 20-25                    | 278         | 48904             | 13595312            |
| 25-30                    | 247         | 68911             | 17021017            |
| 30-35                    | 216         | 91681             | 19803096            |
| 35-40                    | 277         | 117082            | 32431714            |
| 40-45                    | 316         | 145006            | 45821896            |
| 45-50                    | 353         | 175363            | 61903139            |
| 50-55                    | 218         | 208076            | 45360568            |
| 55-60                    | 164         | 243076            | 39864464            |
| 60-65                    | 104         | 280304            | 29151616            |
| 65-70                    | 80          | 319706            | 25576480            |
| 70-75                    | 41          | 361233            | 14810553            |
| 75-80                    | 15          | 404843            | 6072645             |
| 80-85                    | 14          | 450495            | 6306930             |

Table 4.5 Sediment load (calculated total load) for different discharge classes as 400-600, 800-1000, 1400-1600, 1800-2000 etc. with  $d_{50} = 0.048$  mm

| Discharge                | Frequency   | Sediment load     | Total sediment load |
|--------------------------|-------------|-------------------|---------------------|
| Q<br>(Hundred, $M^3/S$ ) | f<br>(Days) | Qst<br>(Tons/day) | Qst x f<br>(Tons)   |
| 4-6                      | 631         | 3741              | 2360571             |
| 8-10                     | 269         | 10215             | 2747835             |
| 14-16                    | 138         | 24457             | 3375066             |
| 18-20                    | 86          | 36632             | 3150352             |
| 24-26                    | 108         | 58553             | 6323724             |
| 28-30                    | 91          | 75458             | 6866678             |
| 34-36                    | 94          | 104059            | 9781546             |
| 38-40                    | 111         | 125198            | 13896978            |
| 44-46                    | 103         | 159886            | 16468258            |
| 48-50                    | 88          | 184933            | 16274104            |
| 54-56                    | 72          | 225294            | 16221168            |
| 58-60                    | 61          | 254013            | 15494793            |
| 64-66                    | 41          | 299736            | 12289176            |
| 68-70                    | 23          | 331943            | 7634689             |
| 74-76                    | 16          | 382781            | 6124496             |

Table 4.6 Meander Parameters, 1953

| SL No | Meander Wave length, $M_T$ , in km. | Meander belt, $M_R$ , in km | Channel width, $W$ in km. |
|-------|-------------------------------------|-----------------------------|---------------------------|
| 1.    | 3.725                               | 1.65                        | 0.38                      |
| 2.    | 6.00                                | 3.60                        | 0.49                      |
| 3.    | 8.90                                | 5.60                        | 0.47                      |
| 4.    | 10.50                               | 8.15                        | 0.495                     |
| 5.    | 6.30                                | 3.85                        | 0.337                     |
| 6.    | 5.00                                | 4.15                        | 0.445                     |
| 7.    | 3.30                                | 2.10                        | 0.362                     |
| 8.    | 5.30                                | 1.60                        | 0.321                     |
| 9.    | 7.00                                | 2.60                        | 0.375                     |
| 10.   | 3.70                                | 3.05                        | 0.362                     |
| 11.   | 4.00                                | 2.70                        | 0.437                     |
| 12.   | 1.60                                | 2.35                        | 0.337                     |

Table 4.7 Meander Parameters, 1973

| SL No | Meander Wave length, $M_T$ , in km. | Meander belt, $M_R$ , in km | Channel width, $W$ in km. |
|-------|-------------------------------------|-----------------------------|---------------------------|
| 1.    | 7.75                                | 4.625                       | 0.40                      |
| 2.    | 6.25                                | 3.45                        | 0.40                      |
| 3.    | 3.40                                | 1.90                        | 0.325                     |
| 4.    | 6.075                               | 3.829                       | 0.390                     |
| 5.    | 5.85                                | 5.225                       | 0.435                     |
| 6.    | 5.25                                | 3.575                       | 0.48                      |
| 7.    | 4.45                                | 6.70                        | 0.58                      |
| 8.    | 4.025                               | 4.125                       | 0.45                      |
| 9.    | 4.20                                | 4.375                       | 0.393                     |
| 10.   | 7.125                               | 7.65                        | 0.415                     |
| 11.   | 4.125                               | 2.575                       | 0.356                     |
| 12.   | 6.40                                | 2.925                       | 0.341                     |
| 13.   | 3.85                                | 2.920                       | 0.366                     |



Table 4.8 Meander Parameters, 1980

| SL No | Meander Wave length, $M_T$ in km | Meander belt, $M_B$ in km | Channel width, $W$ in km |
|-------|----------------------------------|---------------------------|--------------------------|
| 1.    | 7.85                             | 4.50                      | 0.41                     |
| 2.    | 6.25                             | 3.40                      | 0.38                     |
| 3.    | 3.20                             | 1.75                      | 0.340                    |
| 4.    | 5.86                             | 3.80                      | 0.36                     |
| 5.    | 5.62                             | 5.20                      | 0.44                     |
| 6.    | 4.75                             | 4.50                      | 0.262                    |
| 7.    | 4.25                             | 5.90                      | 0.412                    |
| 8.    | 4.25                             | 5.75                      | 0.406                    |
| 9.    | 4.00                             | 4.91                      | 0.380                    |
| 10.   | 6.40                             | 8.15                      | 0.425                    |
| 11.   | 5.20                             | 2.10                      | 0.337                    |
| 12.   | 6.25                             | 2.89                      | 0.30                     |
| 13.   | 3.64                             | 2.95                      | 0.295                    |

Table 4.9 Meander Parameters, 1990

| SL No | Meander Wave length, $M_T$ in km | Meander belt, $M_B$ in km | Channel width, $W$ in km |
|-------|----------------------------------|---------------------------|--------------------------|
| 1.    | 8.00                             | 4.50                      | 0.425                    |
| 2.    | 6.25                             | 3.40                      | 0.370                    |
| 3.    | 3.10                             | 1.625                     | 0.341                    |
| 4.    | 5.775                            | 3.725                     | 0.340                    |
| 5.    | 5.45                             | 5.10                      | 0.44                     |
| 6.    | 3.85                             | 4.57                      | 0.475                    |
| 7.    | 4.20                             | 6.40                      | 0.44                     |
| 8.    | 3.25                             | 5.00                      | 0.431                    |
| 9.    | 3.75                             | 5.20                      | 0.347                    |
| 10.   | 6.975                            | 8.875                     | 0.560                    |
| 11.   | 4.50                             | 2.475                     | 0.218                    |
| 12.   | 6.175                            | 2.875                     | 0.245                    |
| 13.   | 3.00                             | 2.92                      | 0.216                    |

**Table 4.10 Cross-sectional parameters of Gorai-Madhumati river**

| Cross-section no. | Cross-sectional area (Sq.m) | Hydraulic radius (m) | Width (m) | Year |
|-------------------|-----------------------------|----------------------|-----------|------|
| GM # 1            | 3606                        | 5.22                 | 1375      | 1968 |
| # 1               | 5299                        | 5.82                 | 1524.39   | 1976 |
| # 1               | 5617                        | 5.26                 | 2219.81   | 1982 |
| # 3               | 6793                        | 8.80                 | 1283      | 1968 |
| # 3               | 4788                        | 5.78                 | 1270      | 1976 |
| # 3               | 6706                        | 7.45                 | 1140      | 1982 |
| # 5               | 3414                        | 7.61                 | 637.19    | 1968 |
| # 5               | 3211                        | 7.95                 | 652.43    | 1976 |
| # 13              | 3202                        | 13.61                | 343       | 1968 |
| # 13              | 3284                        | 12.84                | 343       | 1978 |
| # 13              | 3204                        | 13.82                | 359.75    | 1982 |
| # 16              | 3846                        | 12.97                | 419.20    | 1978 |
| # 16              | 4486                        | 12.42                | 488.41    | 1982 |
| # 16              | 3933                        | 12.43                | 419.20    | 1983 |
| # 17              | 3855                        | 6.80                 | 704.87    | 1978 |
| # 17              | 3906                        | 7.48                 | 650       | 1982 |
| # 17              | 4019                        | 6.85                 | 704.87    | 1989 |
| # 18              | 3688                        | 6.76                 | 737.80    | 1978 |
| # 18              | 3689                        | 7.00                 | 625       | 1982 |
| # 19              | 4426                        | 5.62                 | 1388.7    | 1978 |
| # 19              | 4326                        | 5.64                 | 1200      | 1982 |
| # 19              | 4163                        | 5.72                 | 1007.62   | 1984 |
| # 19              | 3066                        | 4.76                 | 1326      | 1989 |
| # 24              | 4580                        | 7.29                 | 725.6     | 1978 |
| # 24              | 3816                        | 6.34                 | 839.94    | 1982 |
| # 24              | 3767                        | 6.48                 | 777.43    | 1984 |
| # 24              | 5104                        | 7.36                 | 792.68    | 1989 |
| # 28              | 3541                        | 7.48                 | 746.95    | 1978 |
| # 28              | 3939                        | 7.35                 | 769.81    | 1982 |

to be continued

| Cross-section no. | Cross-sectional area (Sq.m) | Hydraulic radius (m) | Width (m) | Year |
|-------------------|-----------------------------|----------------------|-----------|------|
| # 28              | 4037                        | 7.32                 | 628.04    | 1984 |
| # 28              | 4572                        | 7.08                 | 769.81    | 1989 |
| # 29              | 3091                        | 5.17                 | 685.91    | 1978 |
| # 29              | 3936                        | 6.60                 | 789.93    | 1982 |
| # 29              | 6040                        | 7.96                 | 946.34    | 1989 |
| # 30              | 3033                        | 7.12                 | 1105.18   | 1978 |
| # 30              | 7181                        | 7.49                 | 1044.81   | 1982 |
| # 30              | 7607                        | 7.21                 | 1105.18   | 1989 |

Table 4.11 lengths of the Gorai-Madhumati river (reach to reach) in the period between 1953 to 1990.

| Fixed points down stream from the<br>offtake                    | Lengths in<br>1953 (km) | Lengths in<br>1973 (km) | Lengths in<br>1990 (km) |
|-----------------------------------------------------------------|-------------------------|-------------------------|-------------------------|
| Offtake (Talbaria) to Kushtia town                              | 4.80                    | 4.80                    | 4.80                    |
| Kushtia town to Gorai Rly bridge                                | 7.00                    | 7.00                    | 7.00                    |
| Gorai Rly bridge to the point<br>(23°-45', 89°-15')             | 24.70                   | 24.70                   | 24.70                   |
| (23°-45', 89°-15') to Kamarkhali                                | 47.50                   | 49.50                   | 49.50                   |
| Kamarkhali to (23°15', 89°40')                                  | 67.50                   | 65.50                   | 66.25                   |
| (23°15', 89°-40') to Bhatiapara                                 | 12.25                   | 13.75                   | 17.25                   |
| Bhatiapara to Bardia                                            | 38.00                   | 25.00                   | 28.50                   |
| Bardia to offtake of Atharabanki<br>(Mollahat)                  | 22.35                   | 24.50                   | 27.50                   |
| Offtake of Atharabanki to<br>bifurcation point (d/s of Patgati) | 34.25                   | 32.00                   | 33.50                   |
| Total length                                                    | 258.35                  | 246.75                  | 258.50                  |

\* Approximate total length of the river in 1918 was 222.04 km and the lengths of the reach from Gopalganj to Nazirpur in 1771, 1863, 1944 and 1963 were 39.62 miles, 43.68 miles, 31.20 miles and 34.63 miles respectively.

**FIGURE**

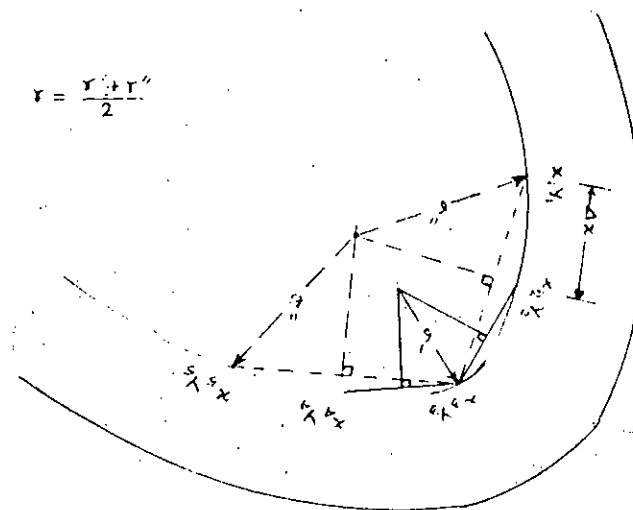


Fig. 2.1 : Definition of mean radius (r)  
(Nanson and Hickin, 1983)

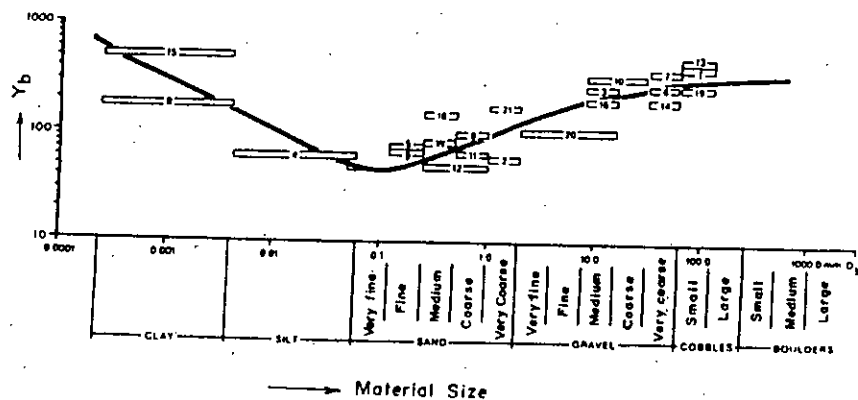


Fig. 2.2 Relation between coefficient of resistance to lateral erosion and bank material size

(Source: Hickin & Nanson, 1984) (After FAP 21/22, 1992)

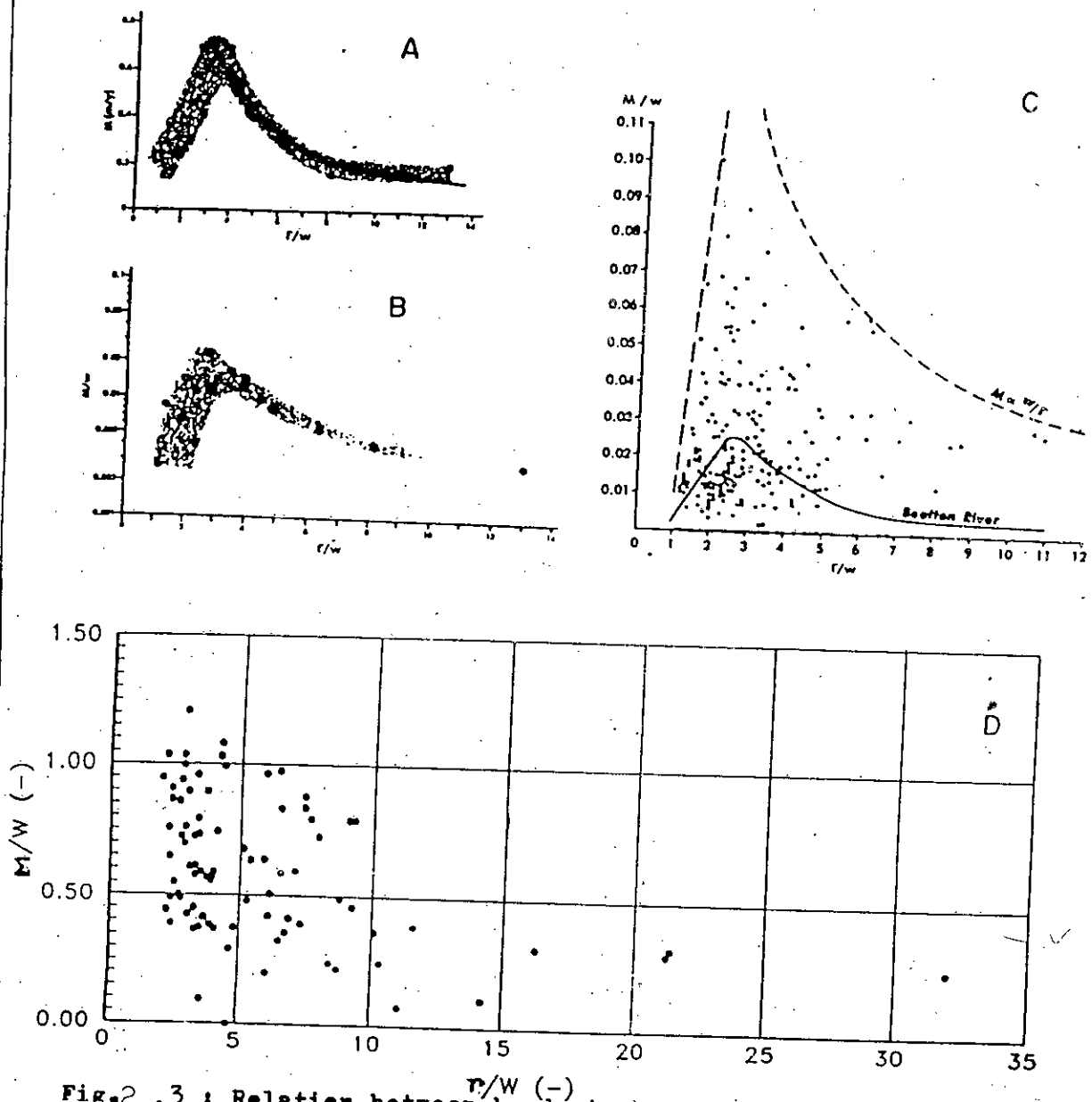
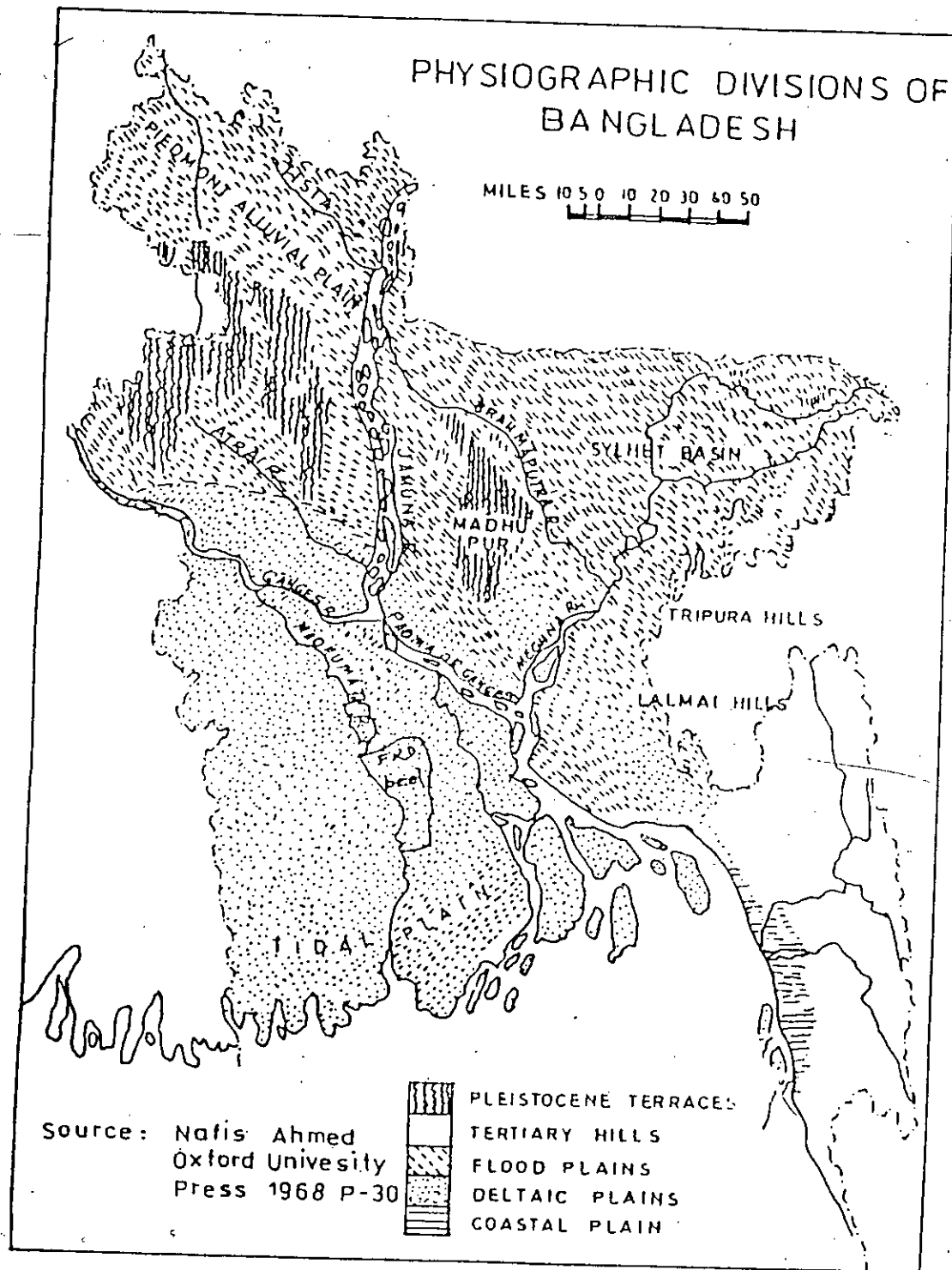


Fig.2 .3 : Relation between bend migration rate ( $M/y$ ), relative bend migration ( $M/w$ ) and relative bend curvature ( $r/w$ ) for bends in Beaton river, Canada (A&B), (C) for many rivers in Canada (Nansen & Hickin, 1984) and (D) for Jamuna river, Bangladesh (Klassen & Masselink, 1992).



**FIG.3.1 PHYSIOGRAPHIC DIVISIONS OF BANGLADESH**  
After : Walliuzzaman, 1986



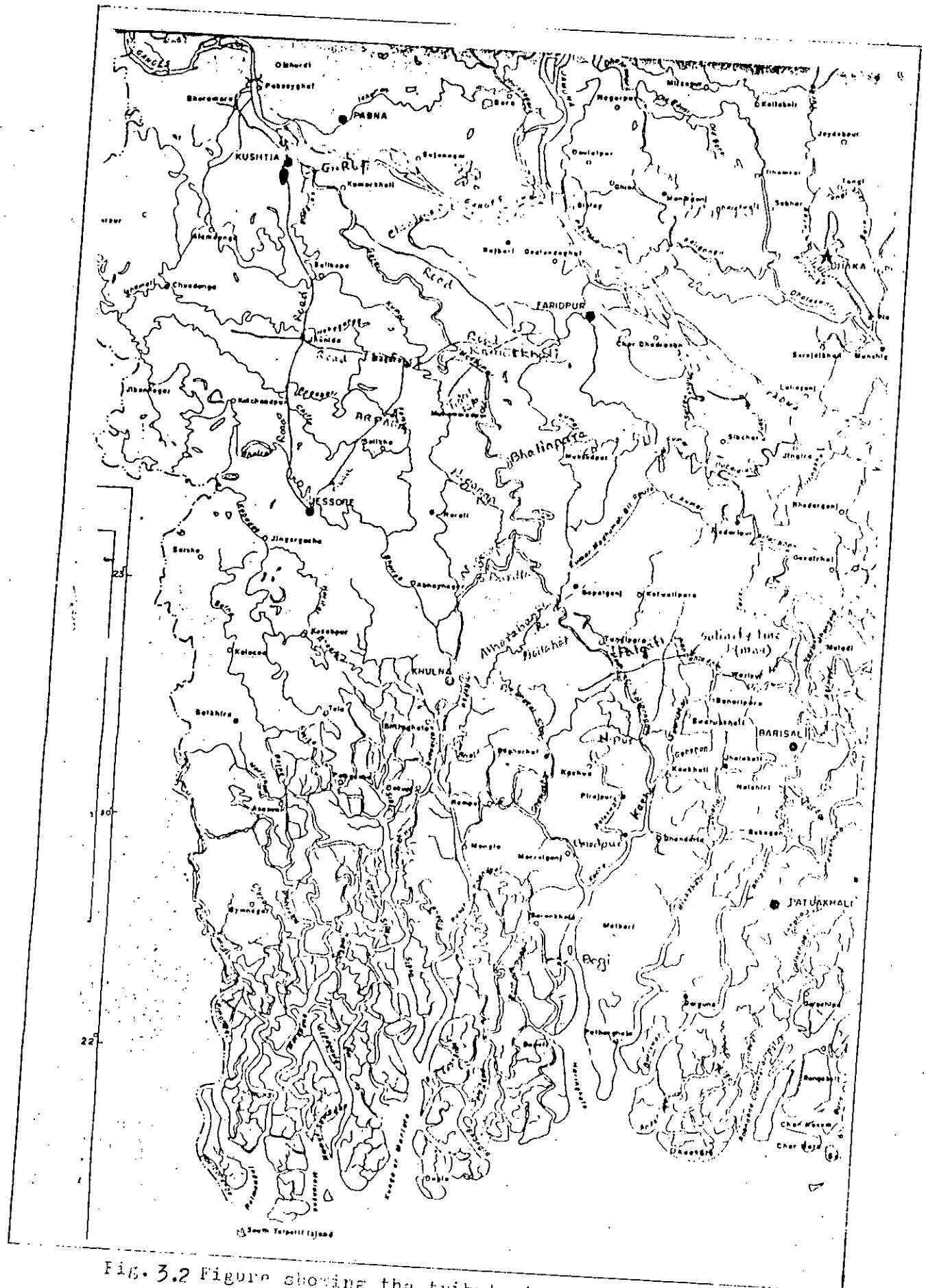


FIG. 3.2 Figure showing the tributaries and distributaries  
Of Gauri-Medhukuti-Paleswar river in 1988  
Source : SPARSO, 1988

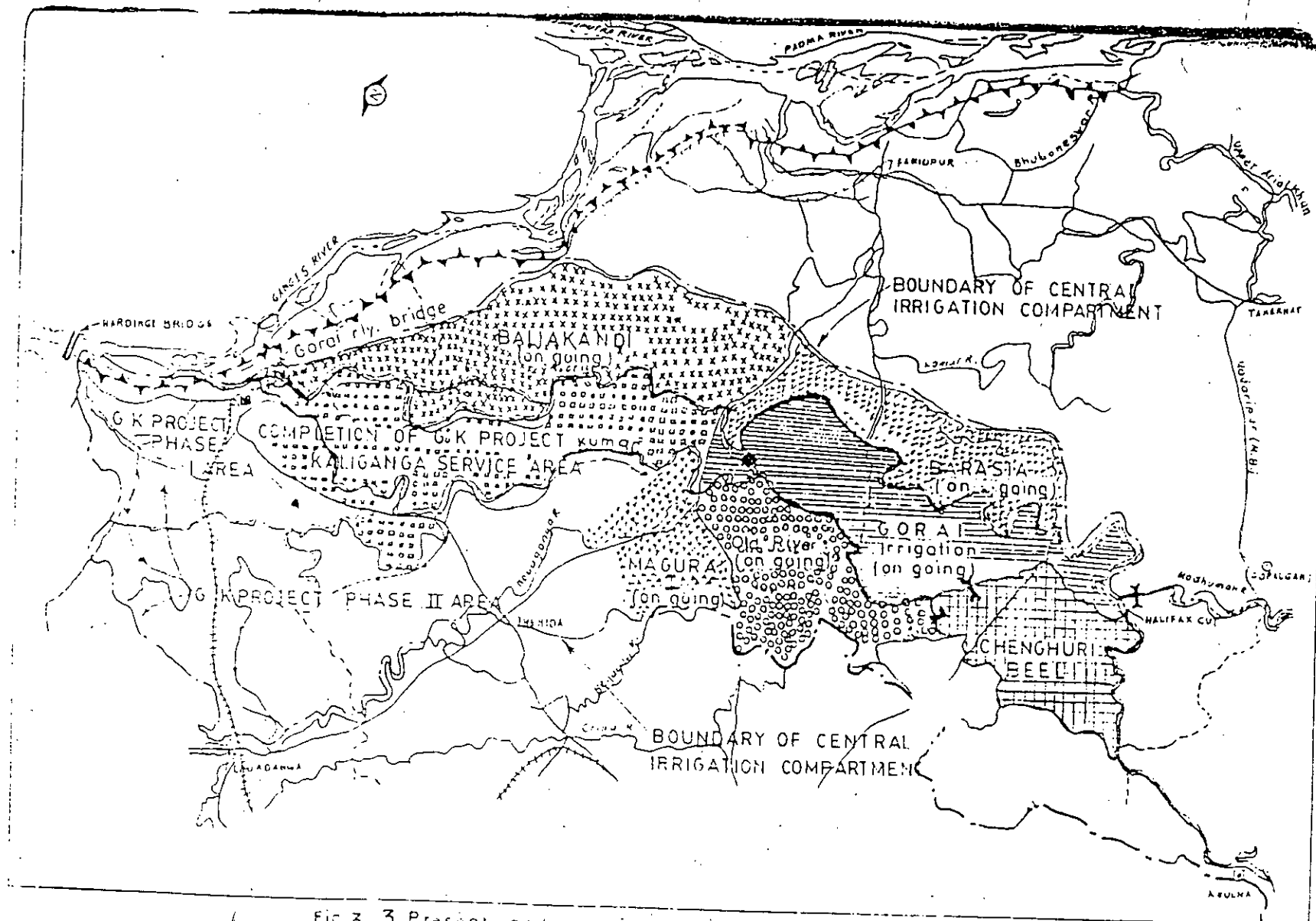


Fig. 3.3 Present and ongoing projects in the Gorai river valley. (IECO, 1981)  
 After : Walliuzzaman, 1986

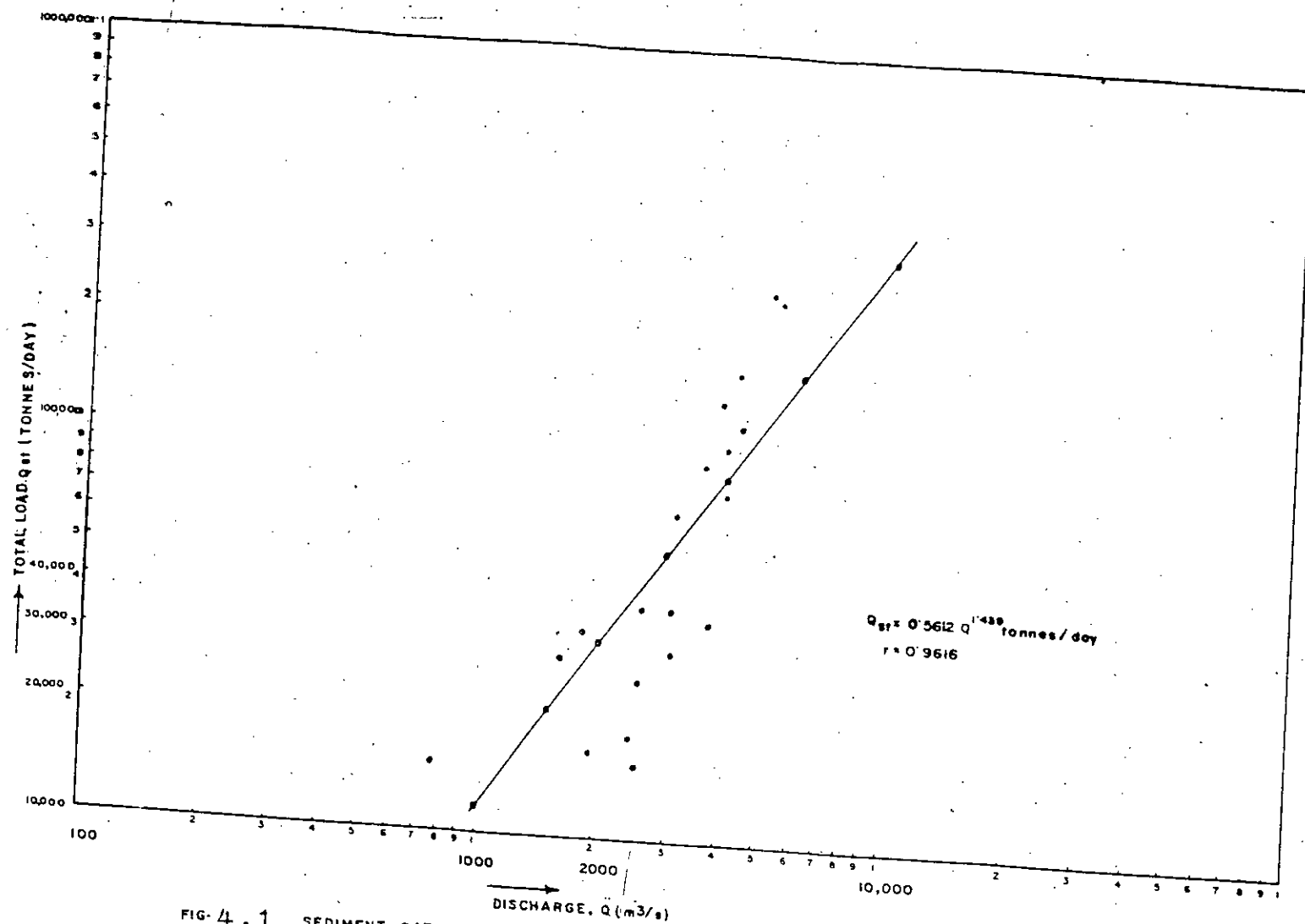


FIG. 4.1 SEDIMENT RATING CURVE FOR GORAI-MADI RIVER  
 AT GORAI RLY. BRIDGE SITE (OBSERVED DATA)

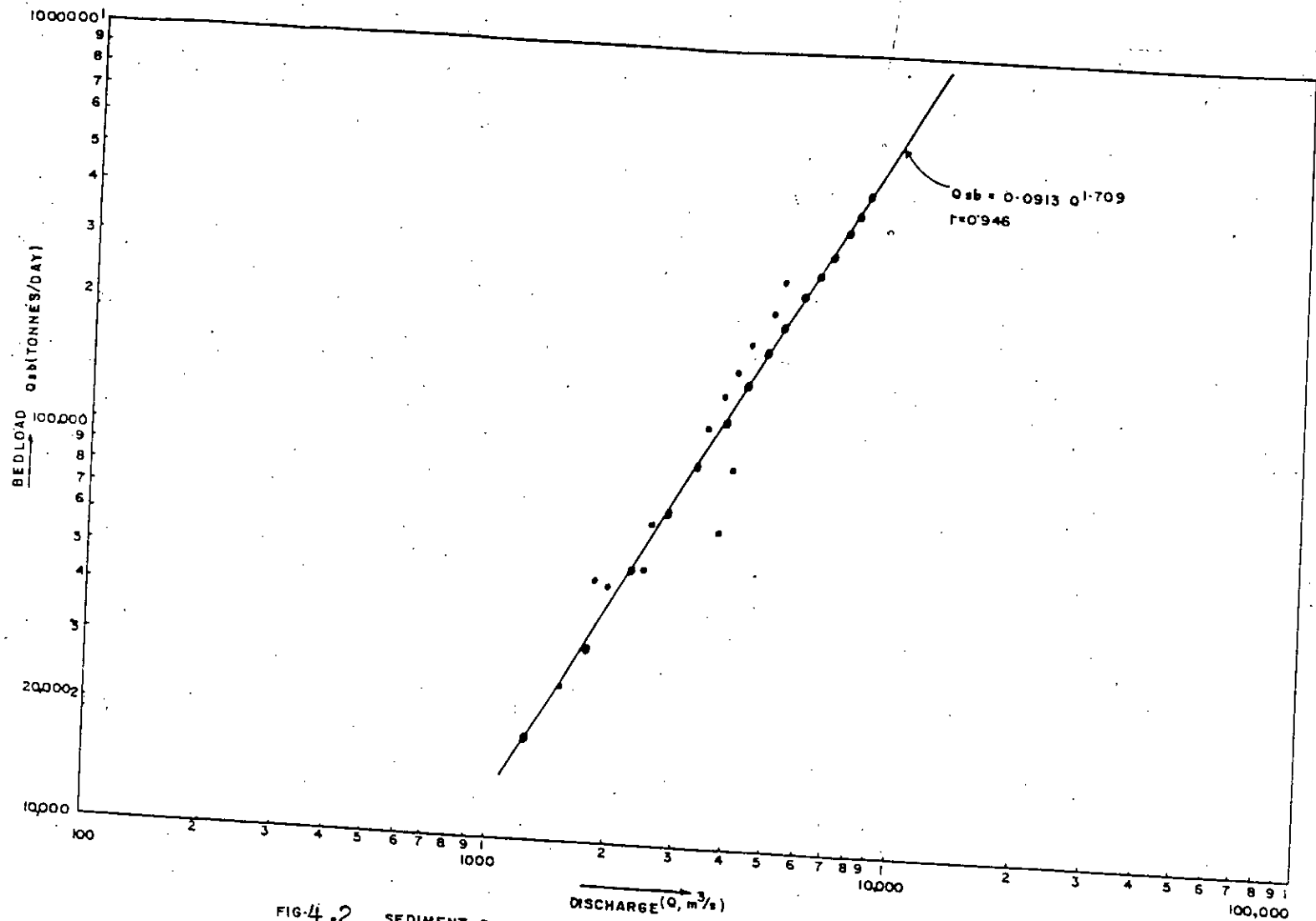


FIG-4.2 SEDIMENT RATING CURVE. DETERMINATION OF D.D. WITH  $d_{50} = 0.048 \text{ mm}$   
 GORAI RLY. BRIDGE SITE FOR GORAI MADH. RIVER (ENGELUND-HANSEN FORMULA)  
 DATA RANGE 0-500, 500-1000, 1000-1500 ETC.

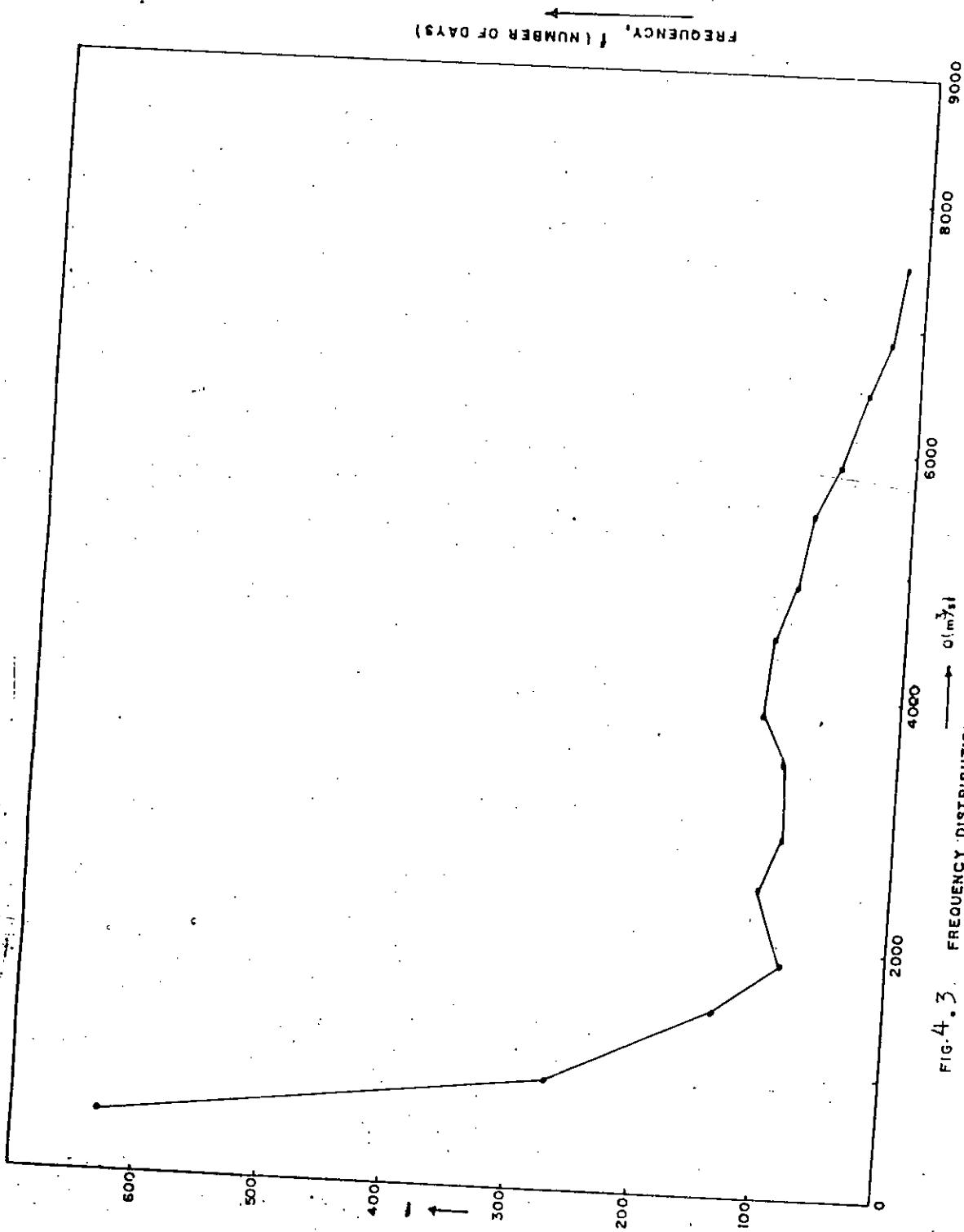


FIG. 4.3. FREQUENCY DISTRIBUTION FOR DISCHARGE AT GORAI RLY. BRIDGE, FOR GORAI MAD. RIVER.  
DATA RANGE 400-600, 800-1000 ETC. (1965-1989)

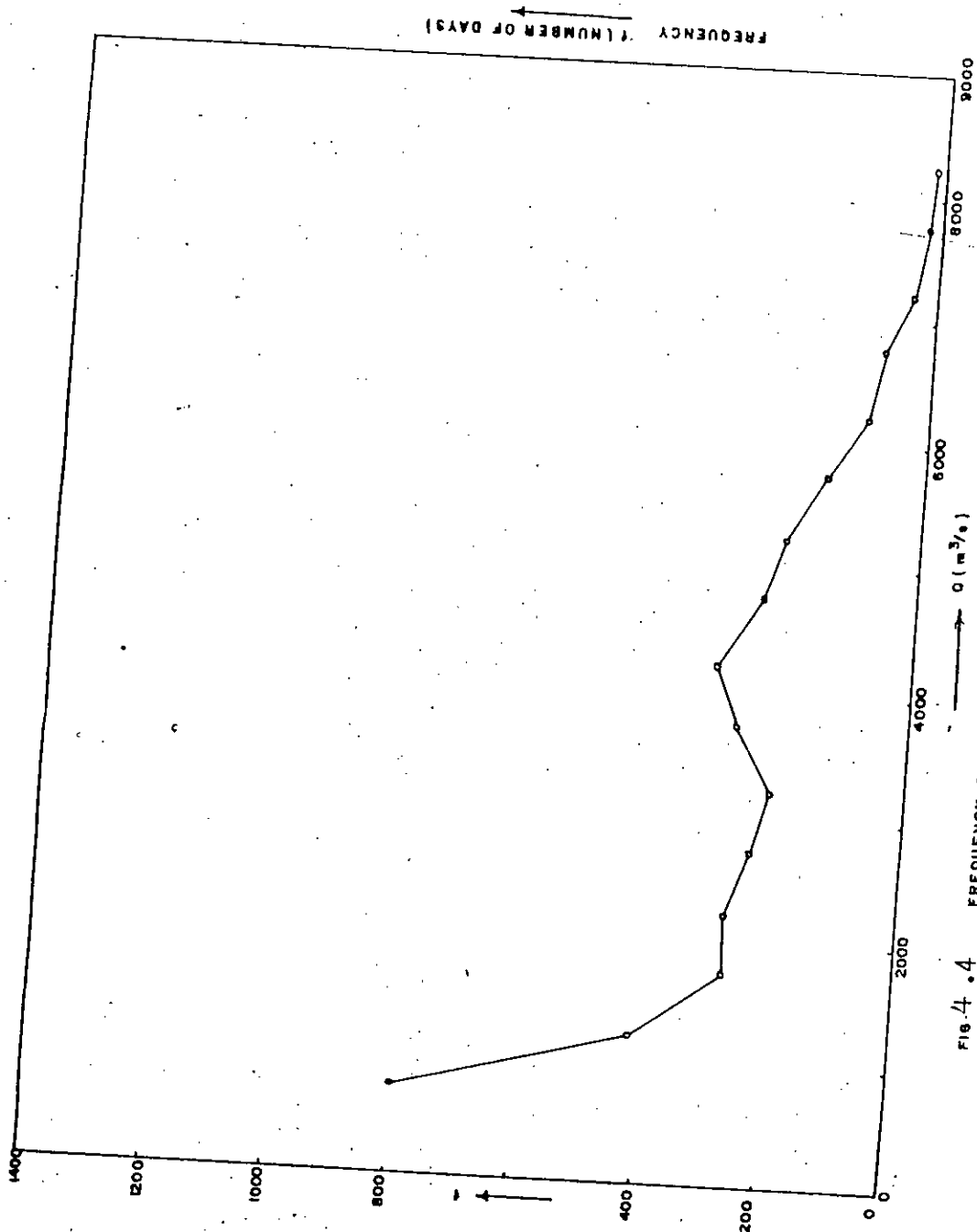


Fig. 4.4 FREQUENCY DISTRIBUTION FOR DISCHARGE AT SORAI RLY. BRIDGE FOR SORAI-MADH. RIVER.  
DATA RANGE 0-500, 500-1000, 1000-1500 ETC. (1965-69)

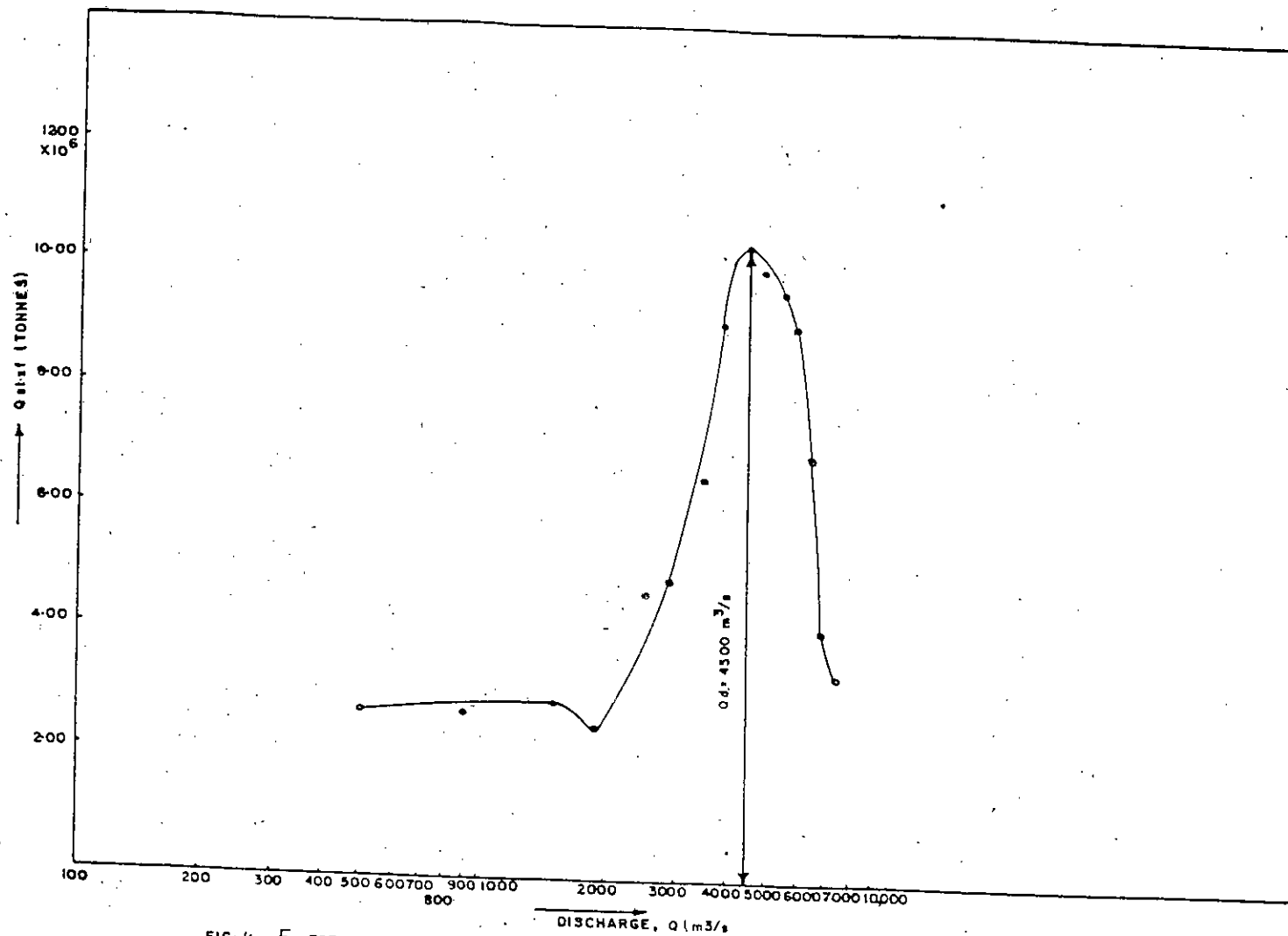


FIG. 4.5 TOTAL SEDIMENT TRANSPORT (OBSERVED) VS DISCHARGE CURVE AT GORAI-RLY. BRIDGE FOR GORAI-MADHUMATI RIVER WITH DATA RANGE AS 400-600, 800-1000 ETC.

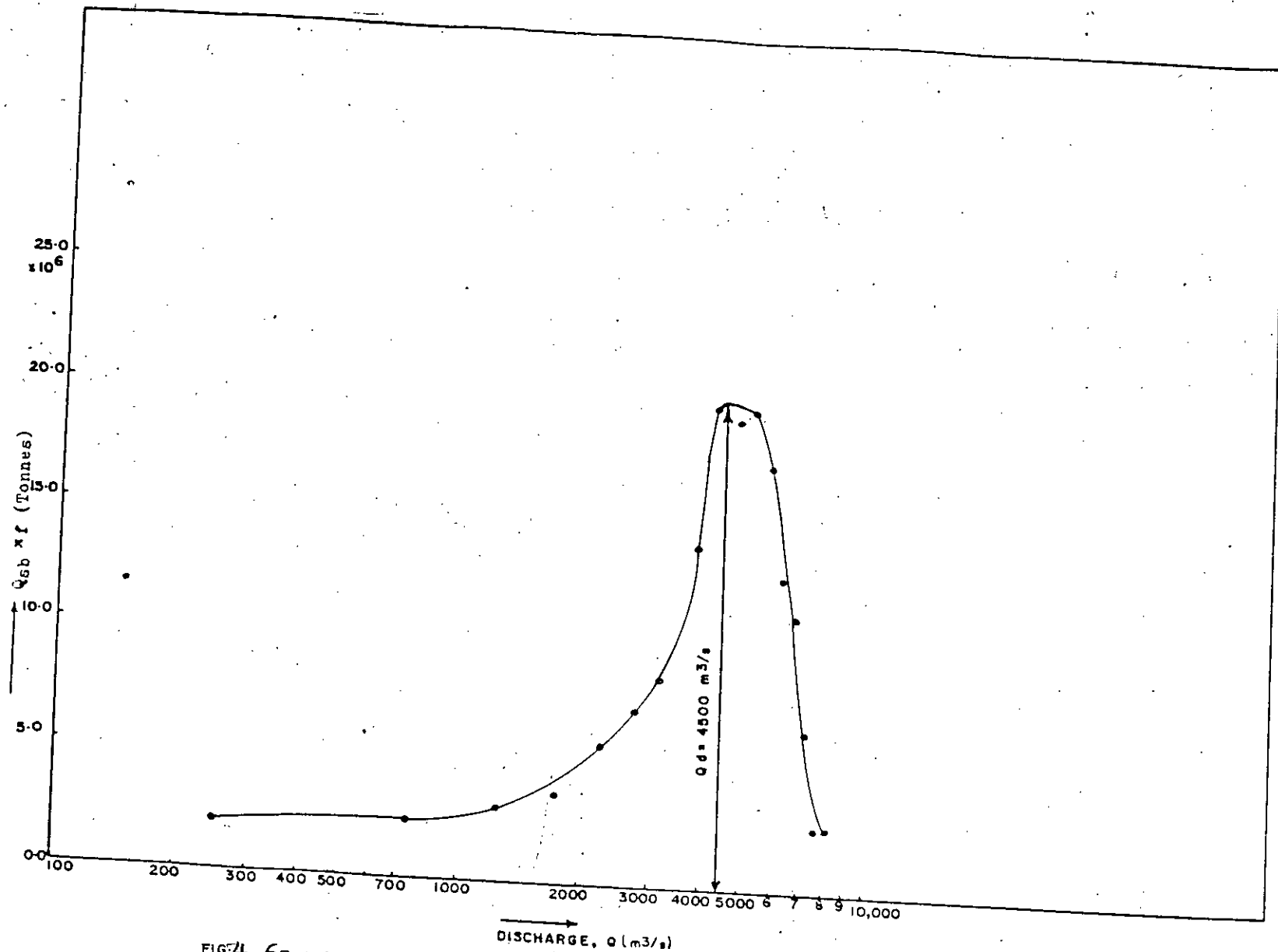


FIG 4.6 Total sediment transport (calculated) Vs. discharge curve with  $d_{50} = 0.15mm$  Data range as 500-1000, 1000-1500 etc. Gorai-Madhupati river at Gorai Rly bridge (1965-89)



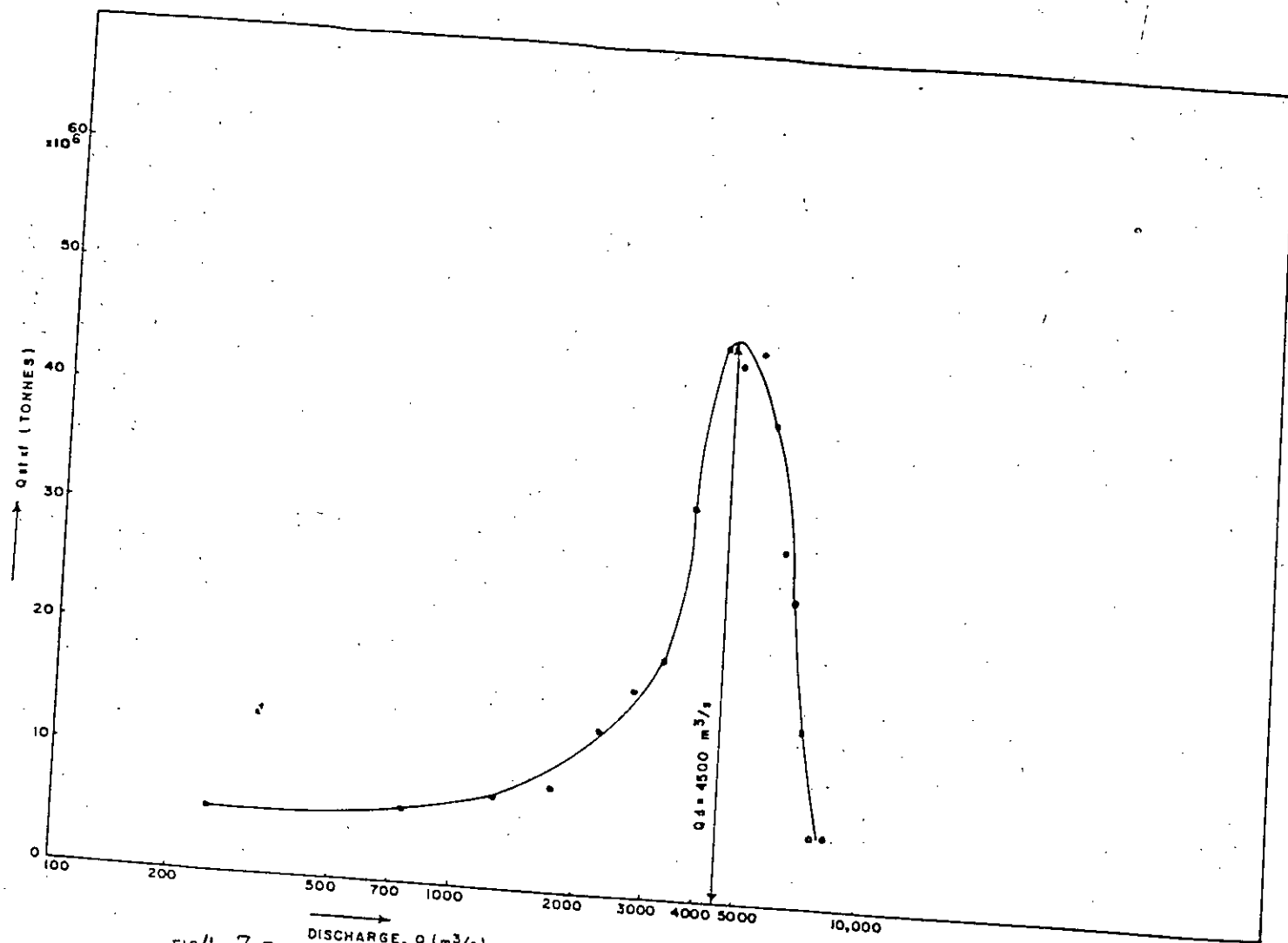


Fig 4.7 Total sediment transport (calculated) Vs. discharge curve with  $d_{50} = 0.043mm$  Data range as 500-1000, 1000-1500 etc. Gorai-Madhumati river at Gorai Rly bridge (1965-89)

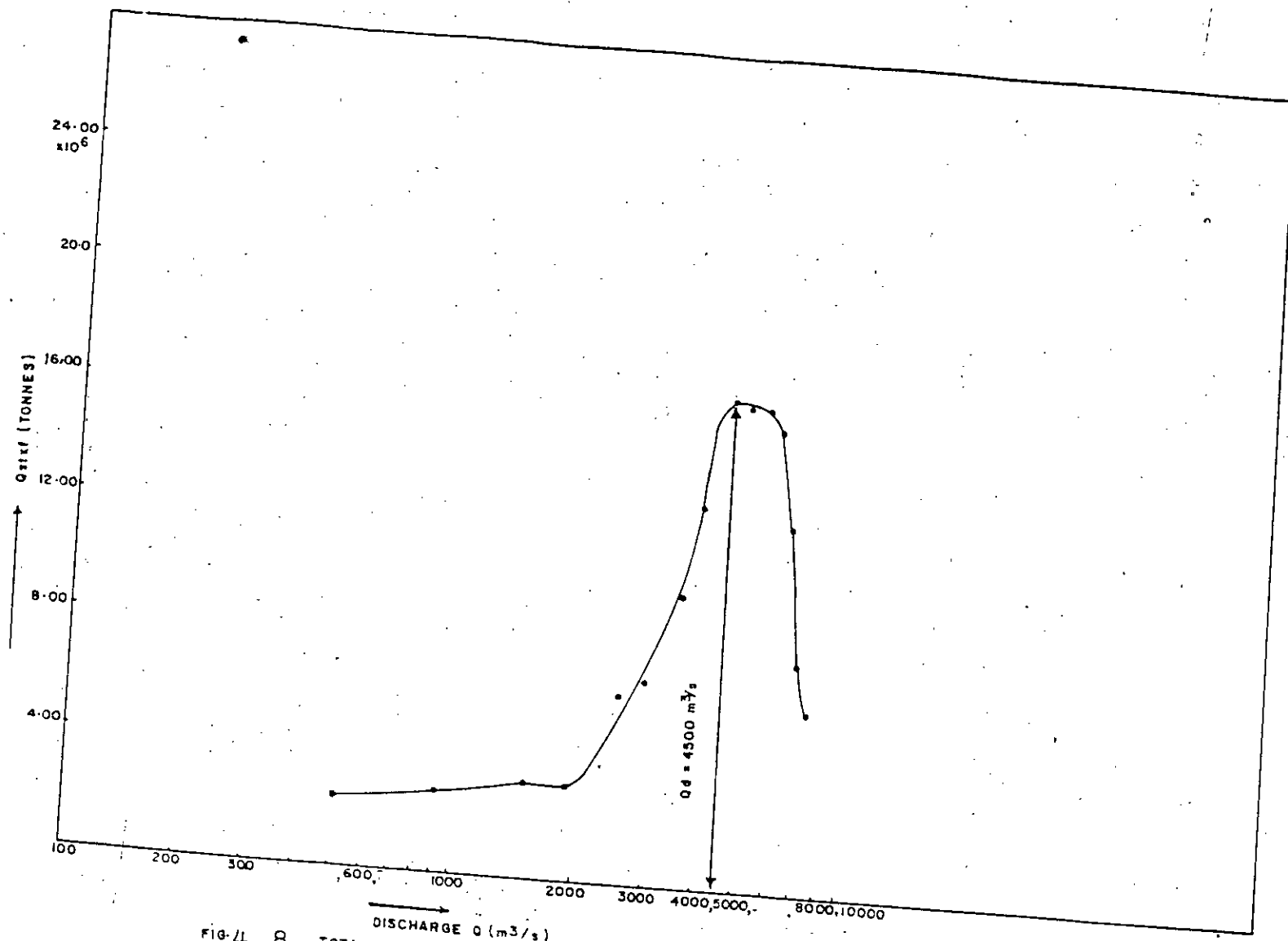


FIG-4.8 TOTAL SEDIMENT TRANSPORT (PREDICTED BY ENGELUND HANSEN FORMULA) V. DISCHARGE CURVE AT GORAI RLY. BRIDGE FOR GORAI MADH RIVER WITH  $d_{50} = 0.048$  mm AND DATA RANGE AS 400-600, 800-1000 ETC.

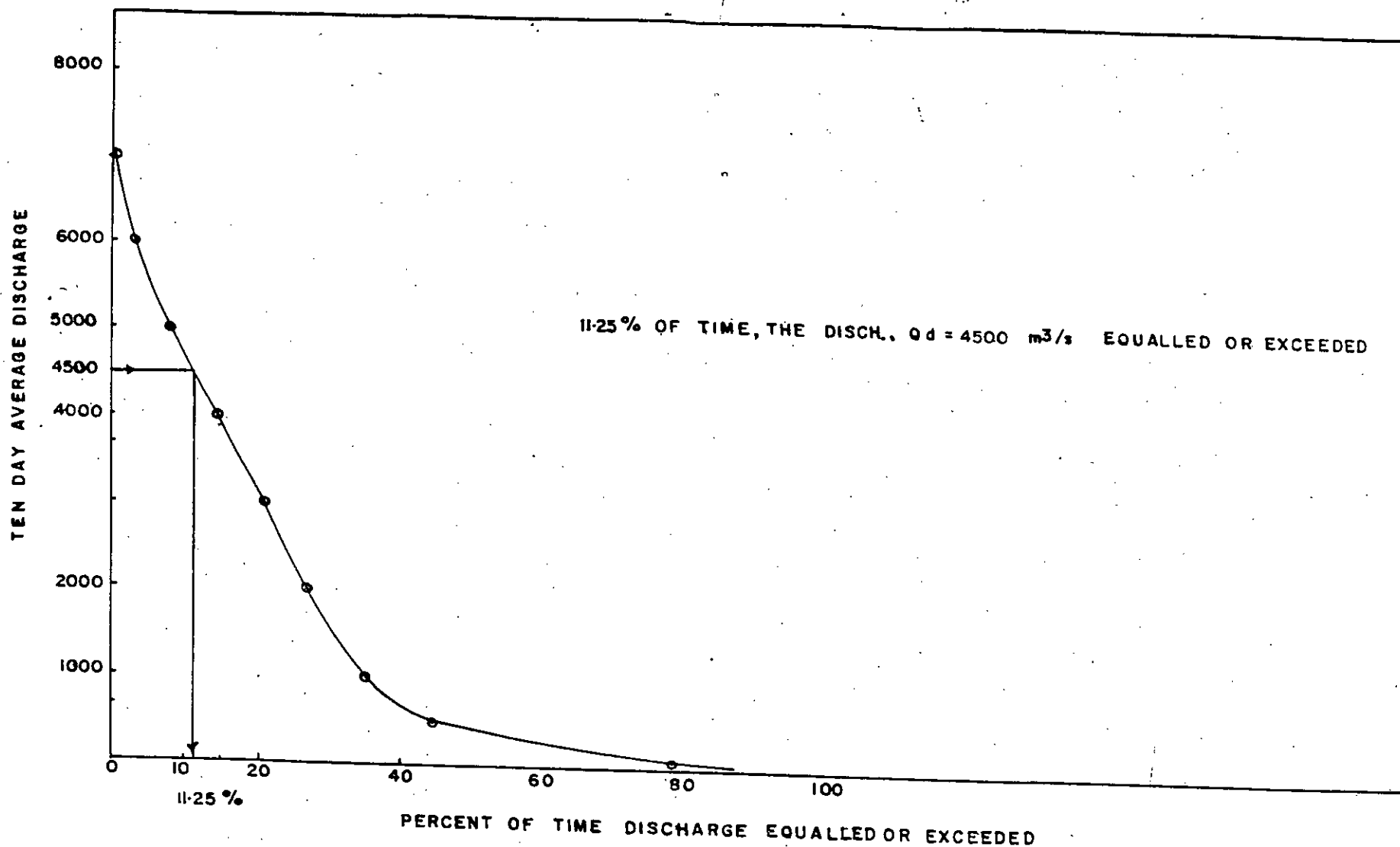


FIG. 4.9 FLOW DURATION CURVE FOR THE GORAI MADHUMATI RIVER (65-89)  
AT GORAI RLY. BRIDGE SITE.

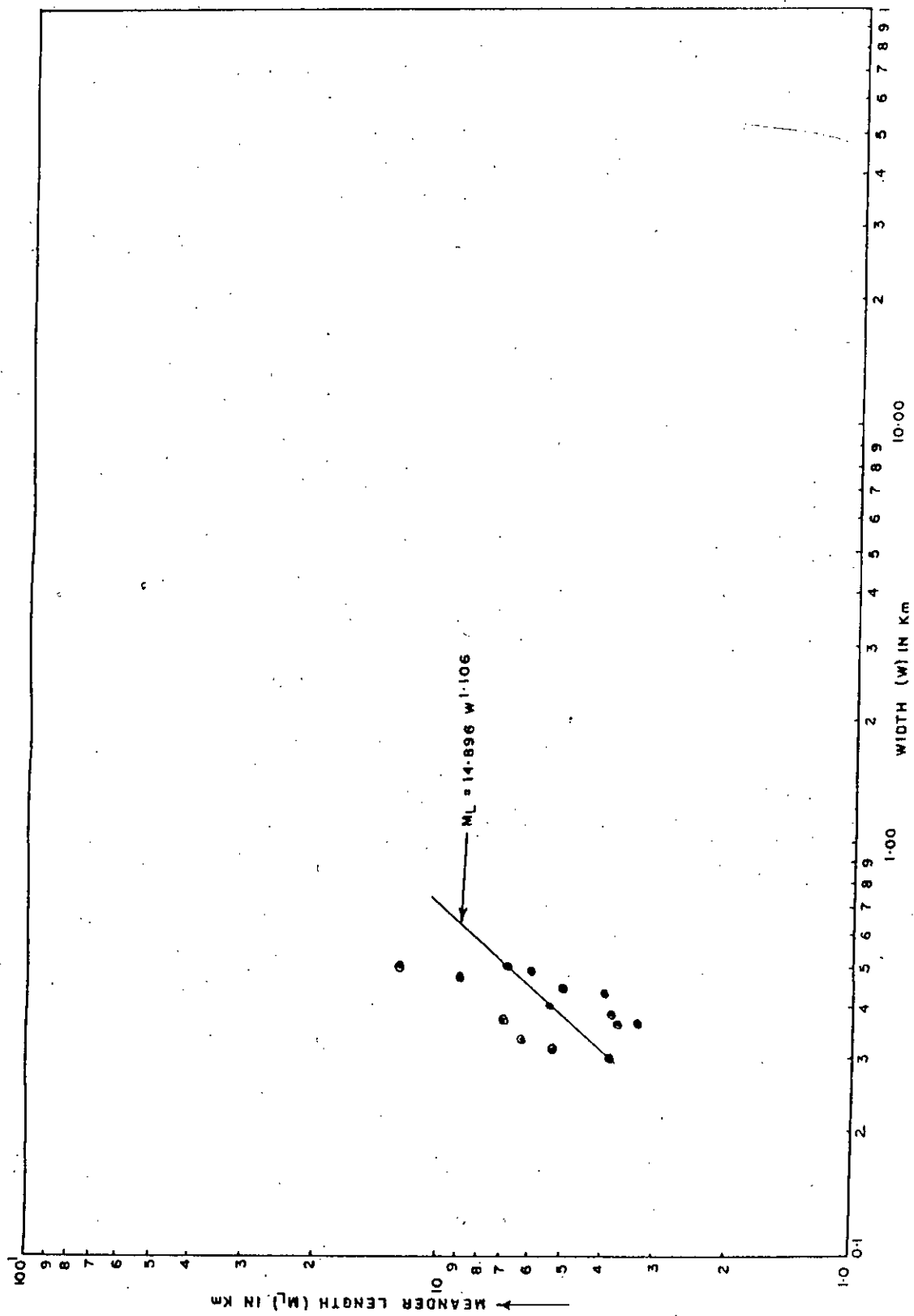


FIG. 4. CORRELATION BETWEEN MEANDER WAVE LENGTH & CHANNEL WIDTH FOR THE YEAR 1953

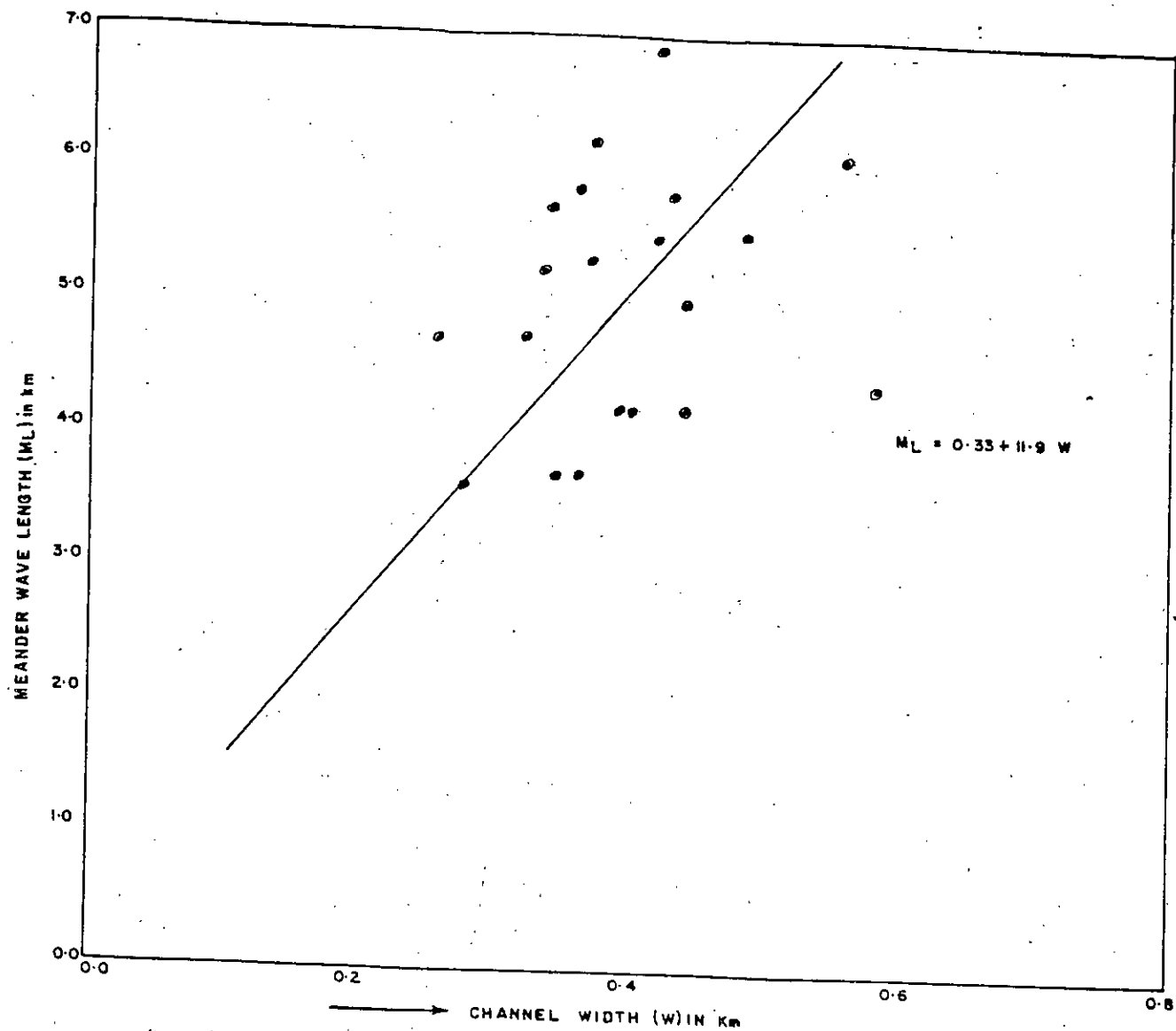


FIG-4.11 THE GENERAL RELATIONSHIP BETWEEN MEANDER WAVE LENGTH ( $M_L$ ) AND THE CHANNEL WIDTH ( $W$ ) (GORAL-MADHUMATI RIVER).

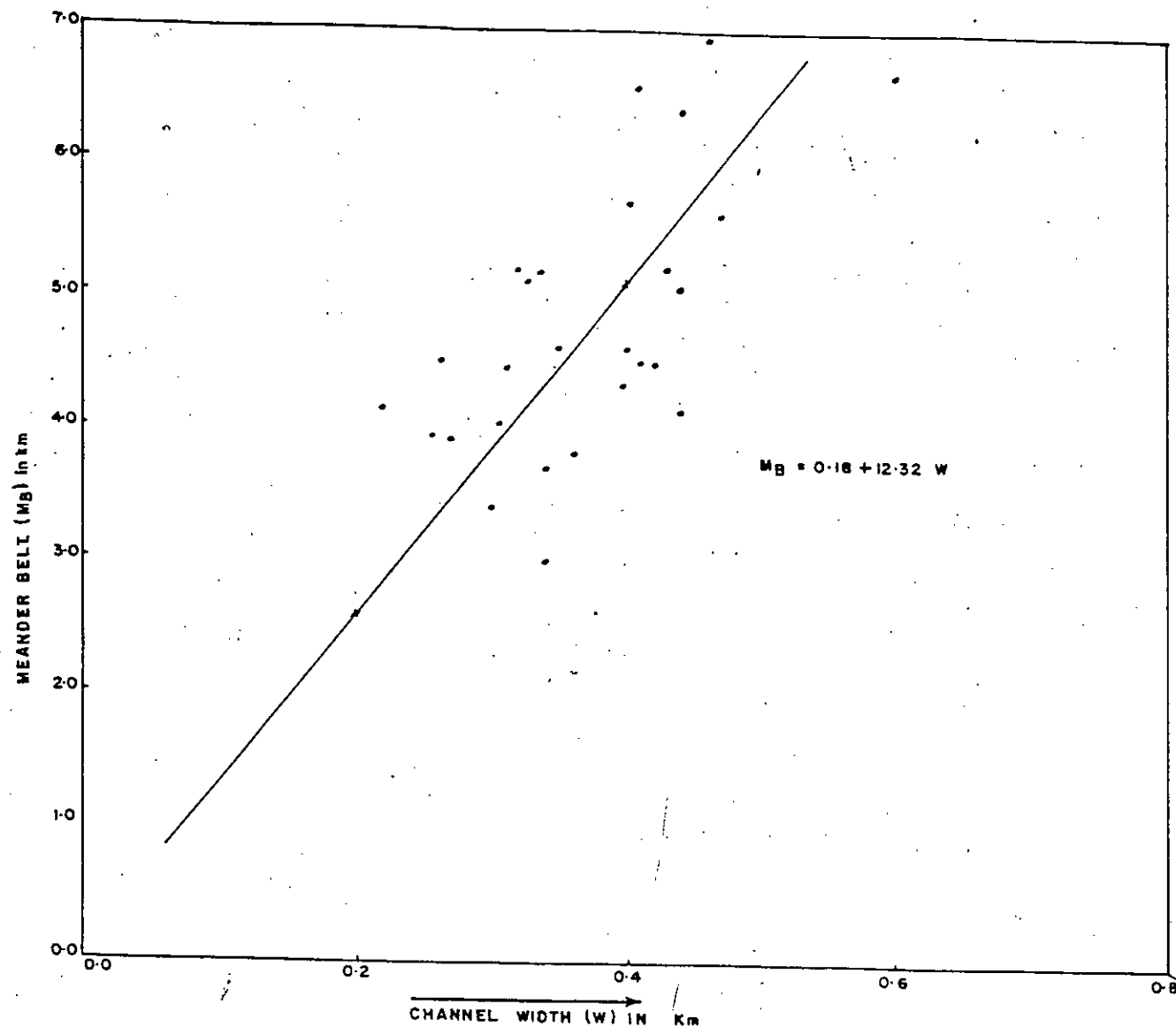


FIG. 4.12 THE GENERAL RELATIONSHIP BETWEEN MEANDER BELT ( $M_B$ ) AND CHANNEL WIDTH ( $W$ ) (GORAI-MADHUMATI RIVER)

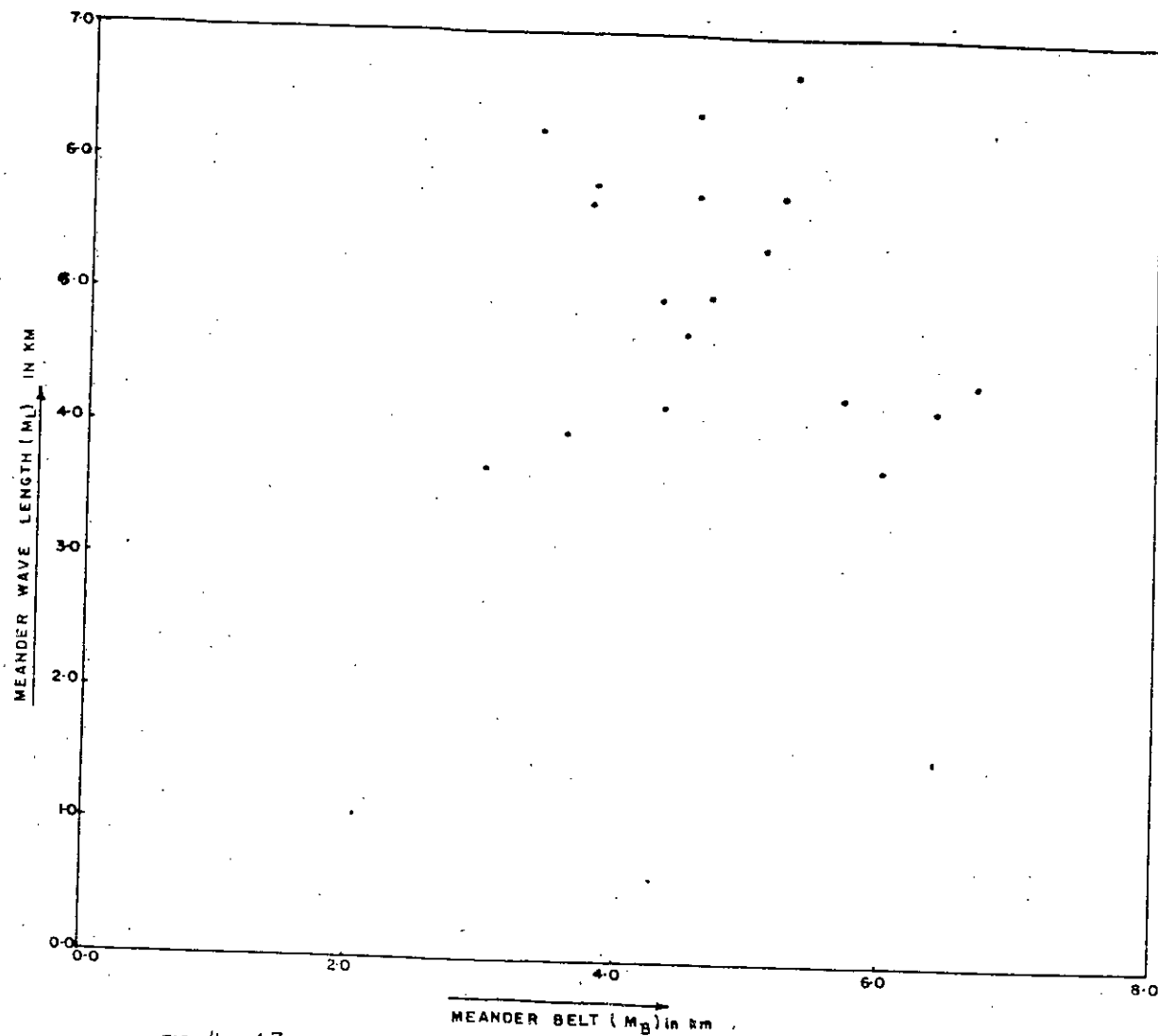


FIG. 4.13 THE GENERAL RELATIONSHIP BETWEEN: MEANDER WAVE LENGTH ( $M_L$ ) AND MEANDER BELT ( $M_B$ ). (GORAI-MADHUMATI RIVER)

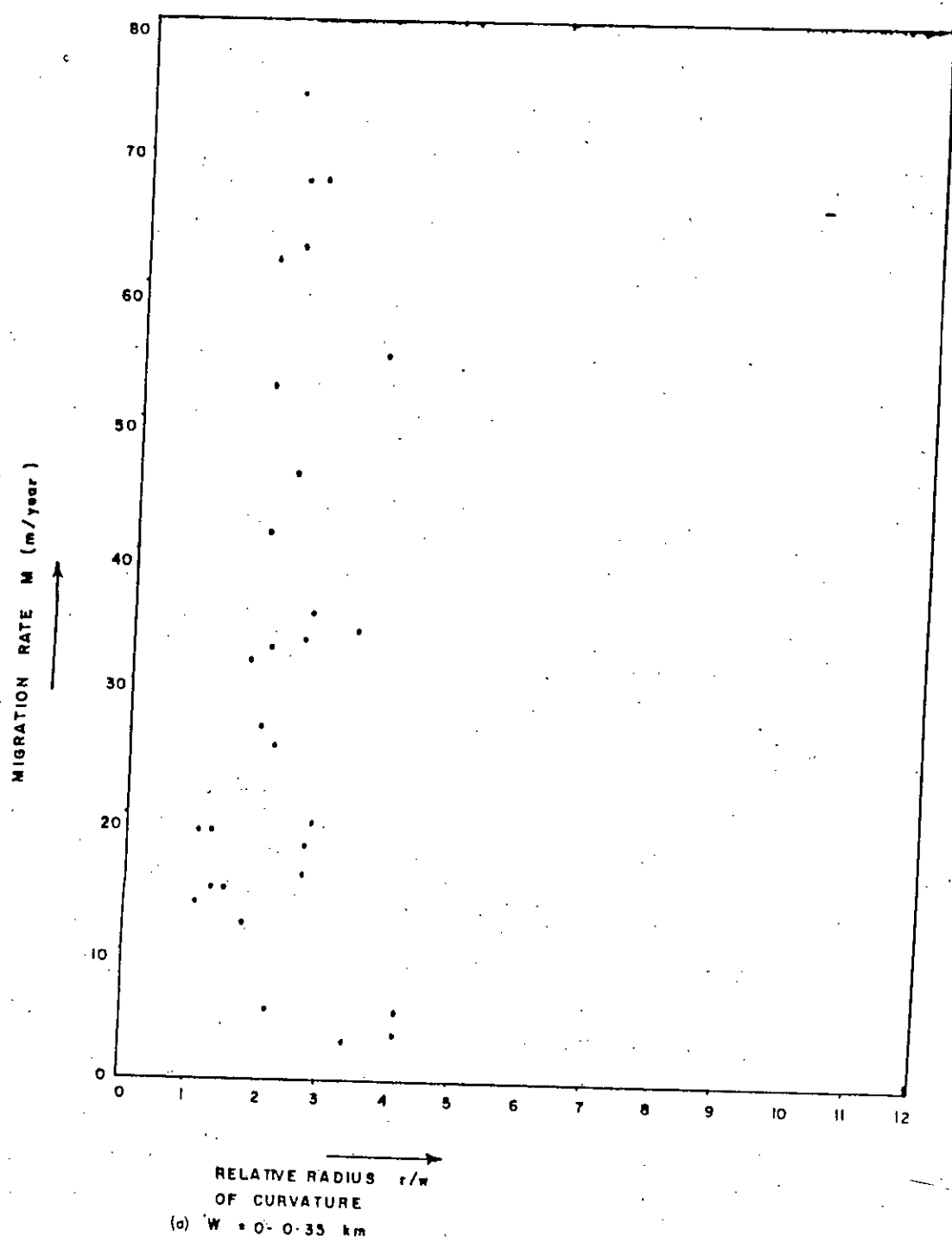


FIG. 4.14 THE RELATION BETWEEN THE BEND MIGRATION RATE ( $M/Y$ ) AND RELATIVE BEND CURVATURE ( $r/w$ ) FOR THE GORAI MADHUMATI RIVER.



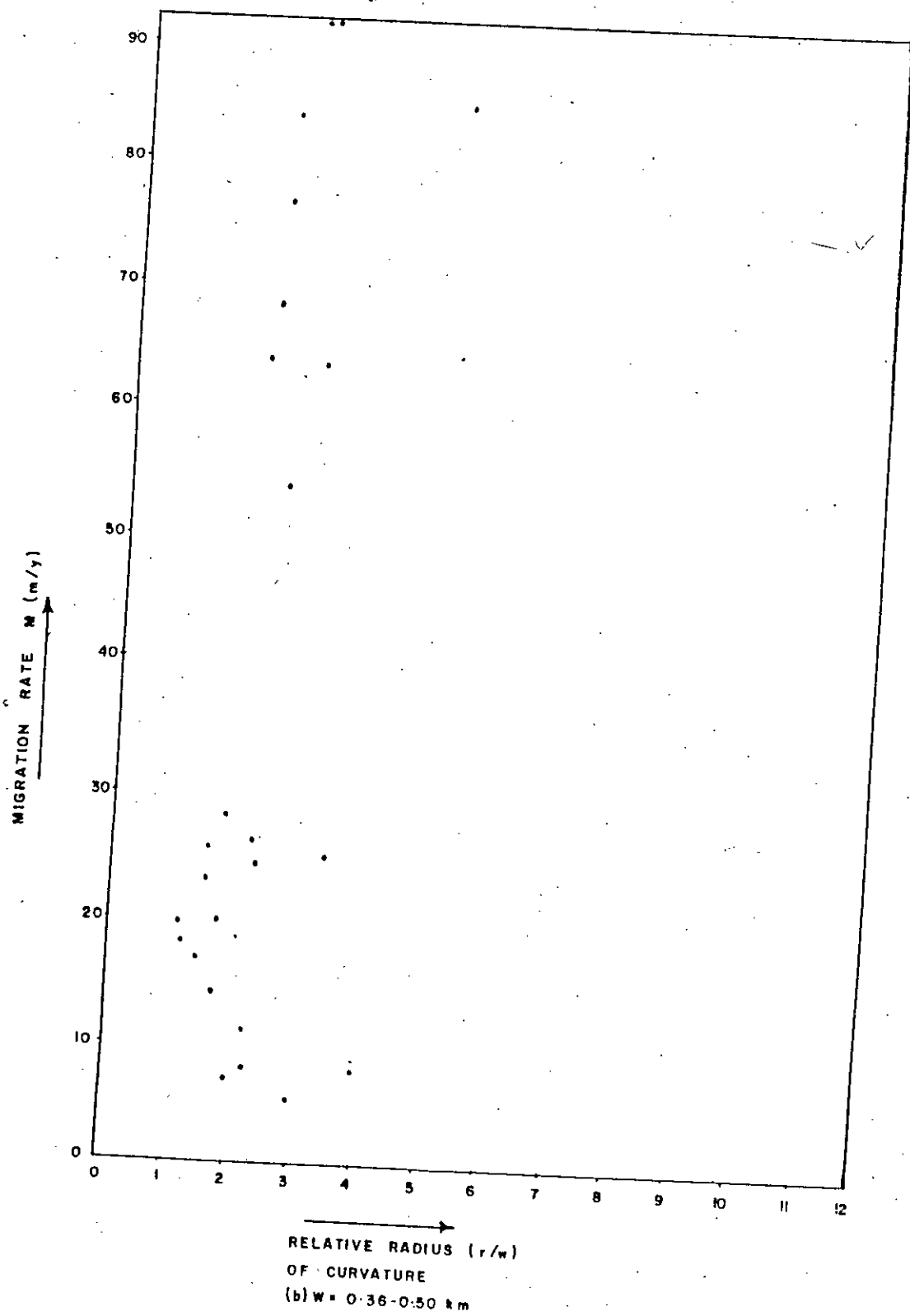


FIG 4.15 THE RELATION BETWEEN THE BEND MIGRATION RATE (m/y) AND RELATIVE BEND CURVATURE ( $r/w$ ) FOR THE GORAI-MADHUMATI RIVER.

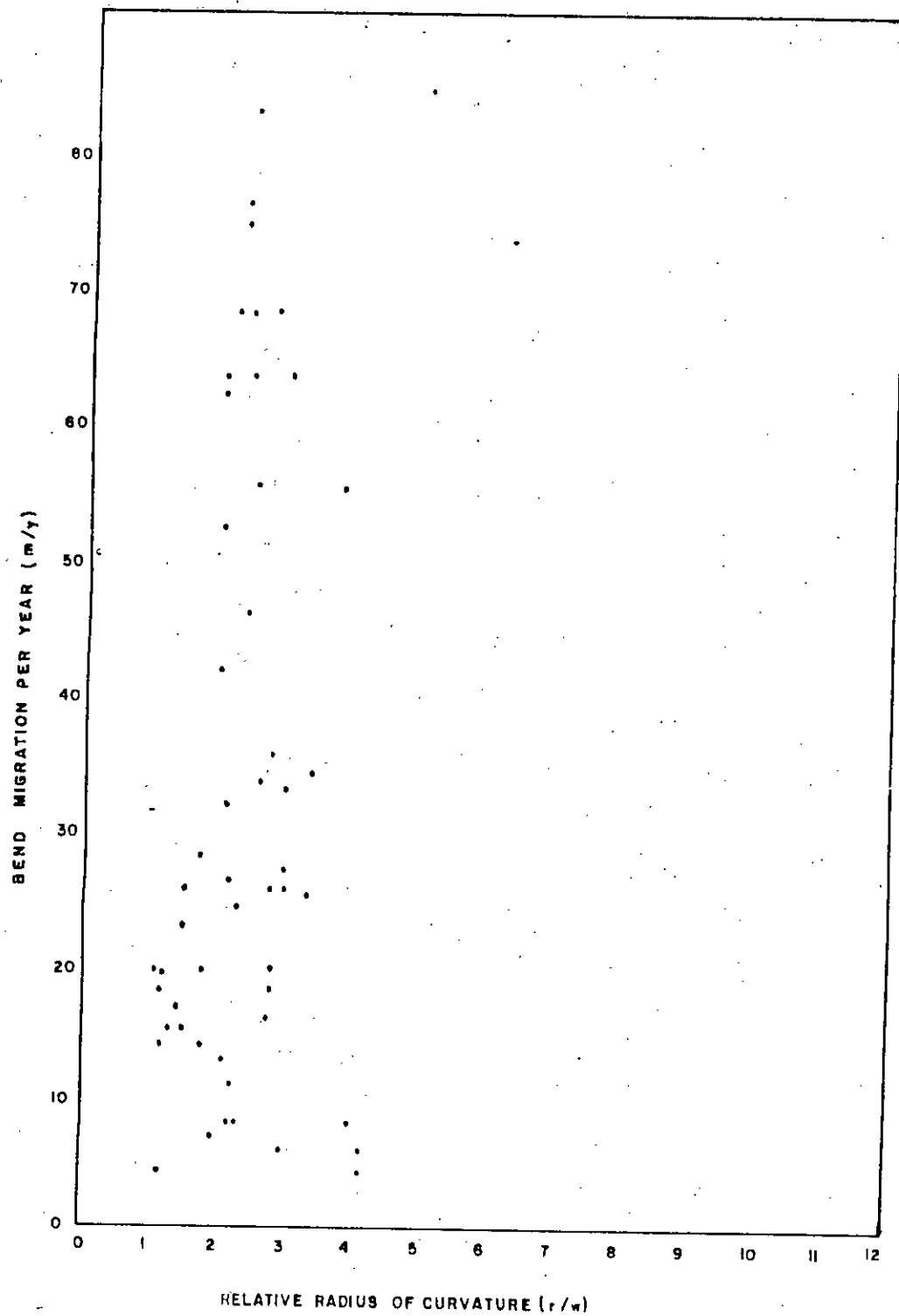


FIG. 4.16 THE GENERAL RELATION BETWEEN BEND MIGRATION RATE (m/y) AND RELATIVE BEND CURVATURE (r/w) FOR THE BORAI-MADHUMATI RIVER.

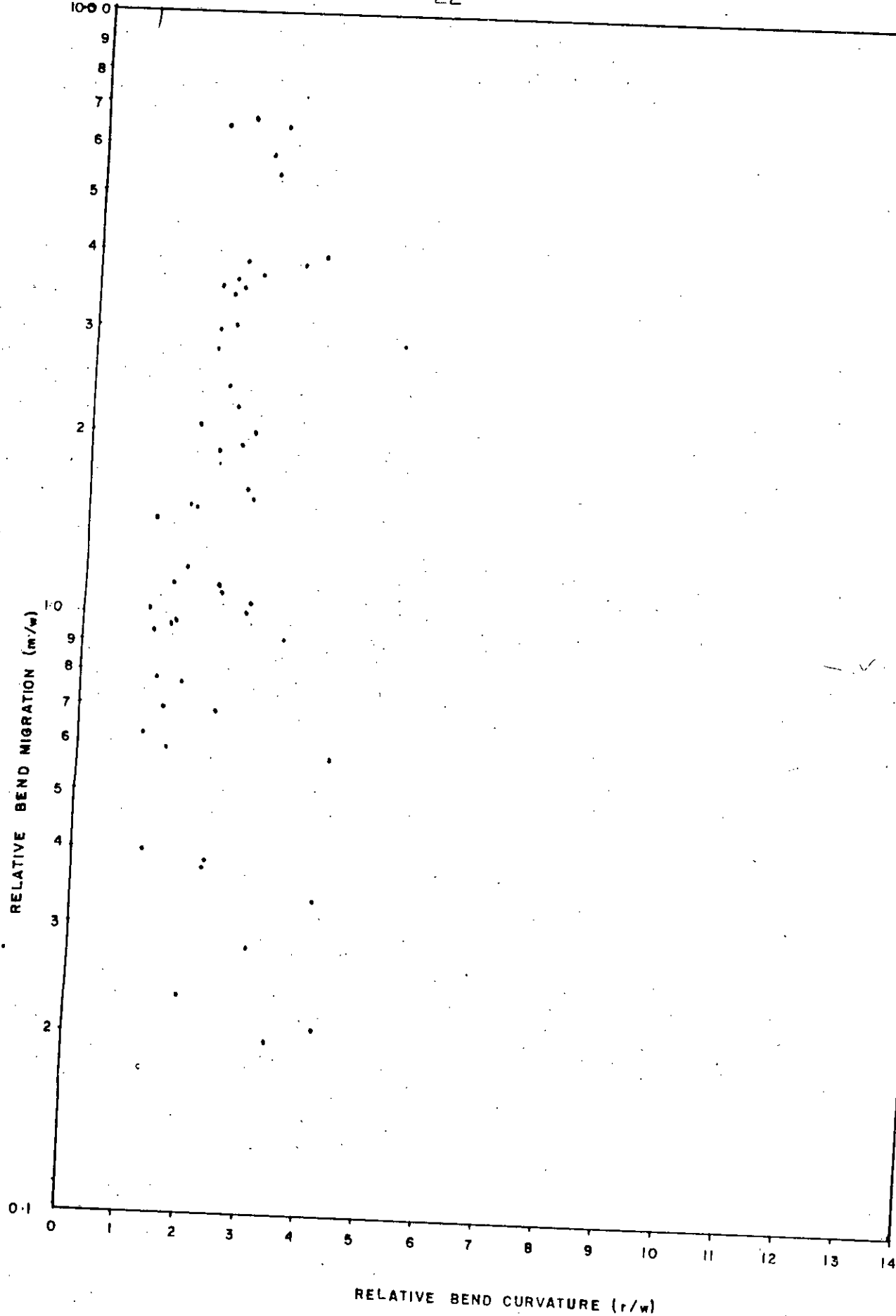


FIG 4.17

THE RELATION BET<sup>n</sup> RELATIVE BEND MIGRATION ( $m/w$ )  
AND RELATIVE BEND CURVATURE ( $r/w$ ) FOR THE GORAI-MADHUMATI RIVER

Frequency of occurrence of bend migration per year (M/y) events

20

15

10

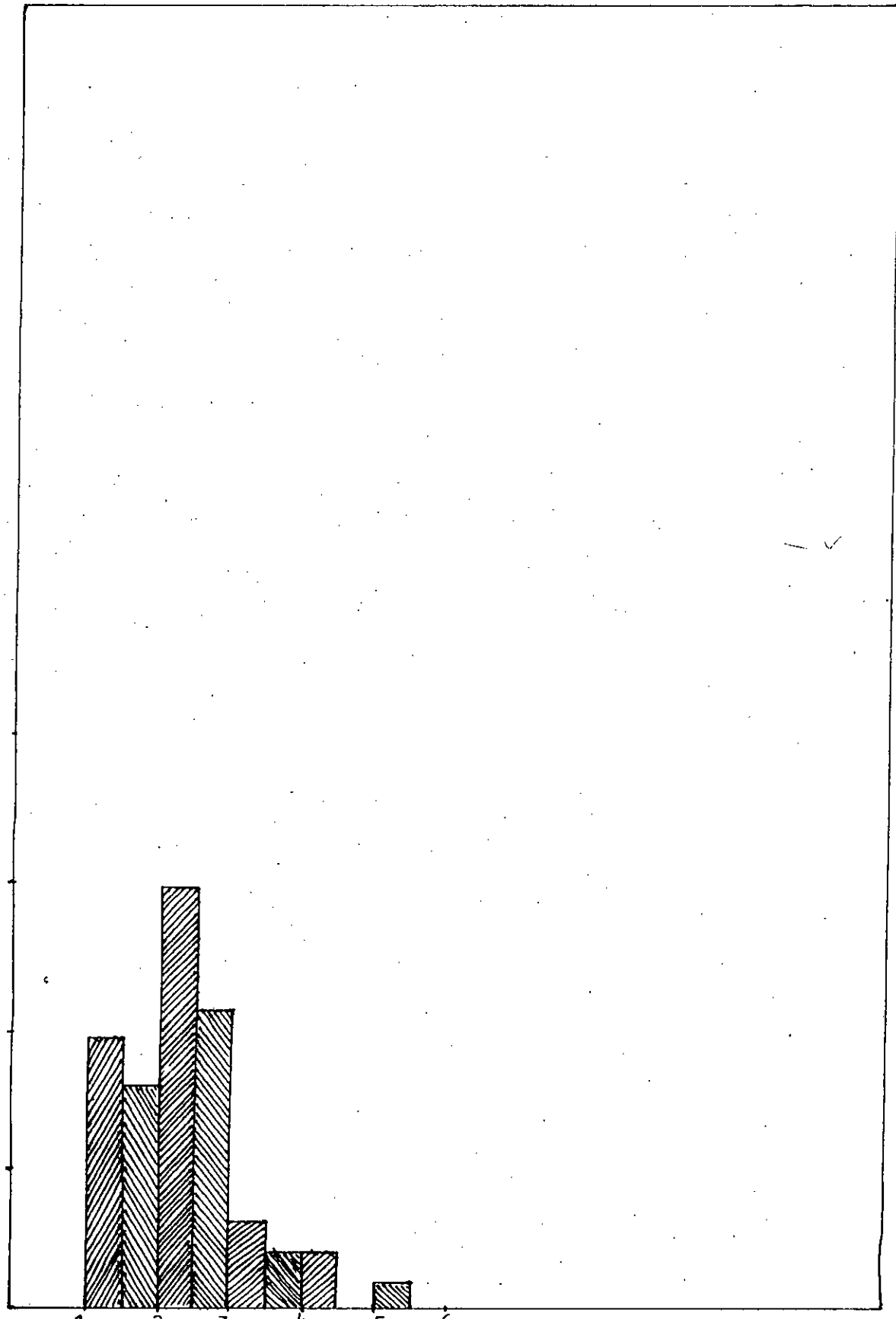
5

0

1

Relative radius of curvature (r/w)

Fig4.18 : The histogram for frequency of occurrence of bend migration(M/y) and relative bend curvature (r/w) for Gorai-Madhumati river



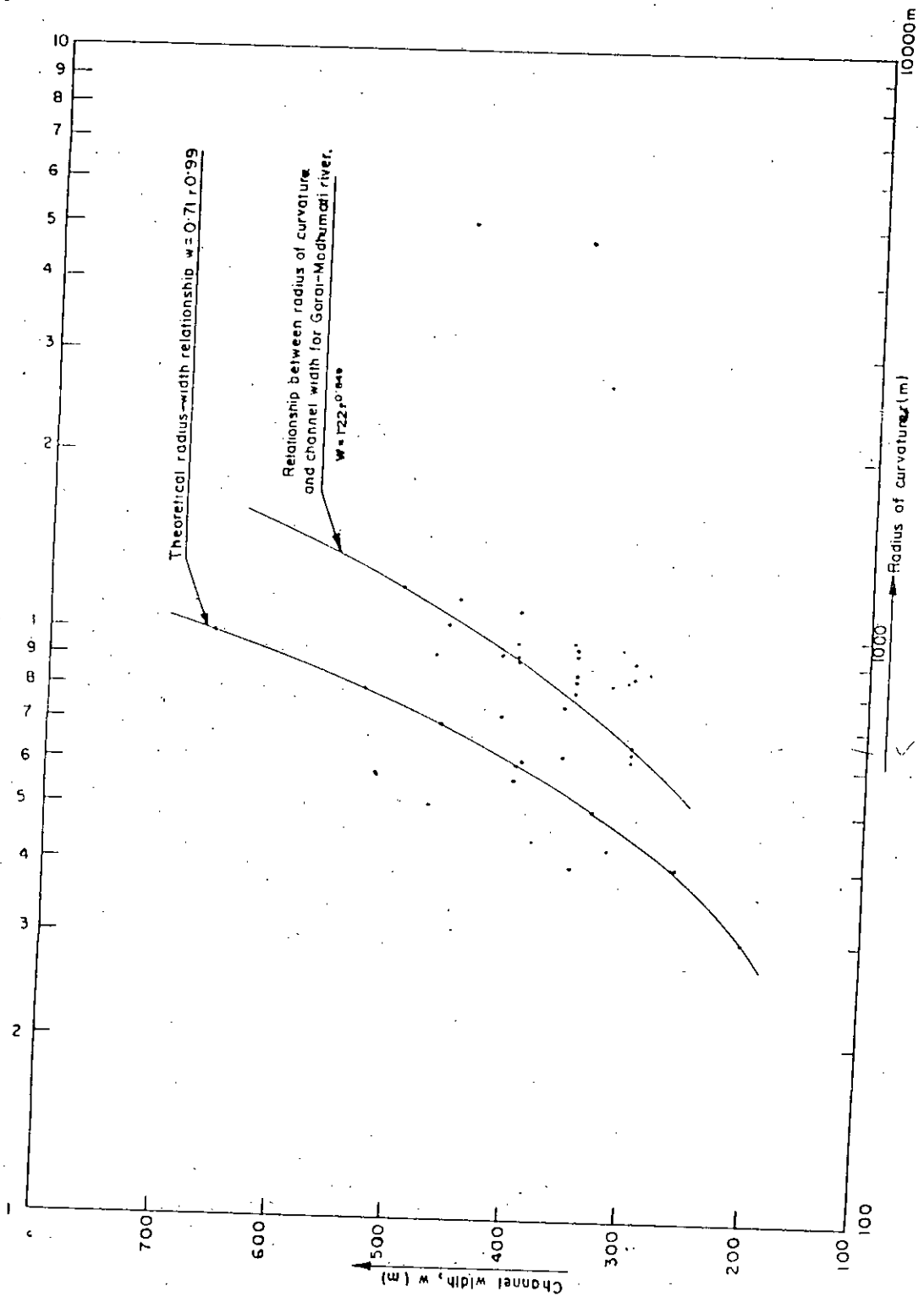


Fig. 19 Relationship between radius of curvature,  $r$ , and channel width ( $w$ ) for the Gorai-Madhumati river.

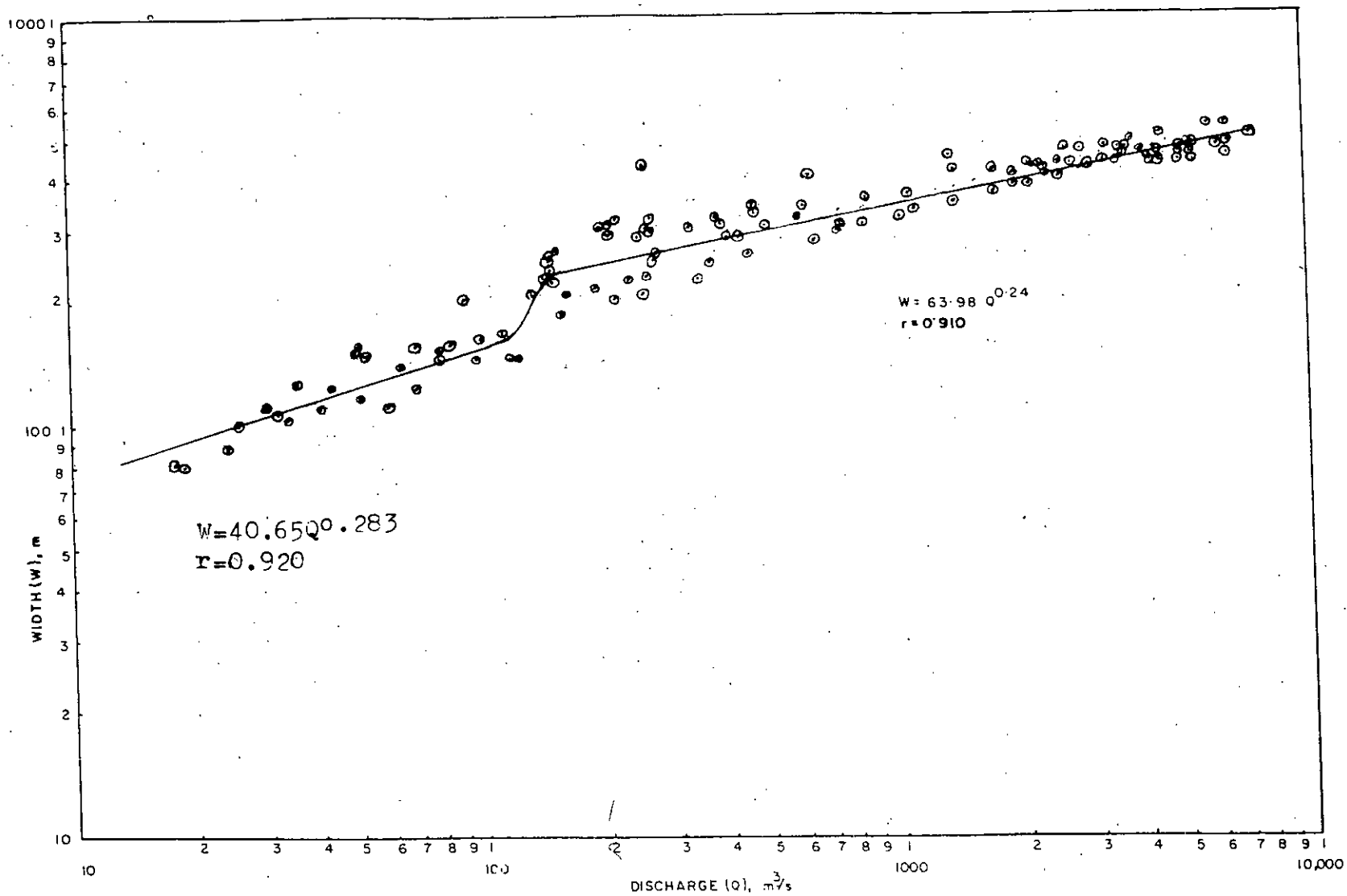


FIG. 4. 20 CHANGES OF WIDTH WITH DISCH. AT - A - RIVER CROSS SECTION  
GORAI R. BRIDGE, GORAI-MADHUMATI RIVER.

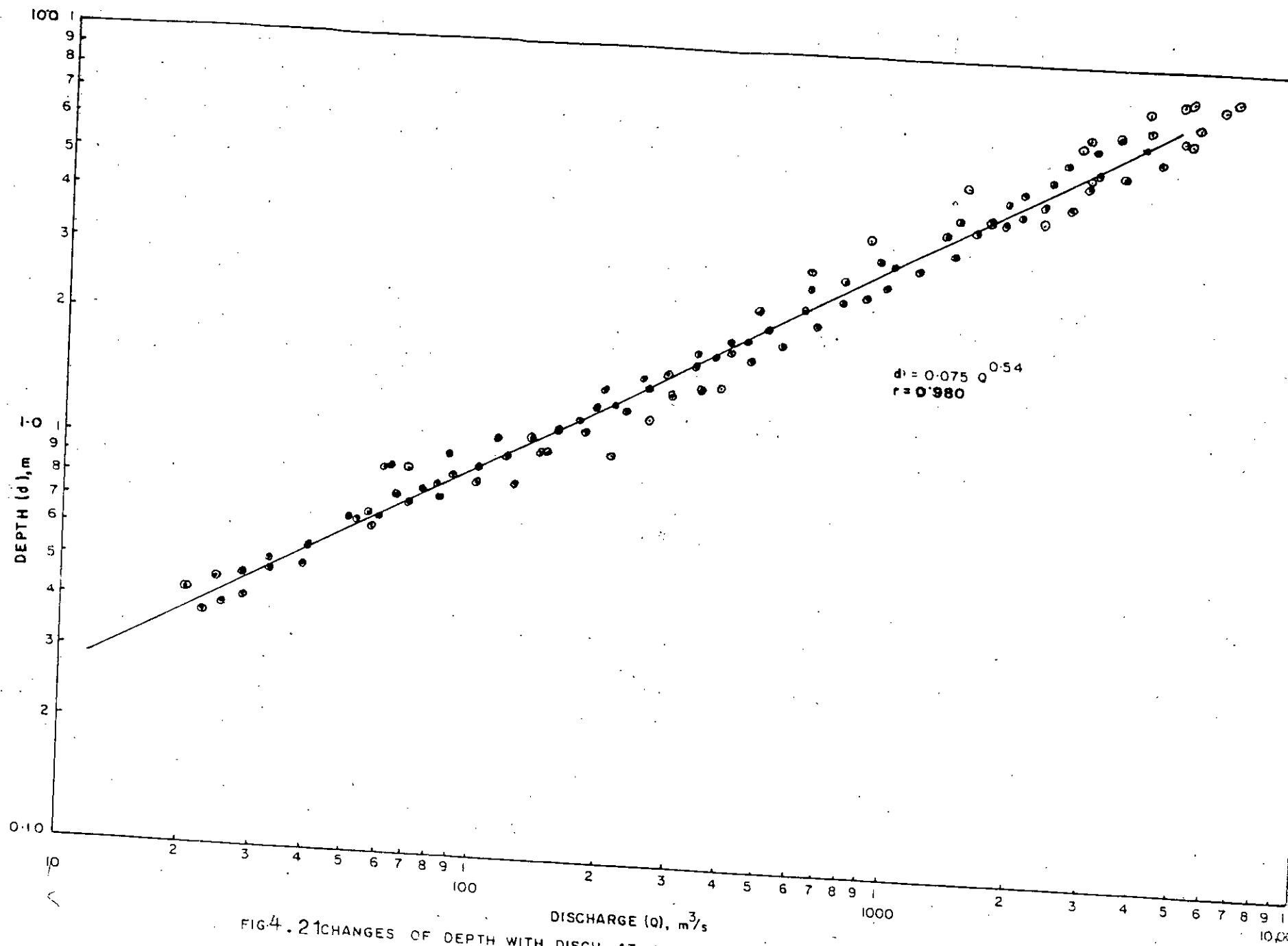


FIG. 4.21 CHANGES OF DEPTH WITH DISCH. AT - A - RIVER CROSS SECTION,  
 GORAI RLY. BRIDGE, GORAI-MADHUMATI RIVER

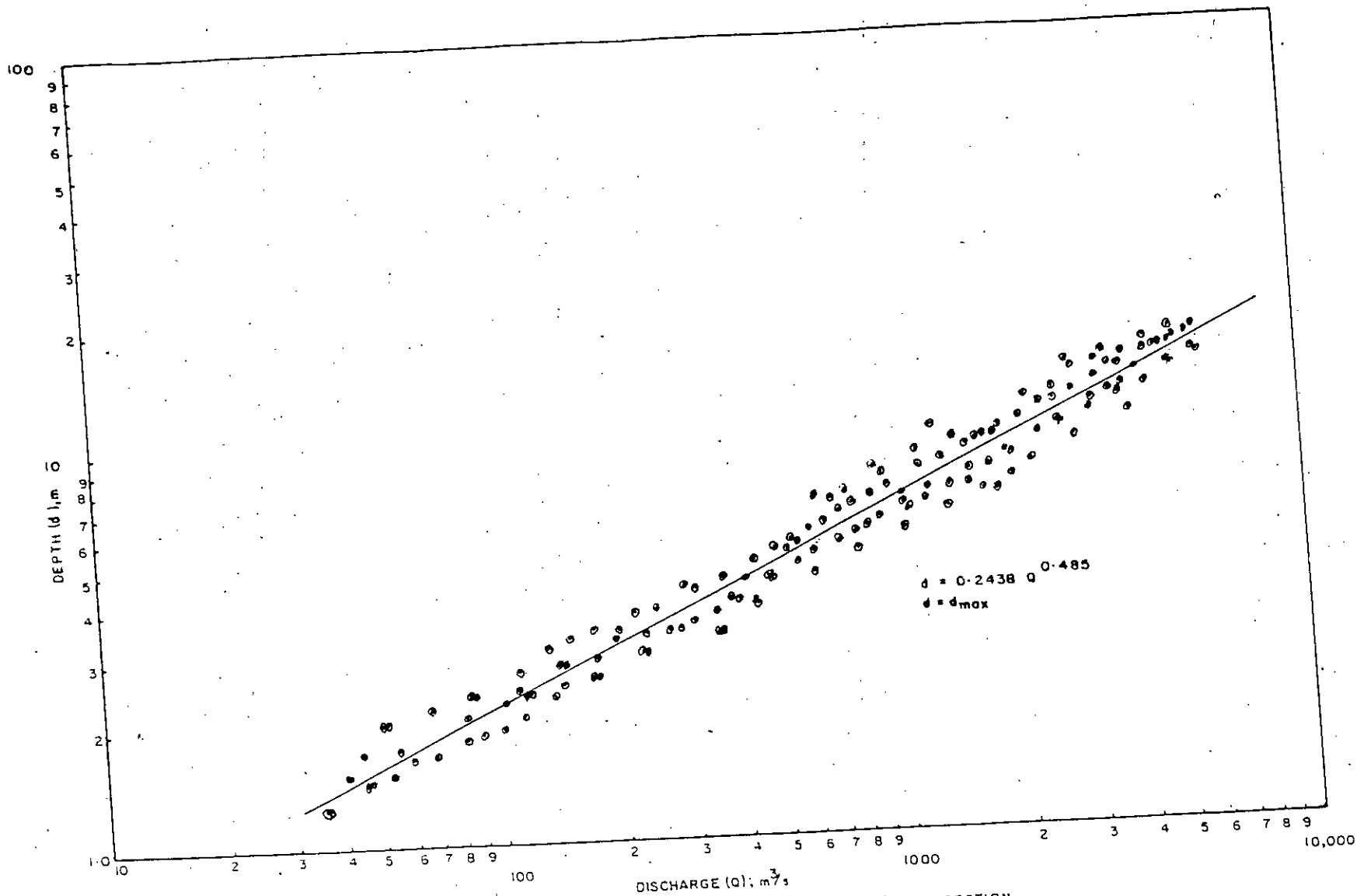


FIG. 4.22 CHANGES OF DEPTH WITH DISCH. AT-A-RIVER CROSS SECTION  
 GORAI RLY. BRIDGE, GORAI-MADHUMATI RIVER.



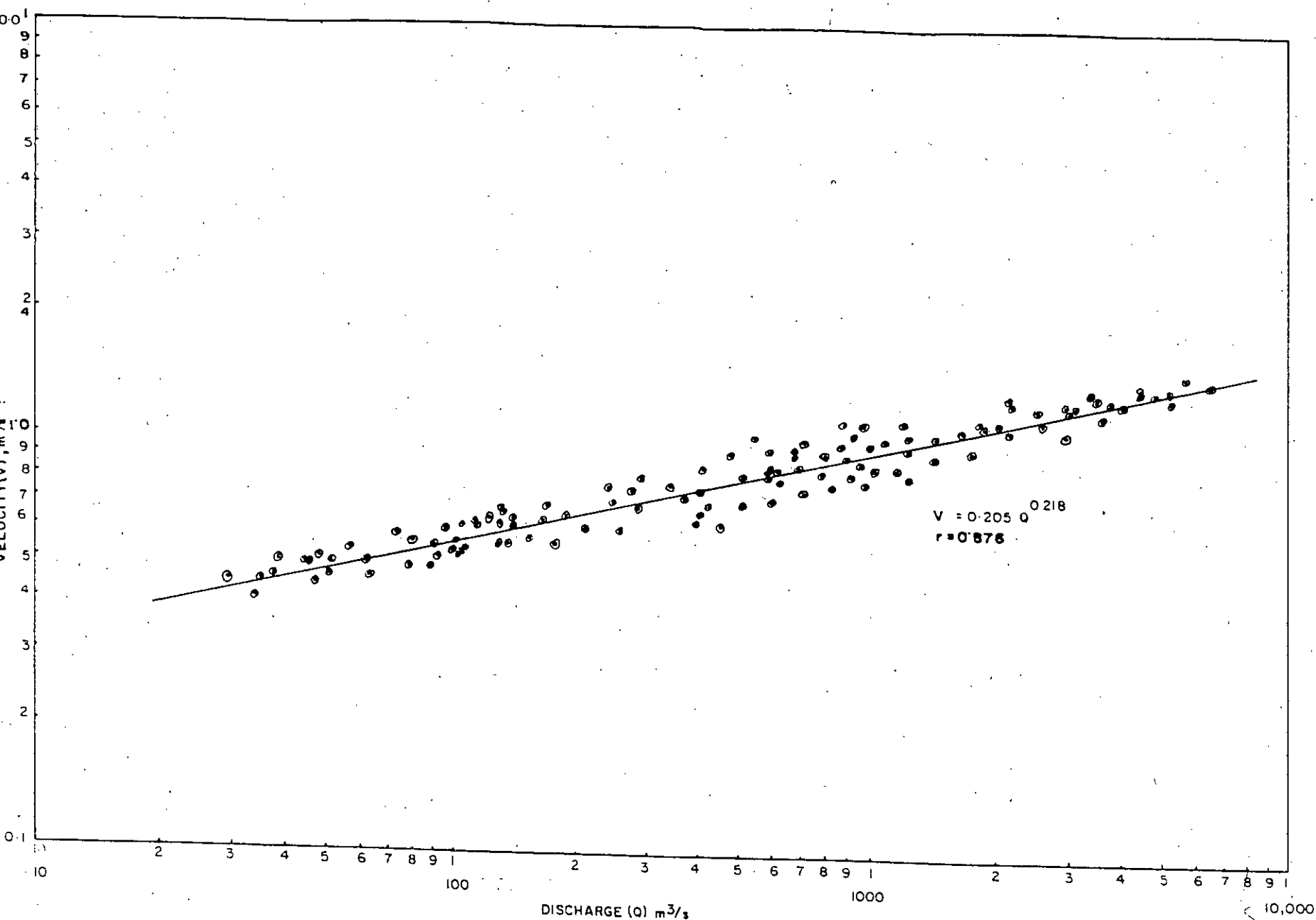


FIG4. 23 CHANGES OF VEL. WITH DISCH. AT - A - RIVER CROSS SECTION,  
 GORAI RLY. BRIDGE, GORAI-MADHUMATI RIVER

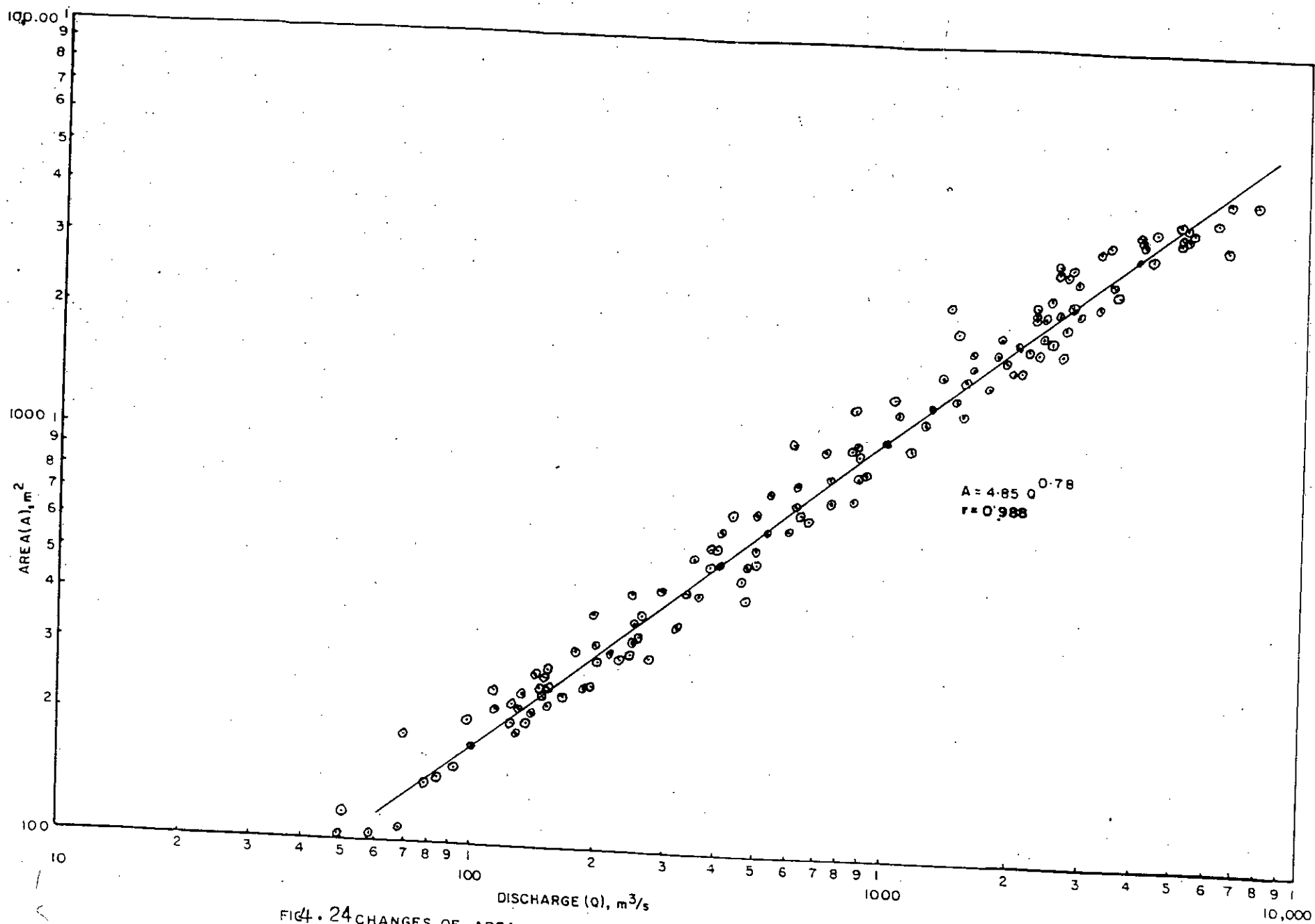


FIG. 24 CHANGES OF AREA WITH DISCH. AT-A- RIVER CROSS SECTION  
 GORAI RLY. BRIDGE, GARAI MADHUMATI RIVER.

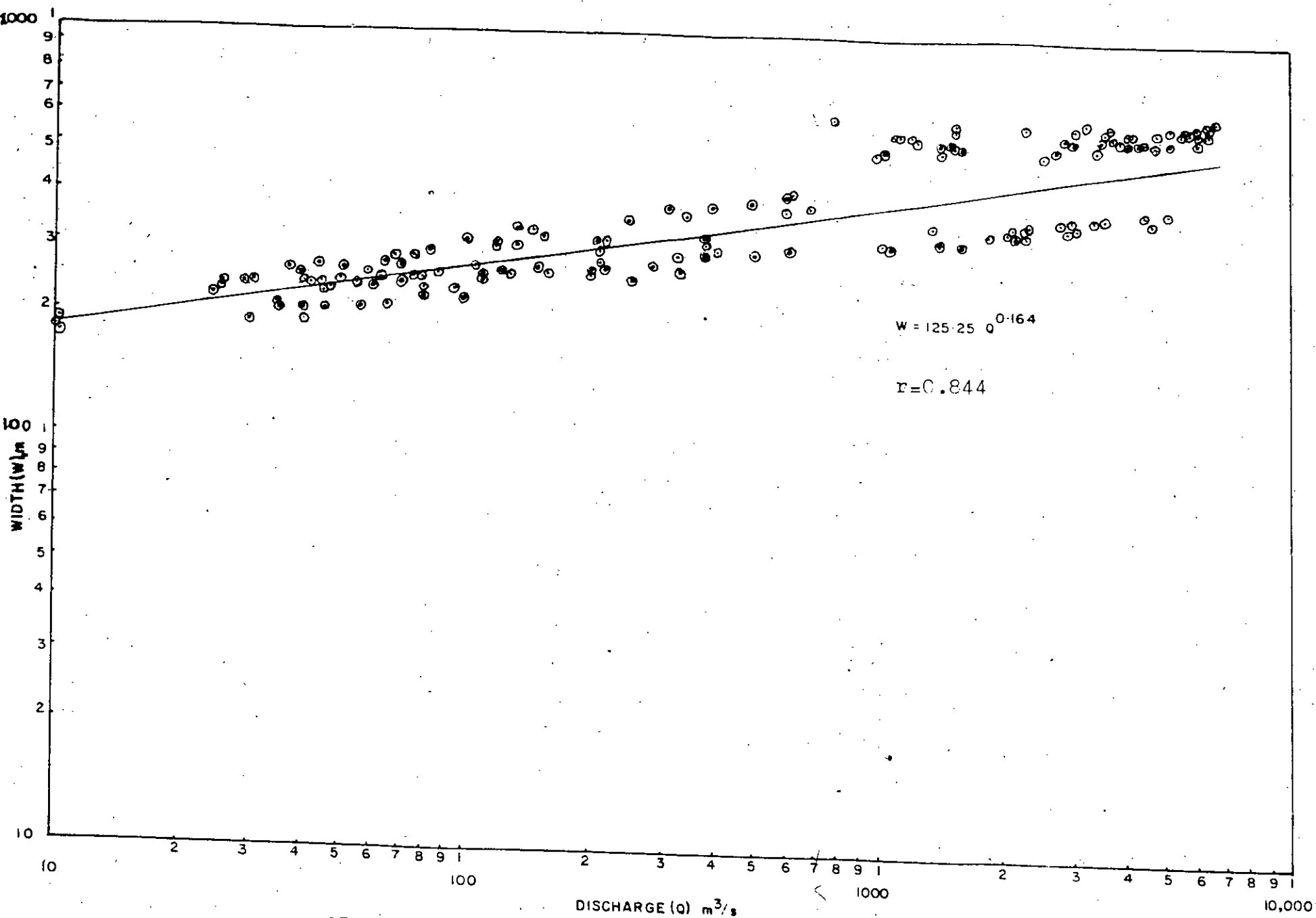


FIG. 4.25 CHANGES OF WIDTH WITH DISCH. AT - A - RIVER CROSS SECTION  
 KAMARKHALI TRANSIT, GORAI-MADHUMATI RIVER.

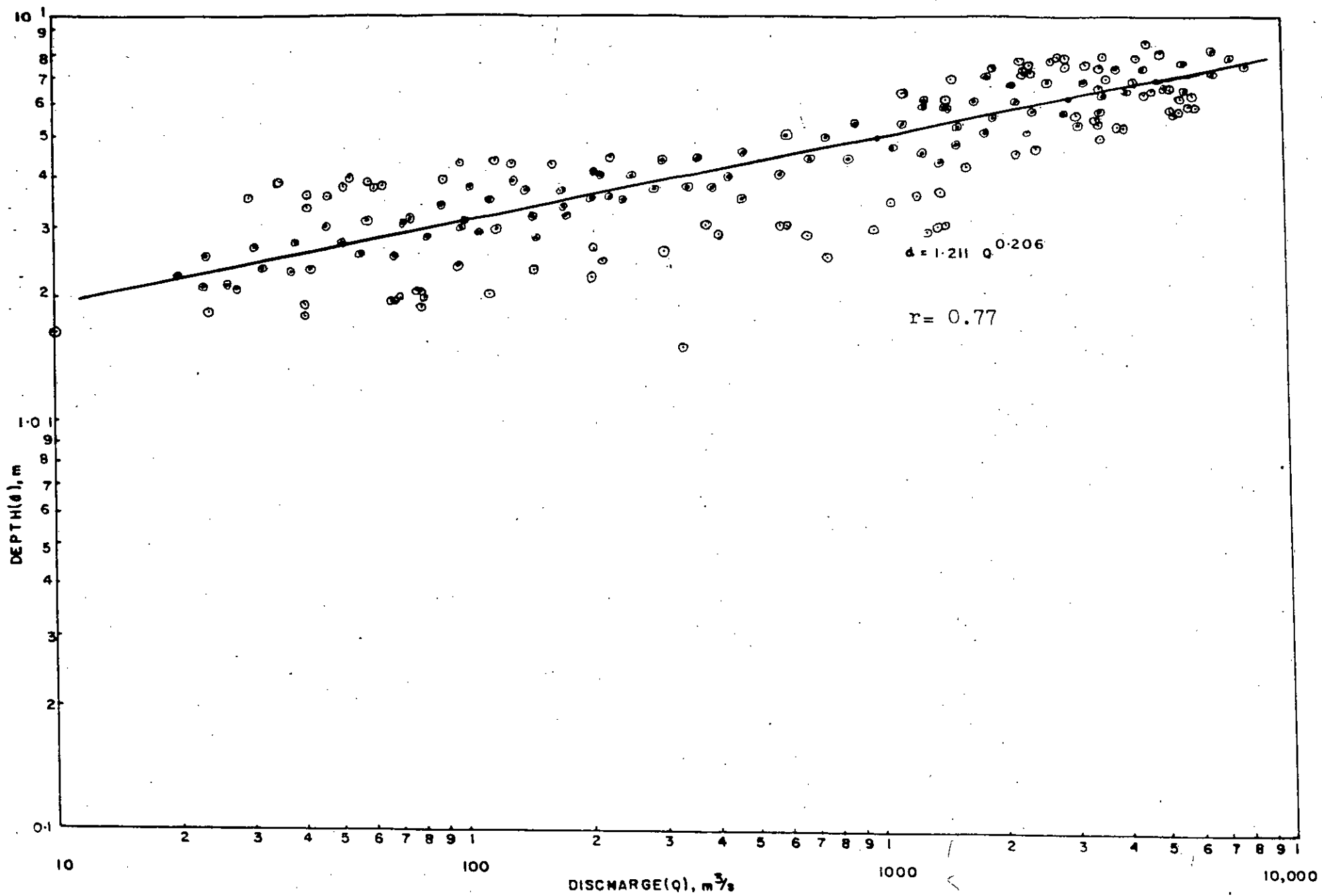


FIG 4.26 CHANGES OF MEAN DEPTH WITH DISCHARGE AT A RIVER CROSS SECTION  
 KAMARKHALI TRANSIT, GORAI-MADHUMATI RIVER.

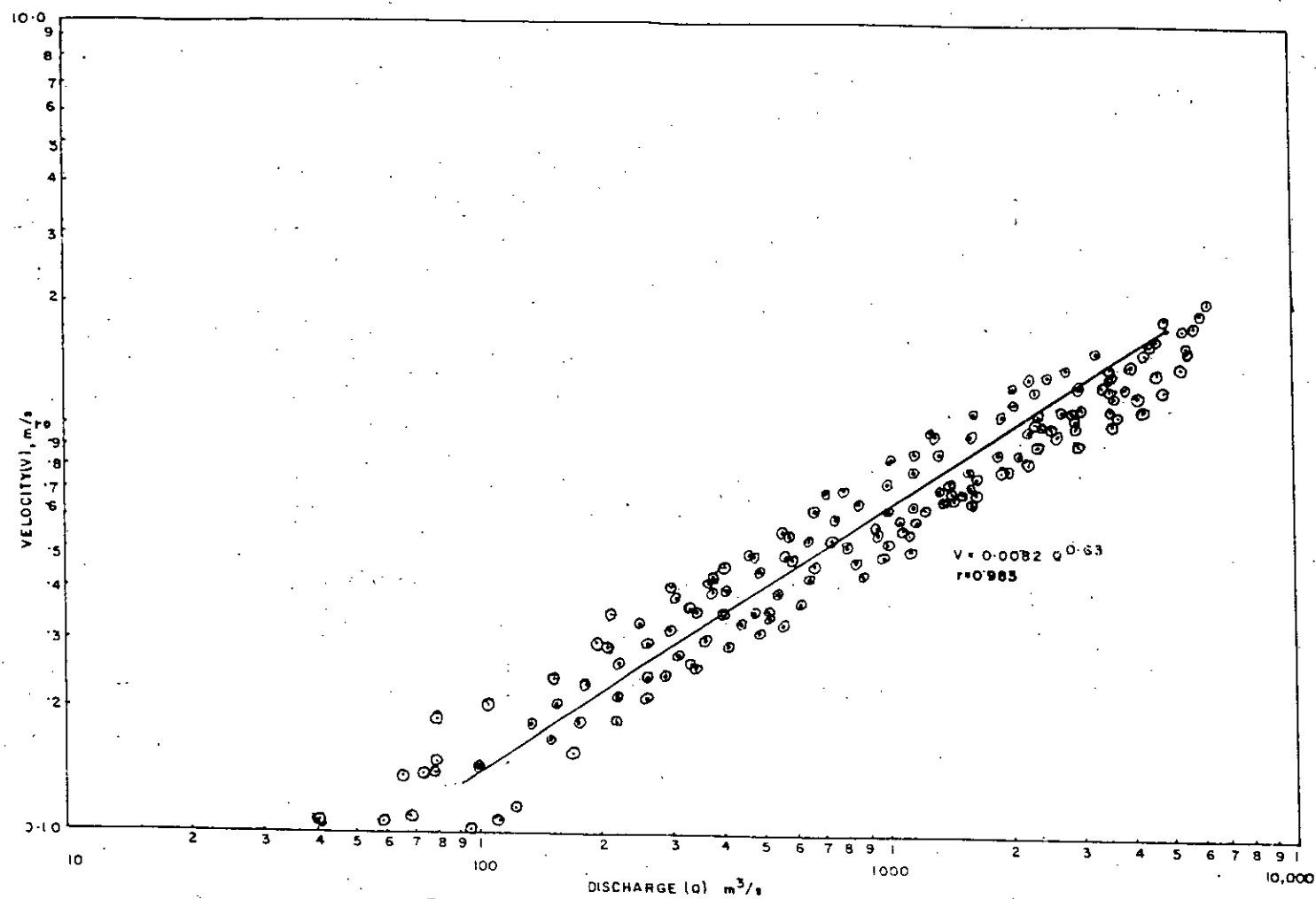


FIG. 4. 27 CHANGES OF MEAN VELOCITY WITH DISCH. AT A- RIVER CROSS SECTION  
KAMARKHALI TRANSIT, GORAI-MADHUMATI RIVER.

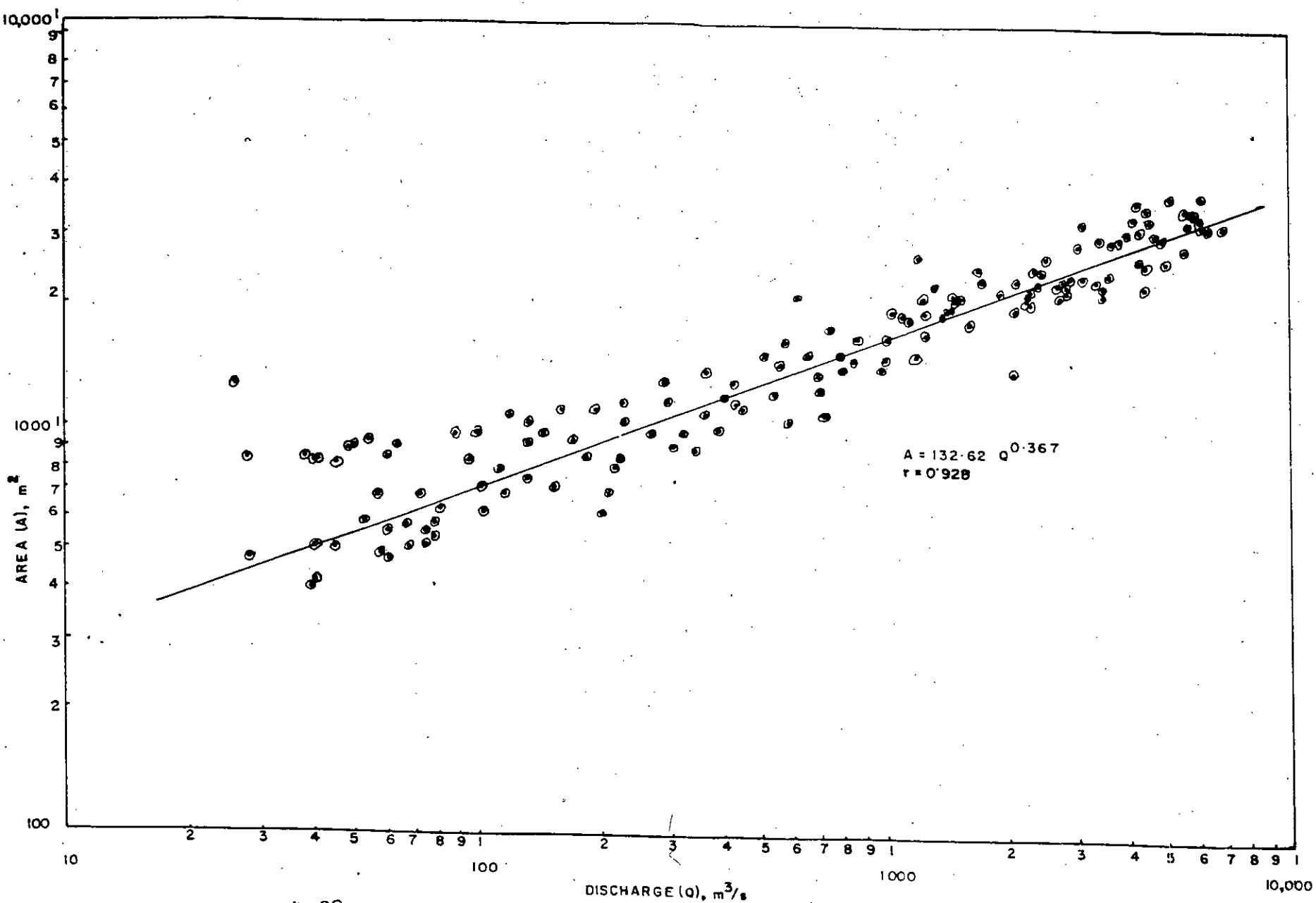


FIG. 28 CHANGES OF CROSS SECTIONAL AREA WITH DISCH. AT - A - RIVER CROSS SECTION,  
KAMARKHALI TRANSIT, GORAI-MADHUMATI RIVER.

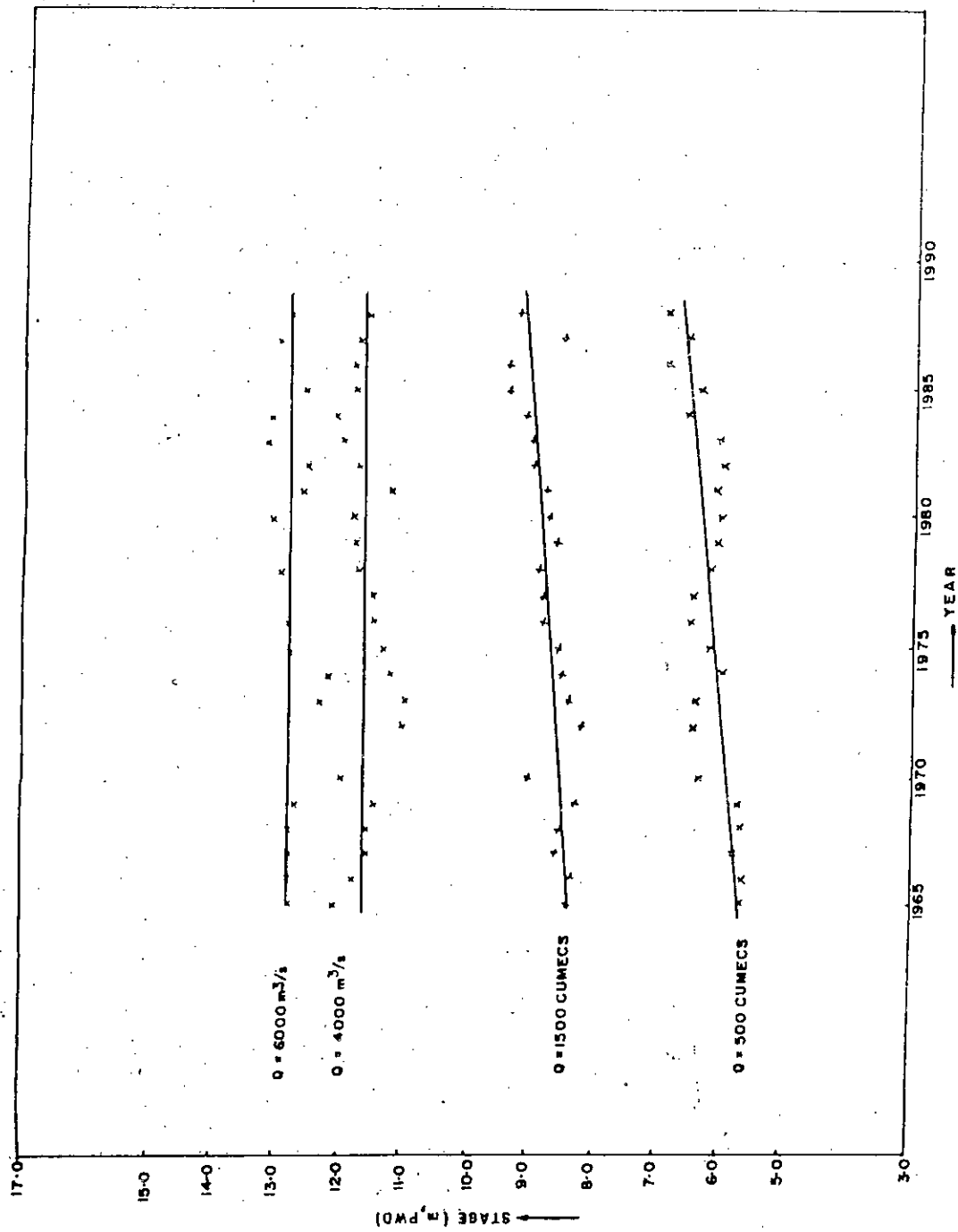


FIG. 29 WATER LEVEL VARIATION FOR FOUR DISCHARGES AT GORAL RLY BRIDGE  
FROM 1965 - 1990

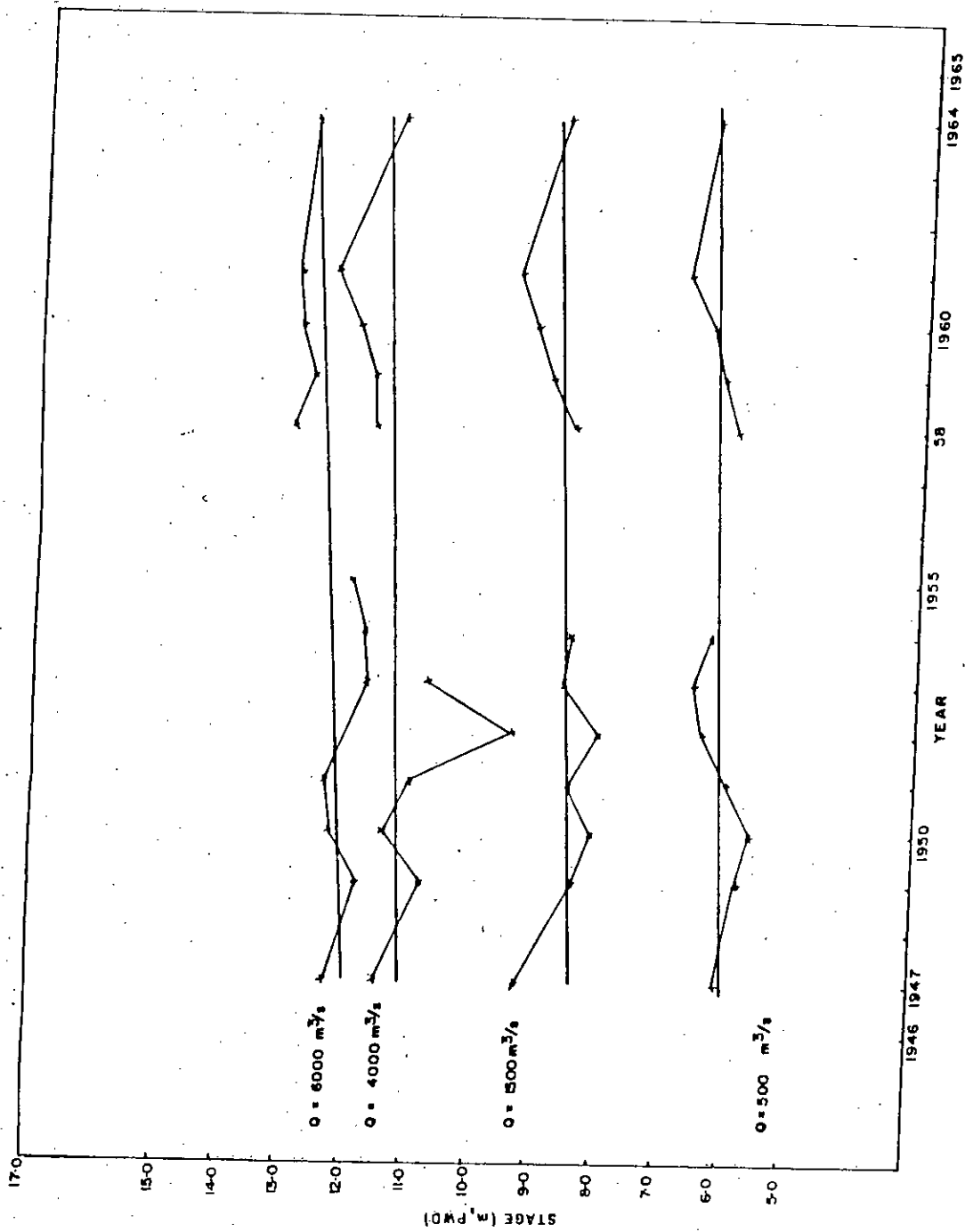


FIG. 4.30 WATER LEVEL VARIATION FOR FOUR DISCHARGES AT GORAI RLY. BRIDGE.  
FROM 1946 - 1965



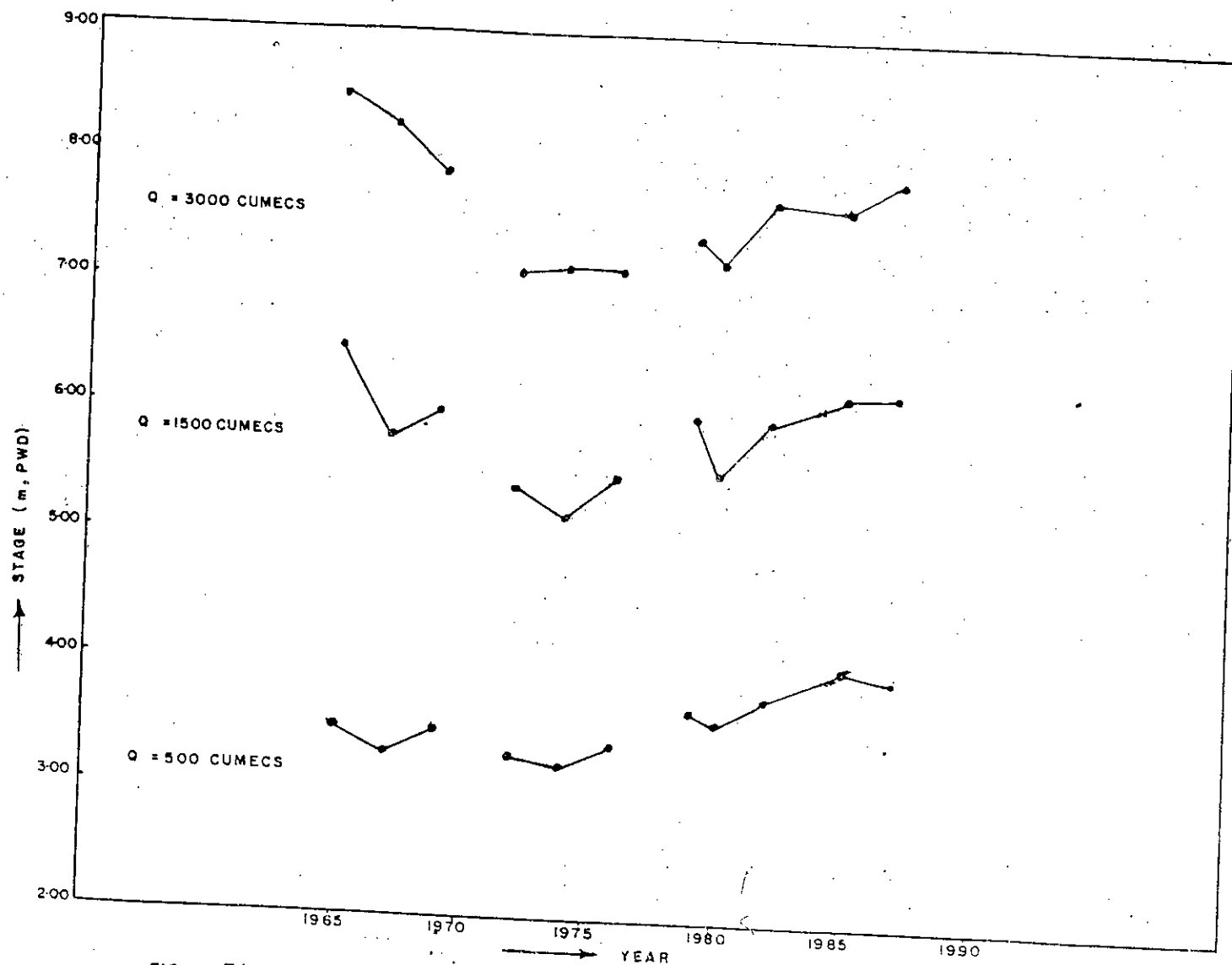


FIG. 4.31 WATER LEVEL VARIATION FOR THREE DISCHARGES AT KAMARKHALI TRANSIT FROM 1965-1990

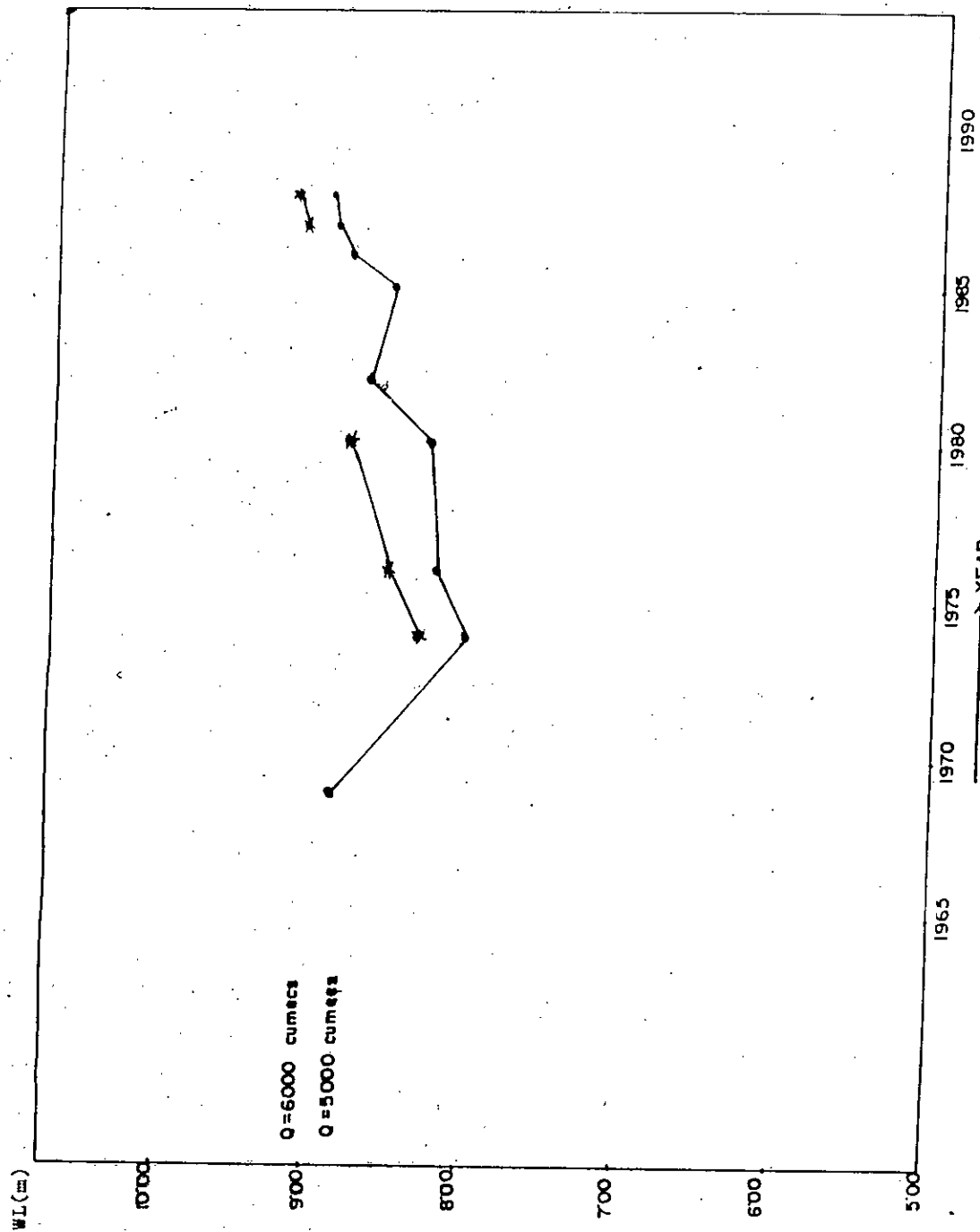


Fig 4.3 Water level variation for specific discharges at kamarkhall transit from 1965-90.

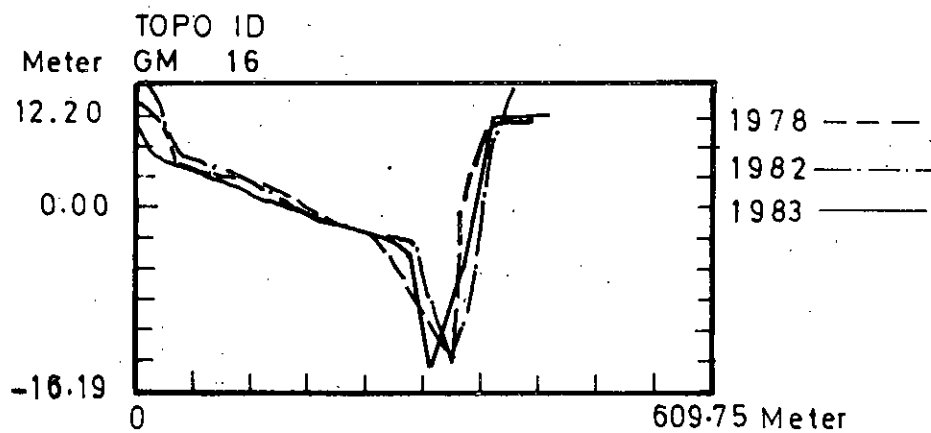
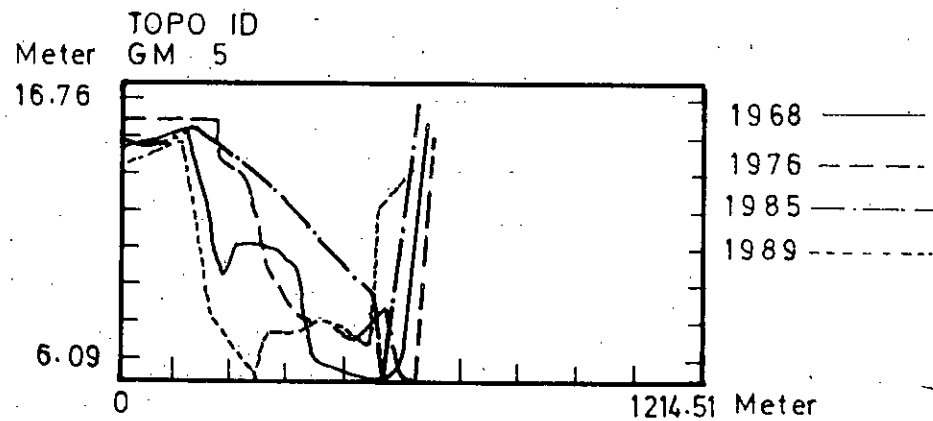
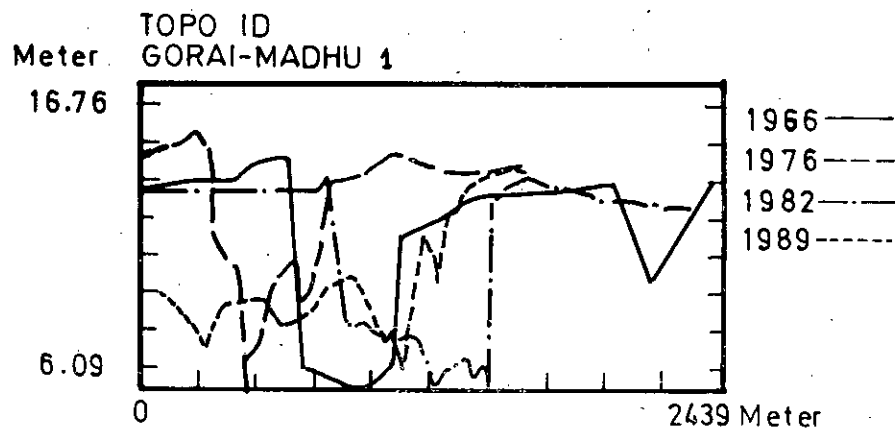


Fig.4.33 Cross-sections of the Gorai-Madhumati River  
(Of the year 1966, 1968, 1976, 1978, 1982, 1983, 1985 & 1989)

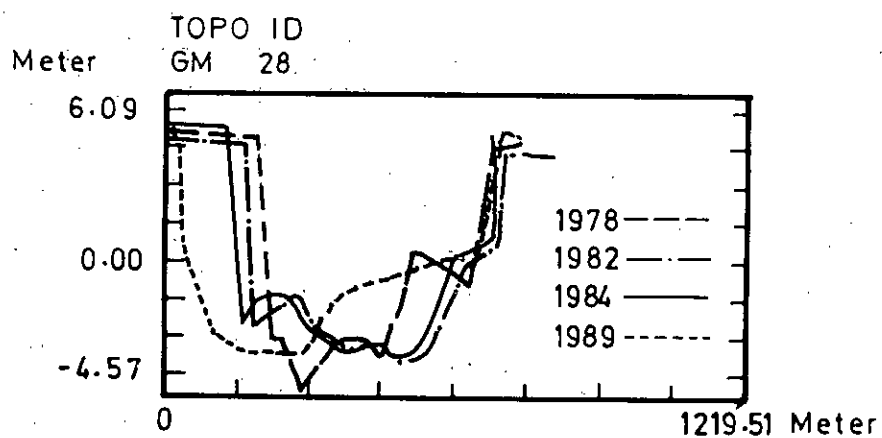
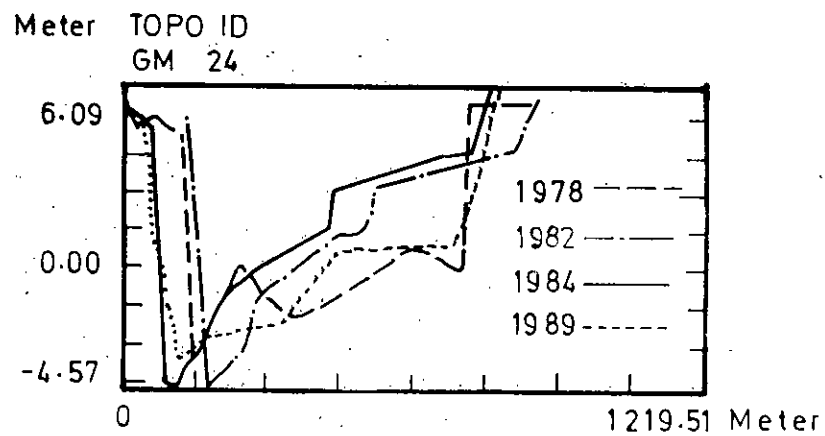


Fig. 4.34 Cross-sections of the Gorai-Madhumati River  
(Of the year 1978, 1982, 1984 & 1989)

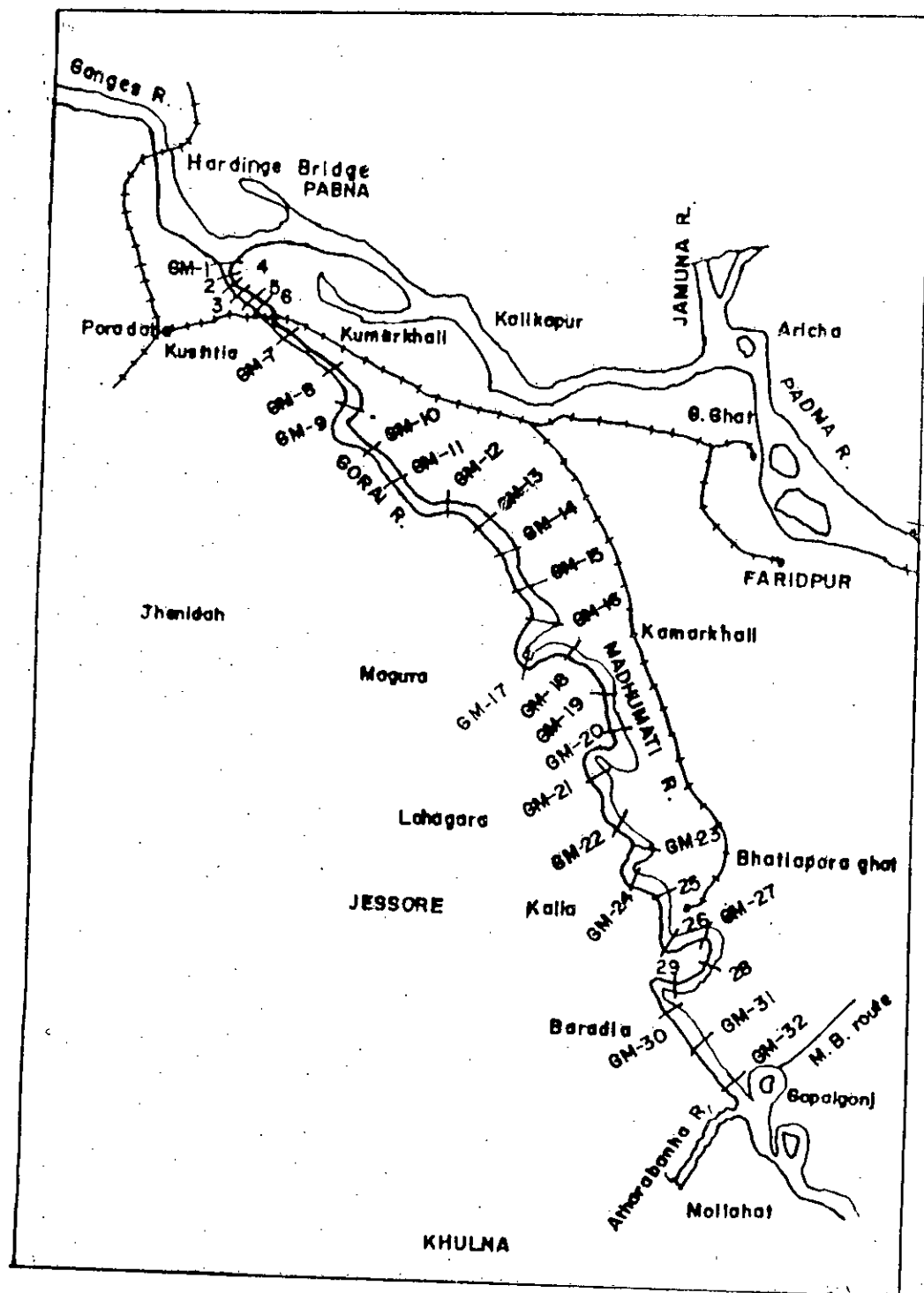


FIG. 4.35 LOCATION OF CROSS-SECTIONS, BORAL-MADHUMATI RIVER.  
(Not to scale)  
(SOURCE - BWDB)

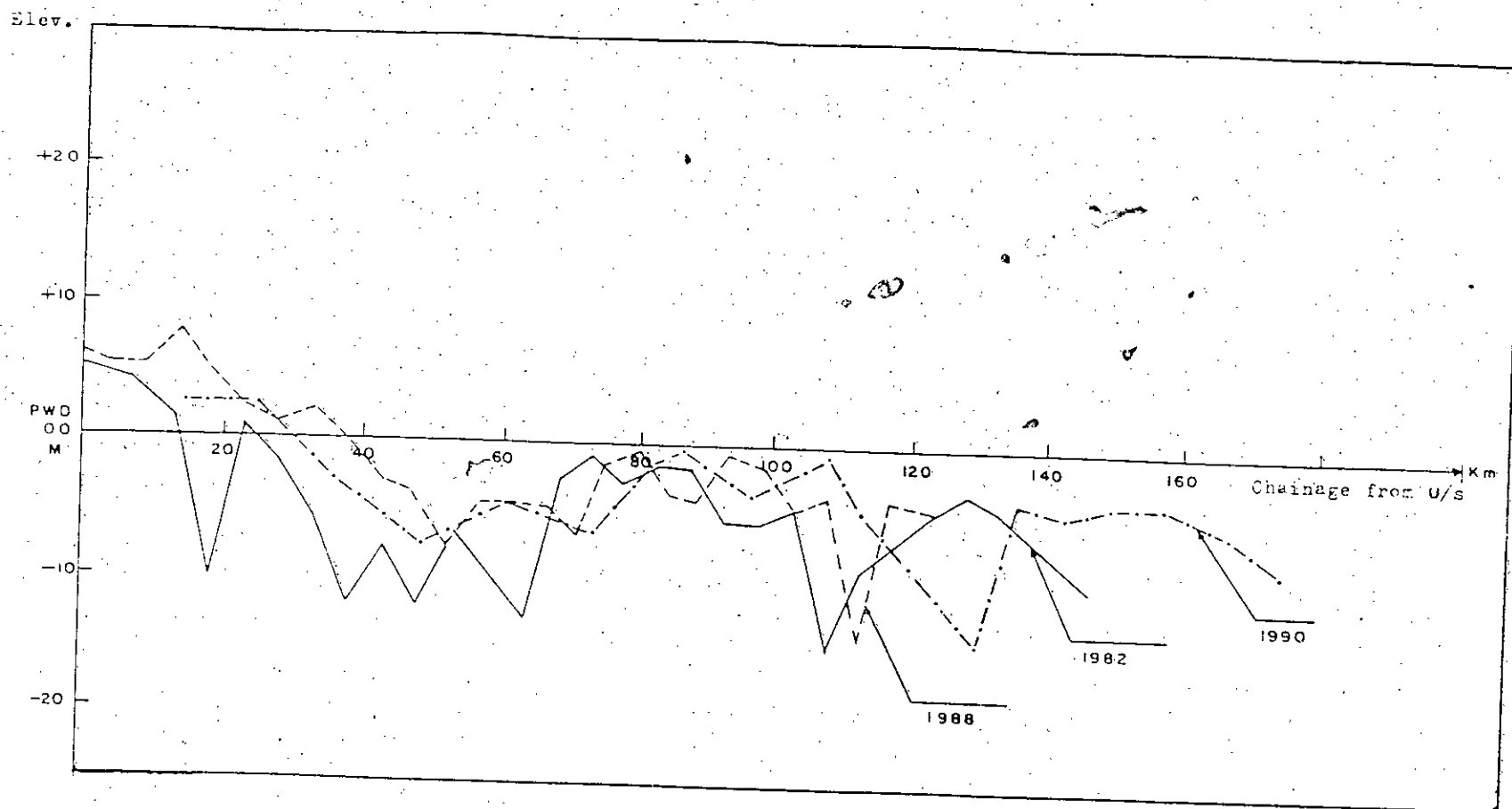


FIG. 4.36 BED PROFILE ALONG THALWEG  
GORAI-MADHUMATI RIVER

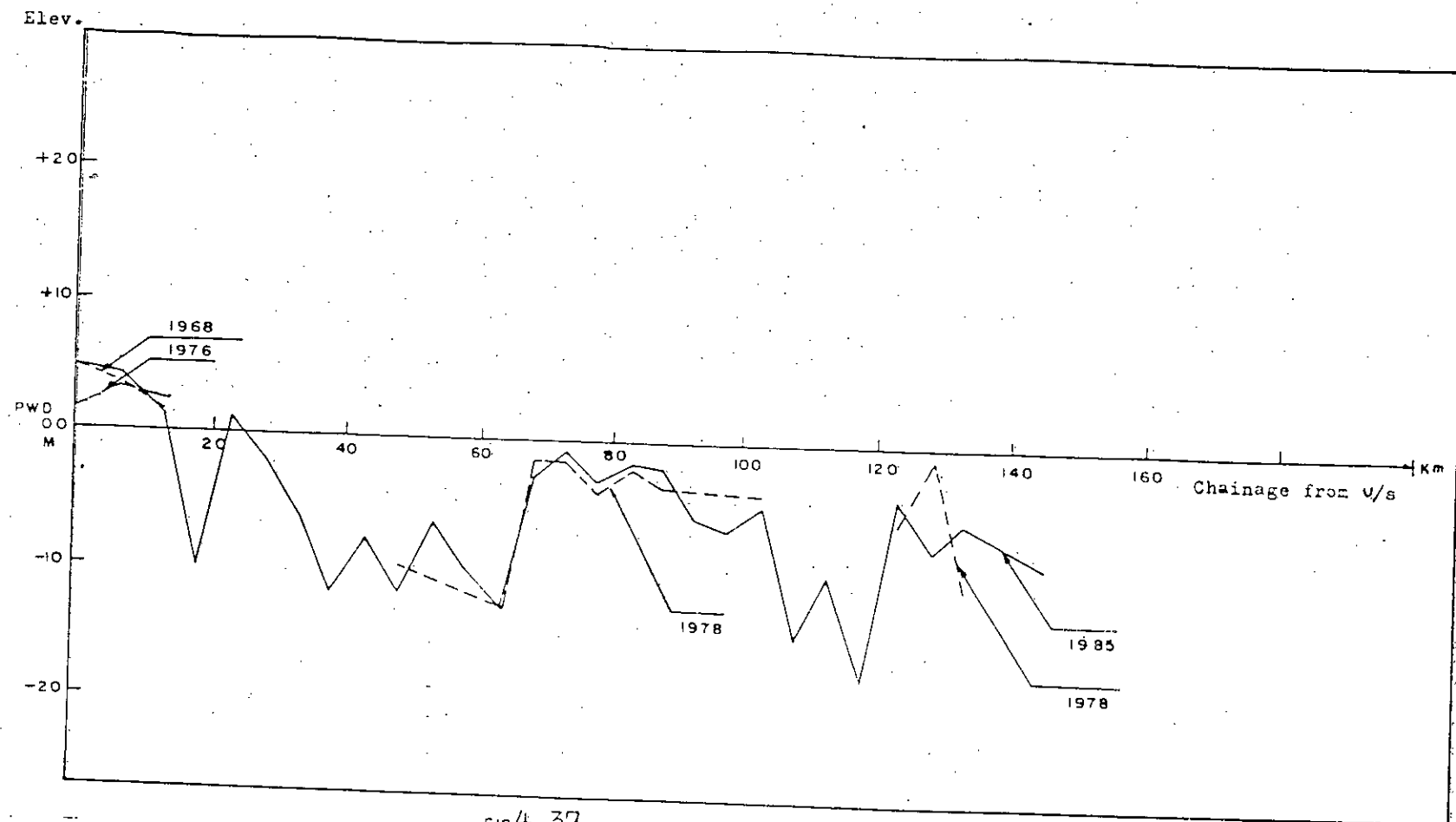


FIG. 4.37 BED PROFILE ALONG THALWEG  
GORAI-MADHUMATI RIVER

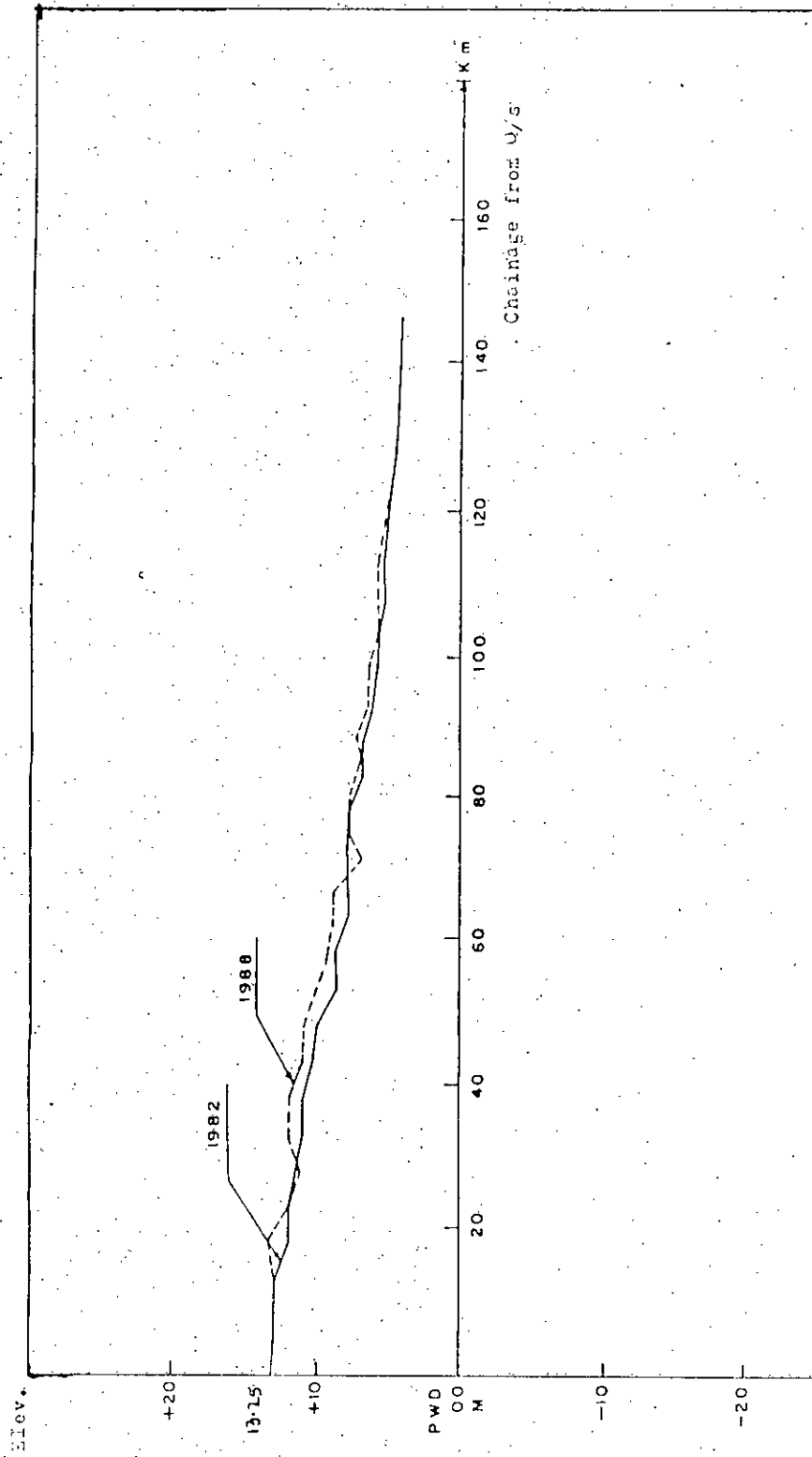


FIG 28 BANK LINE PROFILE  
GORAL MAOHUMATI RIVER



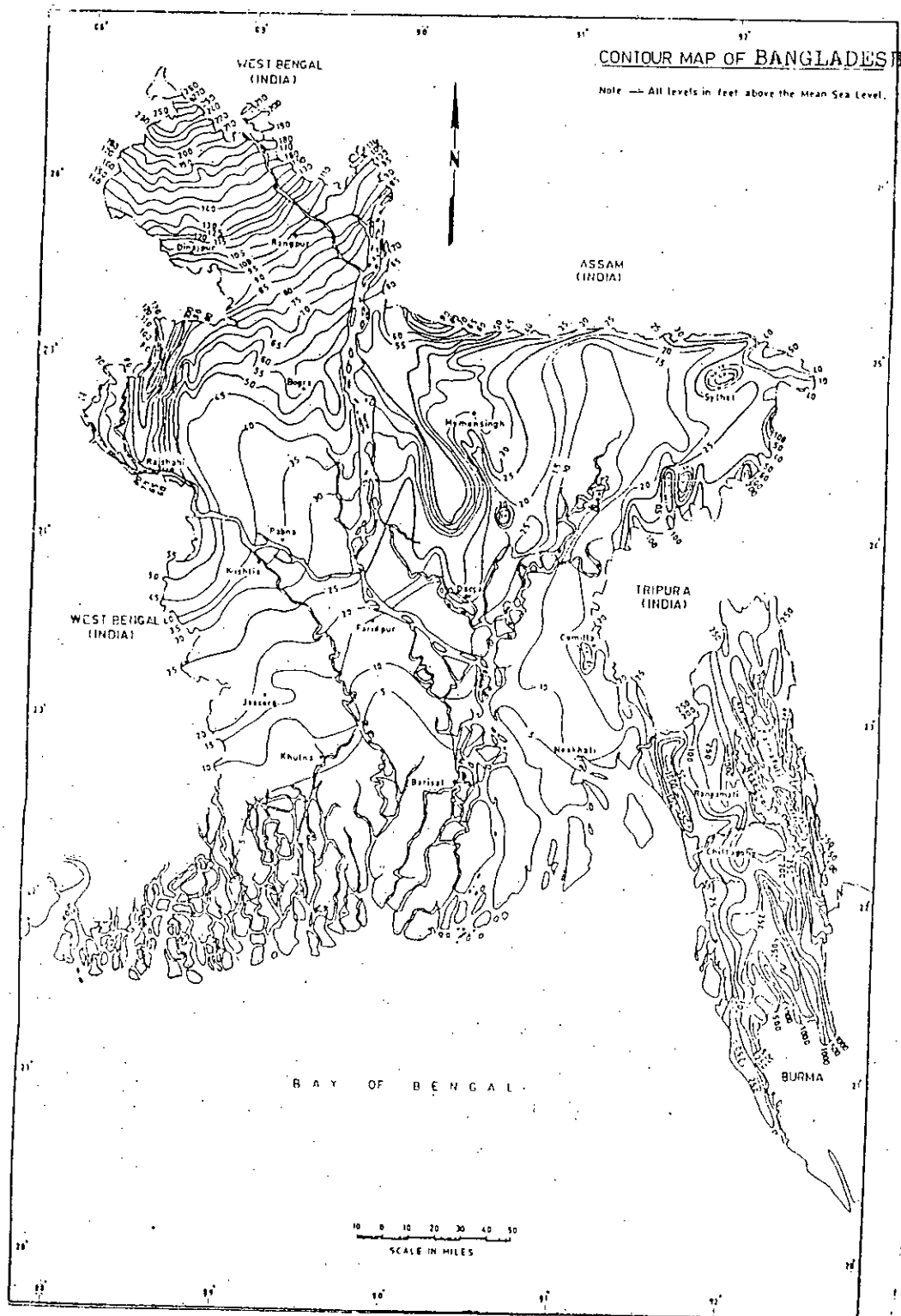


Fig. 4.39 Contour map of Bangladesh  
Source : A. M. M. Ghulam Kibria, 1967

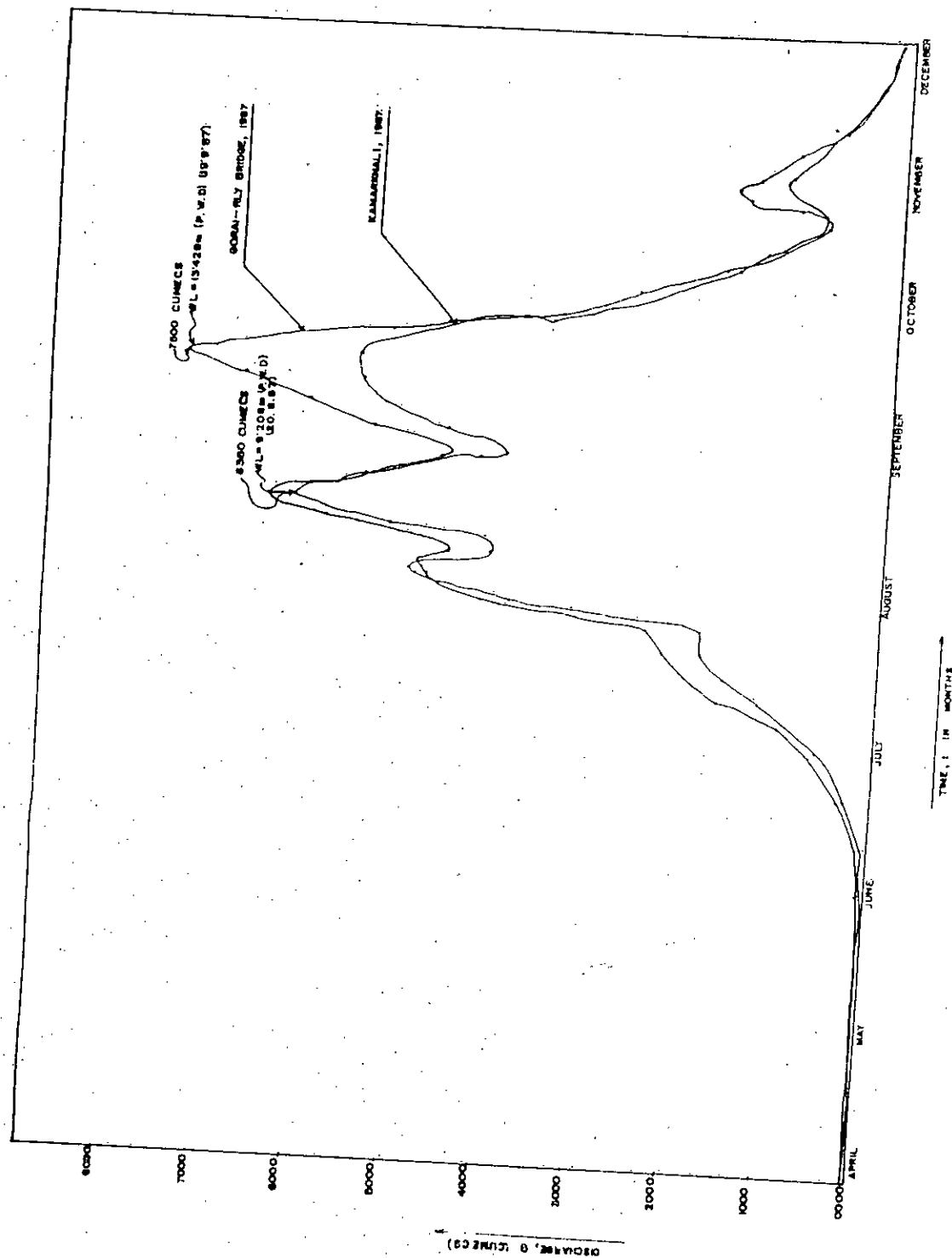


Fig. 4.40 DISCHARGE HYDROGRAPH FOR KORAI-MAHUMATI RIVER AT BORAI RAILWAY BRIDGE & KANARNALI TALVET (1957)

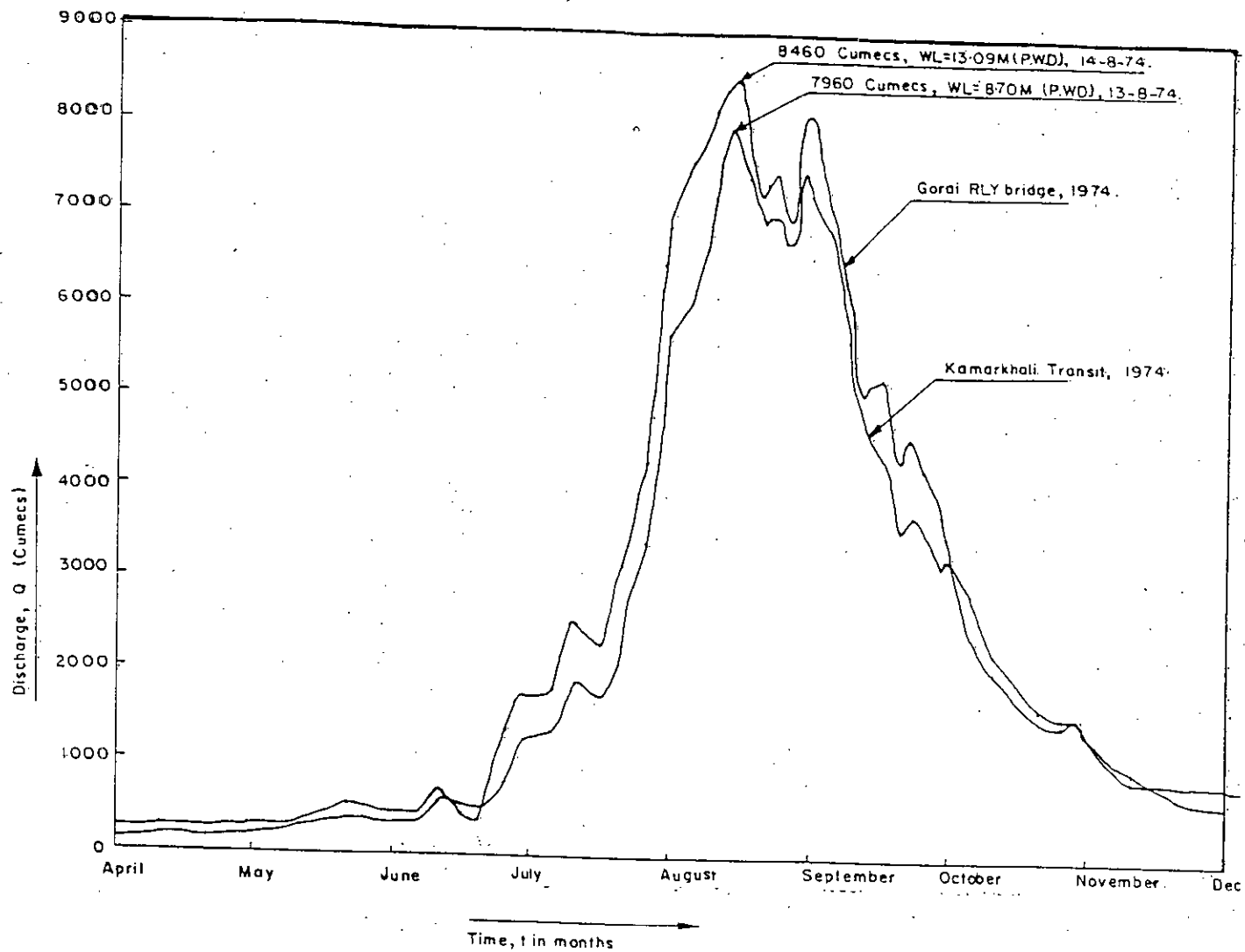


Fig 4.41 Discharge hydrographs for Garai-Madhumati river at Gorai Rly bridge & Kamarkhali transit (1974)

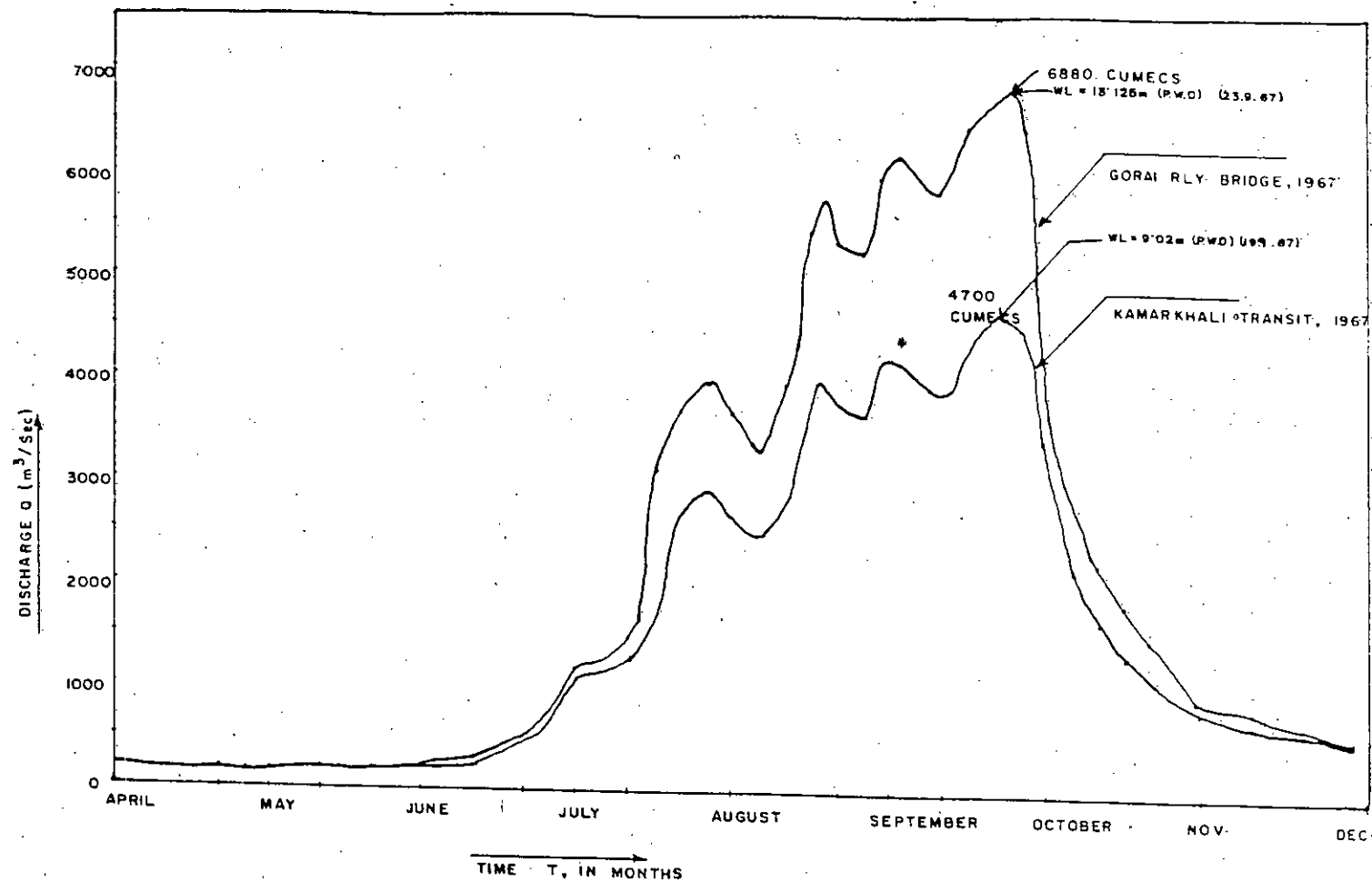


FIG. 4.42 DISCHARGE HYDROGRAPHS FOR GORAI-MADHUMATI RIVER  
AT GORAI RLY. BRIDGE & KAMARKHALI TRANSIT (1967)

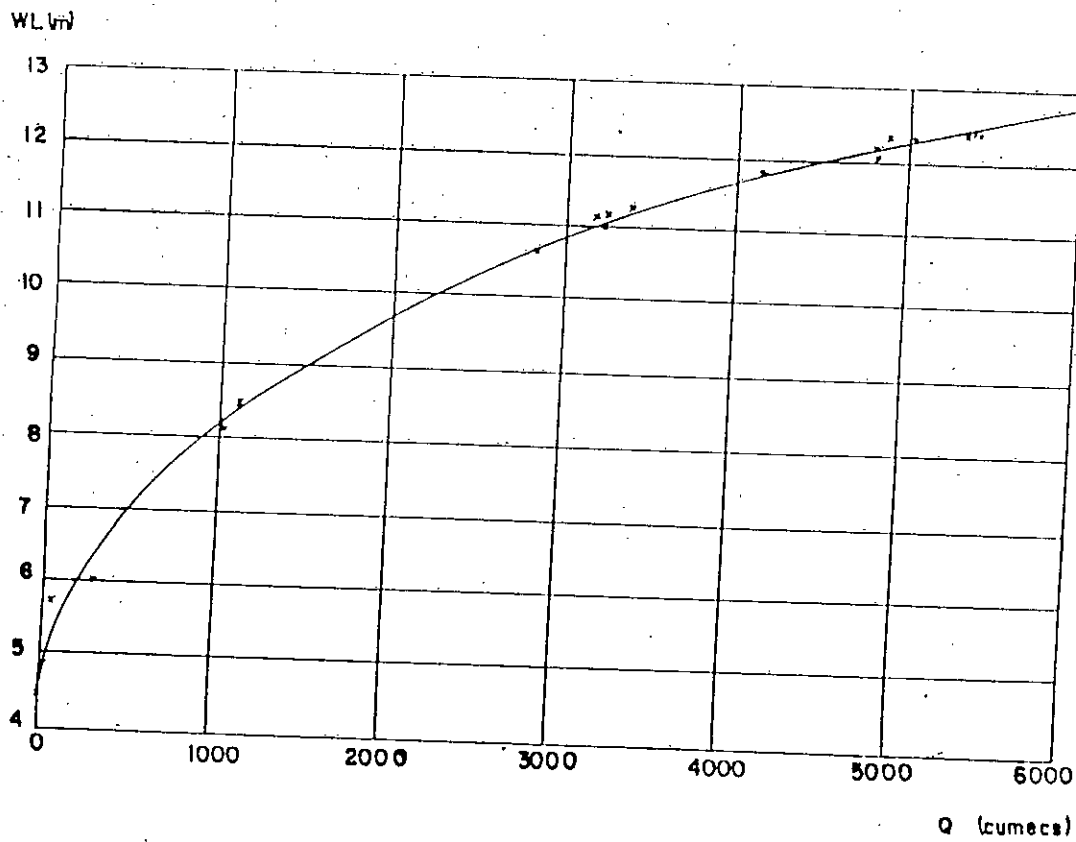


Fig. 4.43 Rating curve for the Goral-Madhumati river  
Station—Goral RLY. bridge (1990)

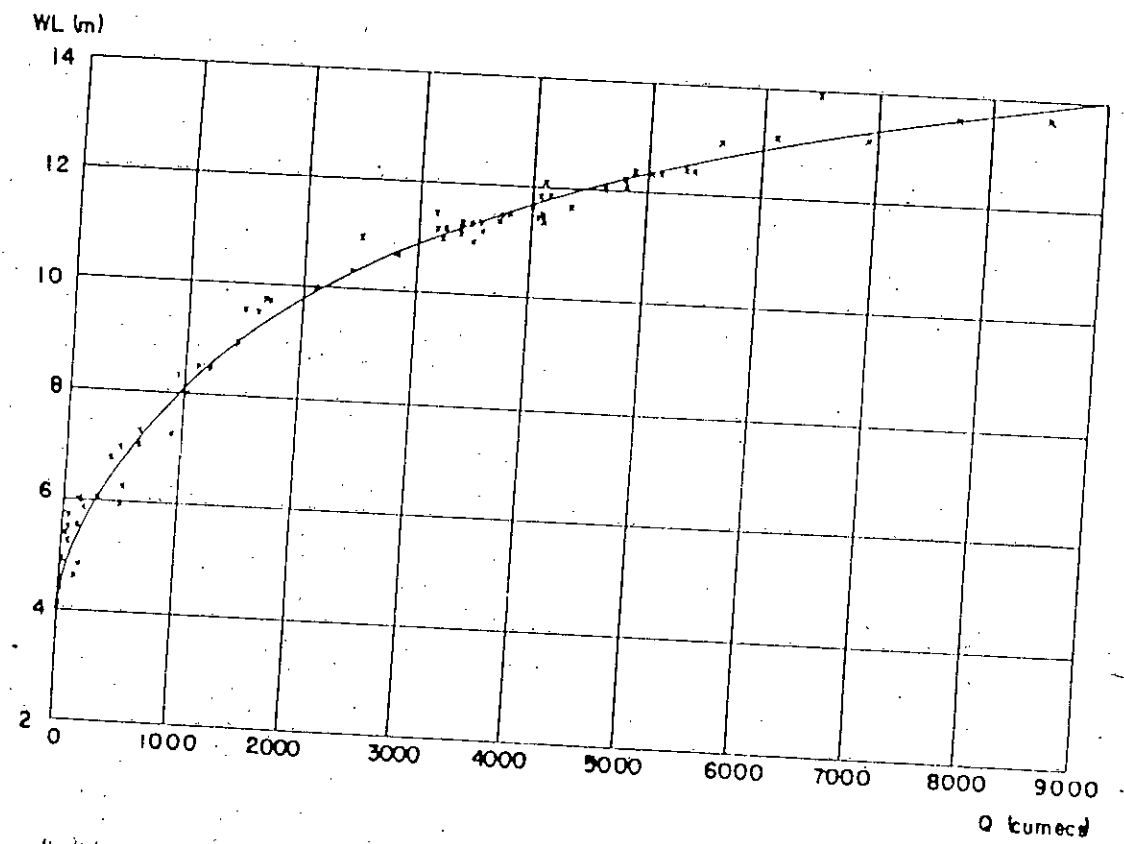


Fig. 4.44 Rating curve for the Goral-Madhumati river  
Station—Goral Rly. bridge (1988-90)

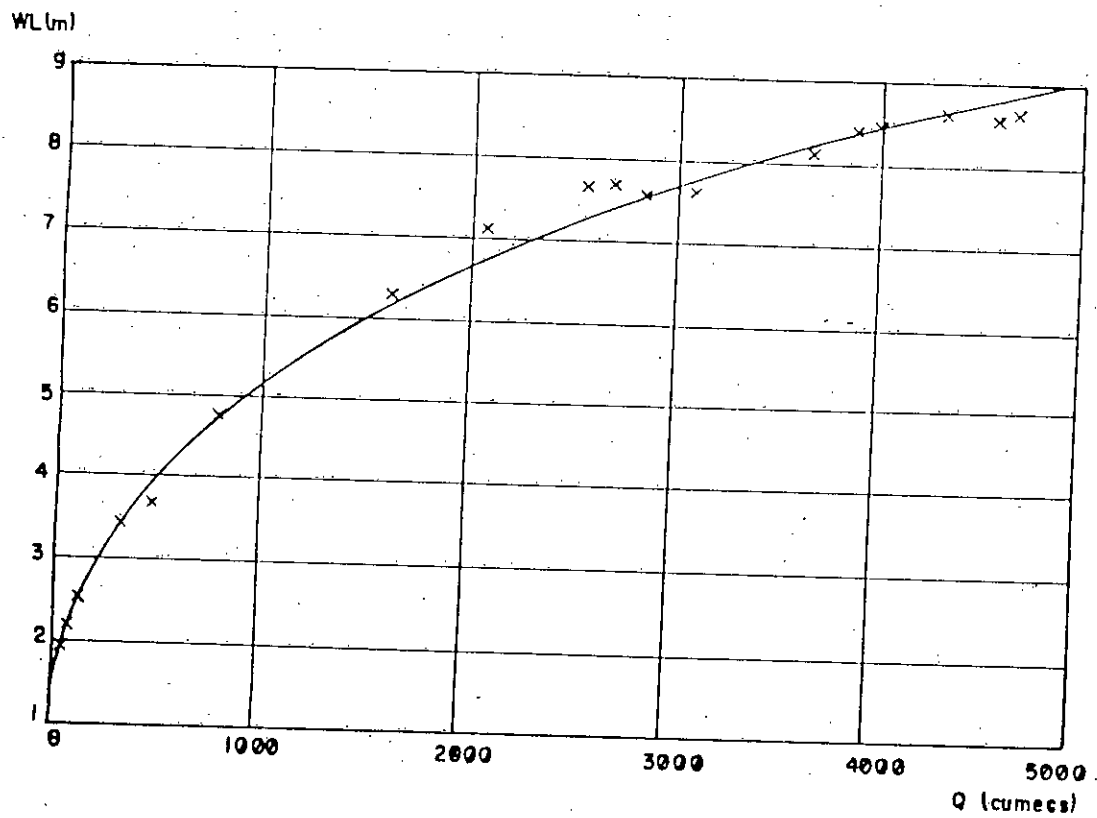
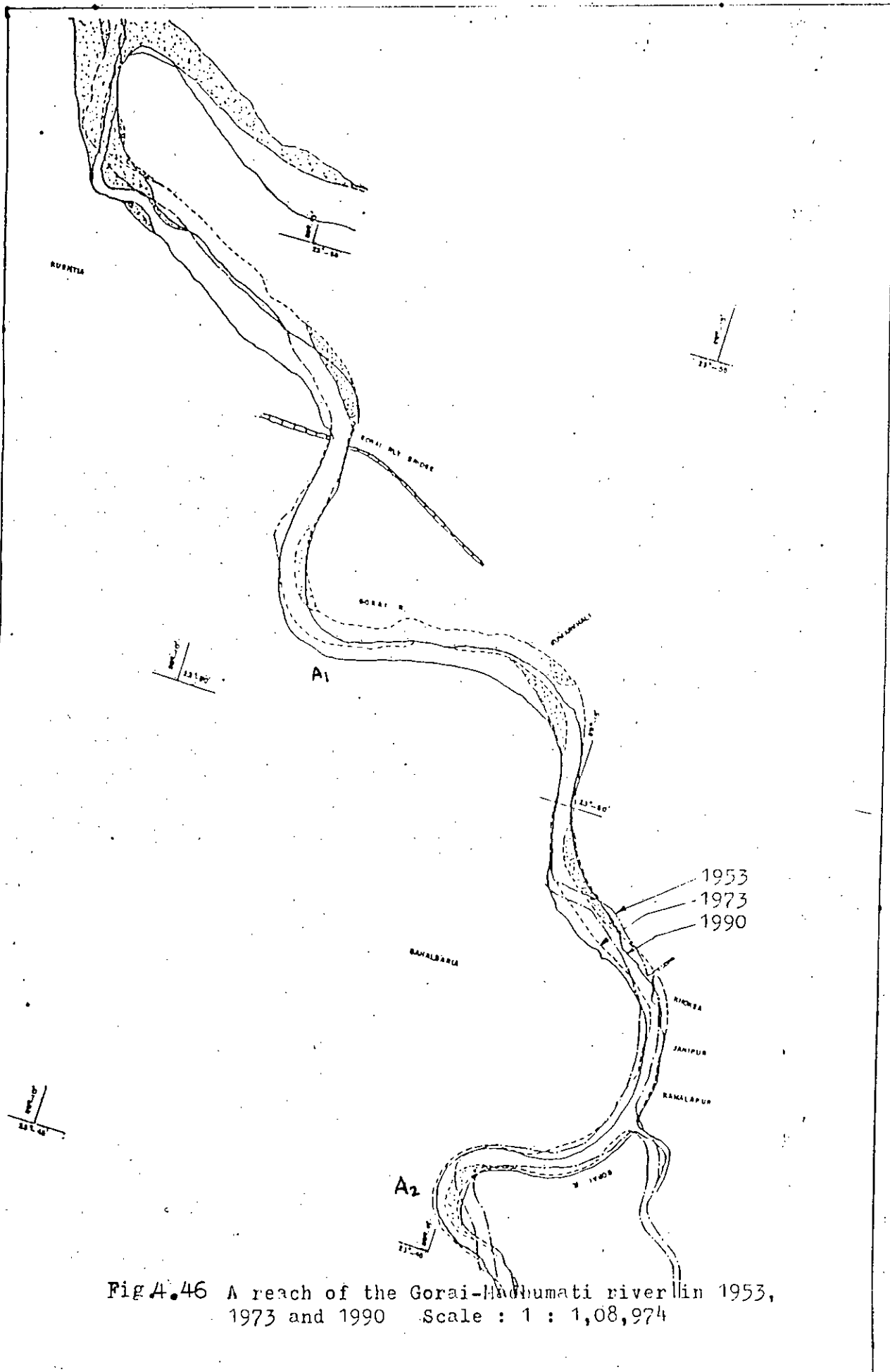


Fig. 4.45 Rating curve for the Goral-Madhumall river station-kamarkhall transit (U99Q)





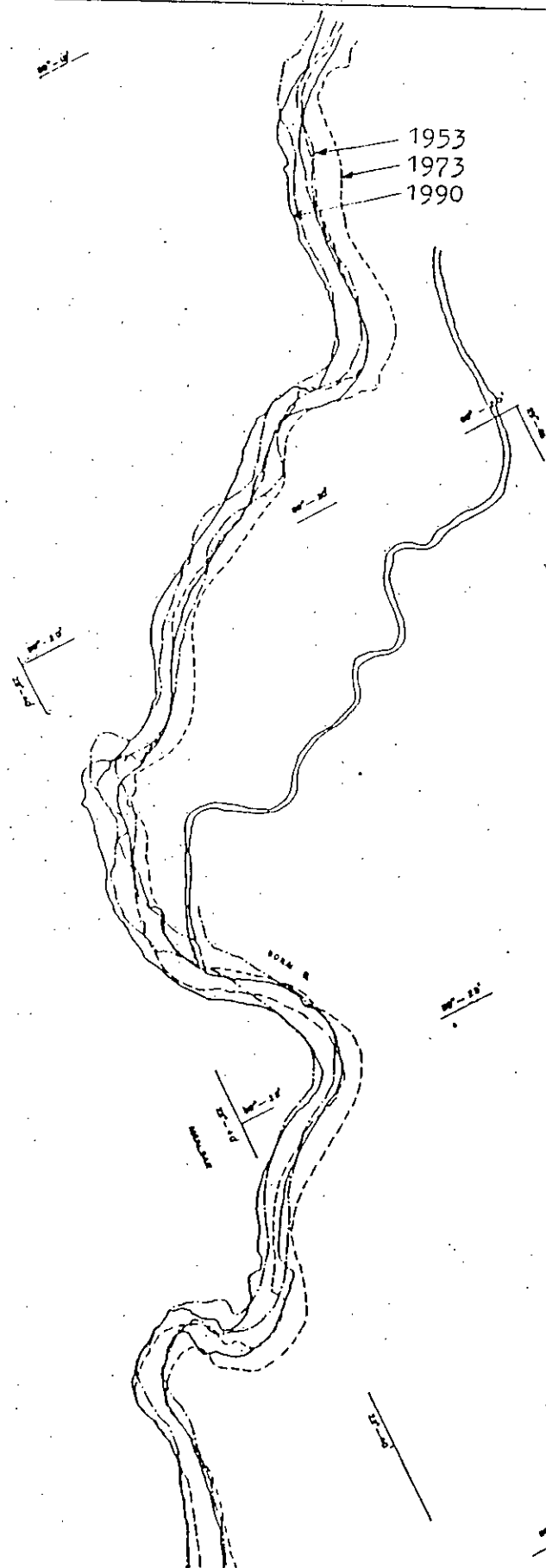


Fig.4.47 A reach of the Gorai-Madhumati river in 1953, 1973 and 1990 Scale : 1 : 1,08,974

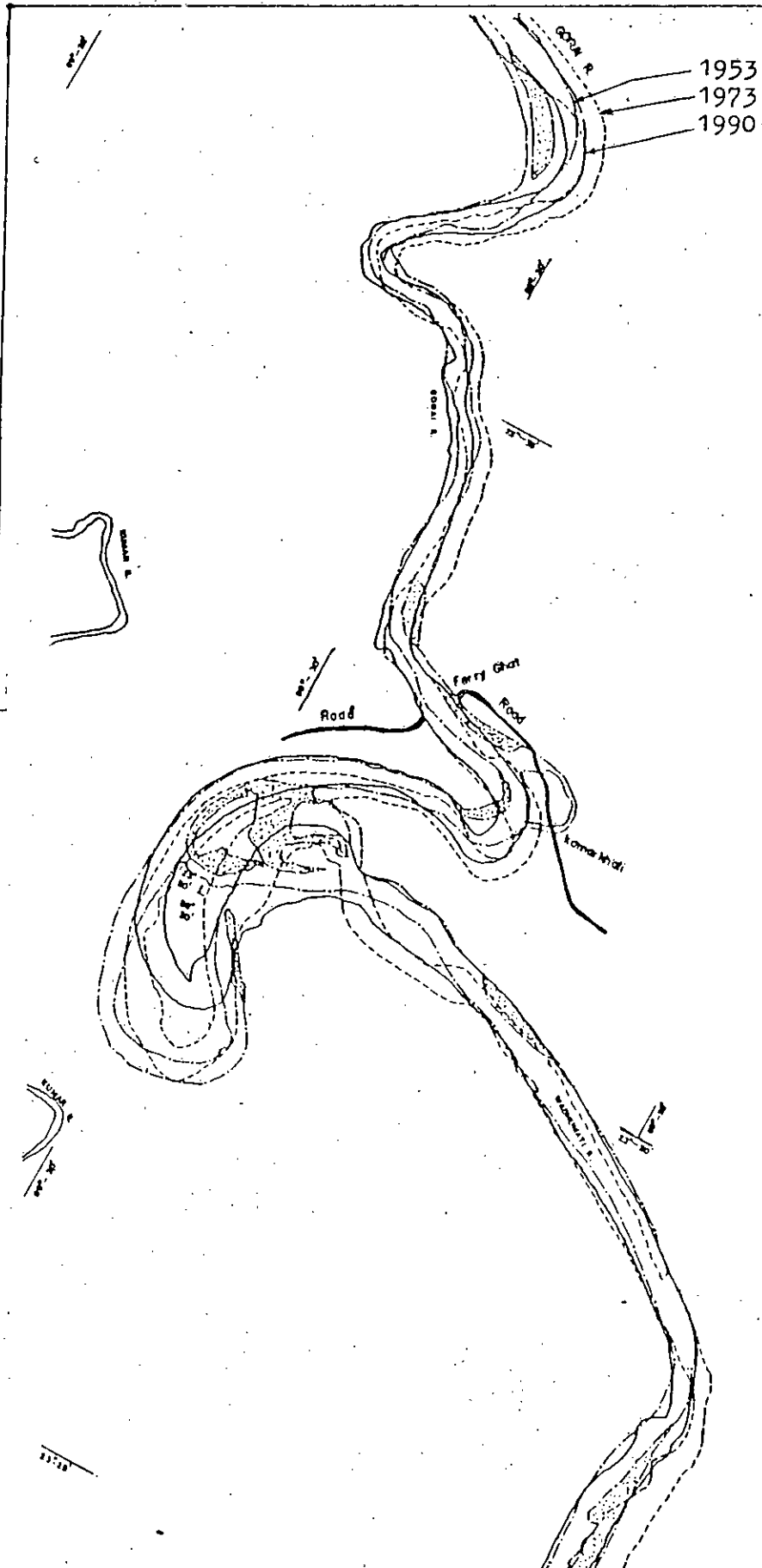


Fig. 4.48 A reach of the Gorai-Madhumati river in 1953, 1973 and 1990 Scale : 1 : 1,08,974

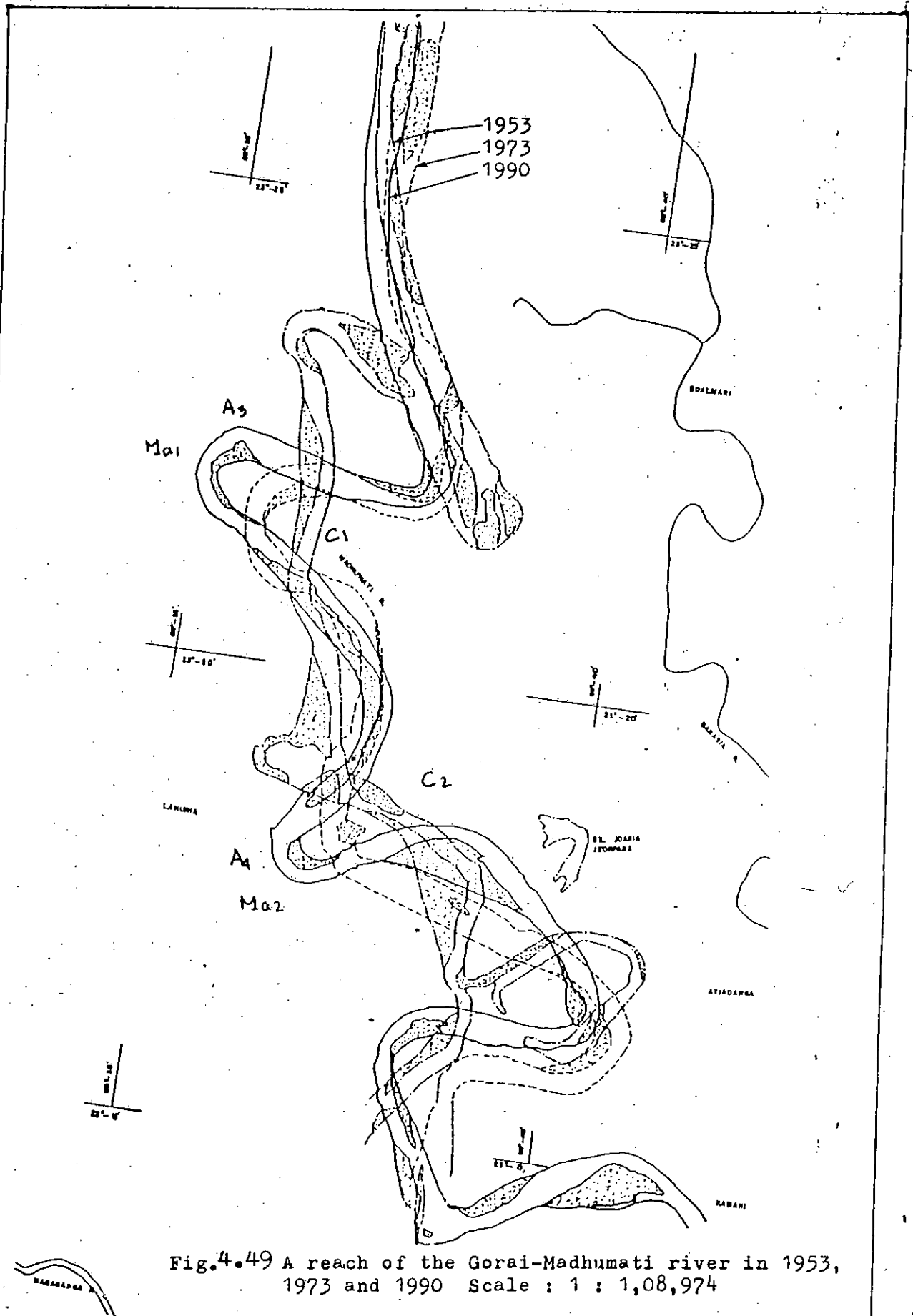


Fig.4.49 A reach of the Gorai-Madhumati river in 1953, 1973 and 1990 Scale : 1 : 1,08,974

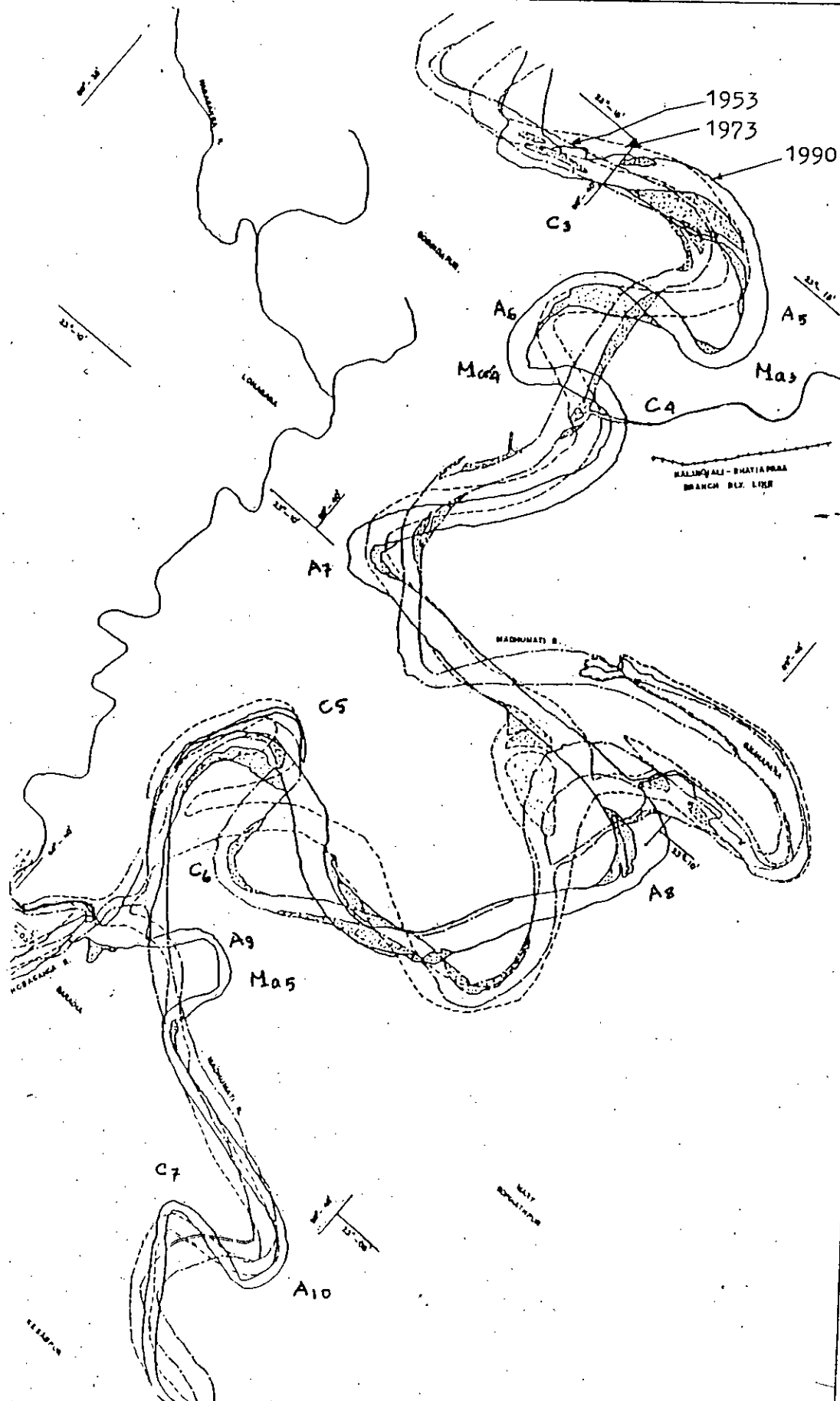


Fig. 4.50 A reach of the Gorai-Madhumati river in 1953, 1973 and 1990 Scale : 1 : 1,08,974

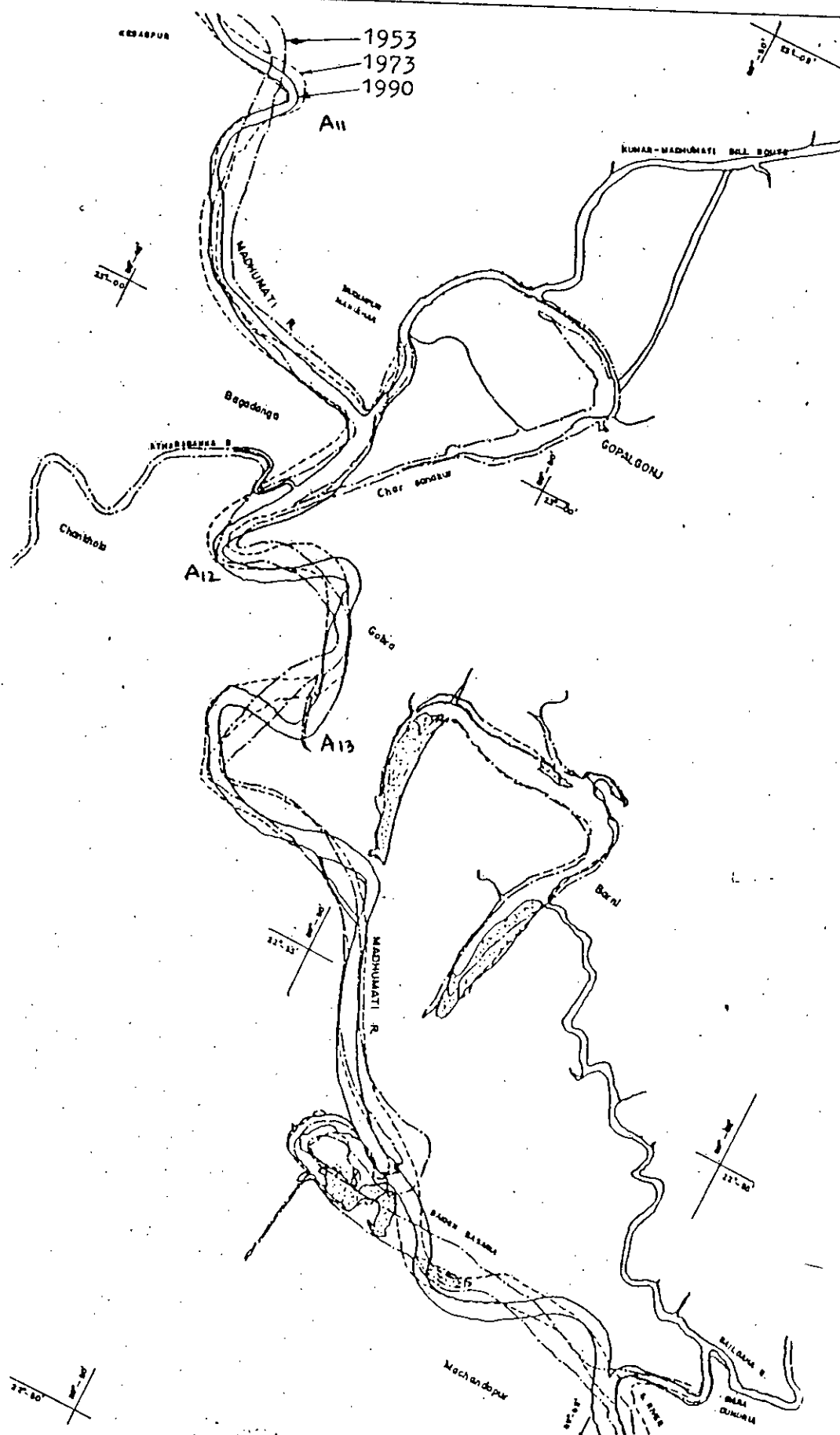


Fig 4.51 A reach of the Gorai-Madhumati river  
in 1953, 1973 and 1990 Scale : 1 : 1,08,974

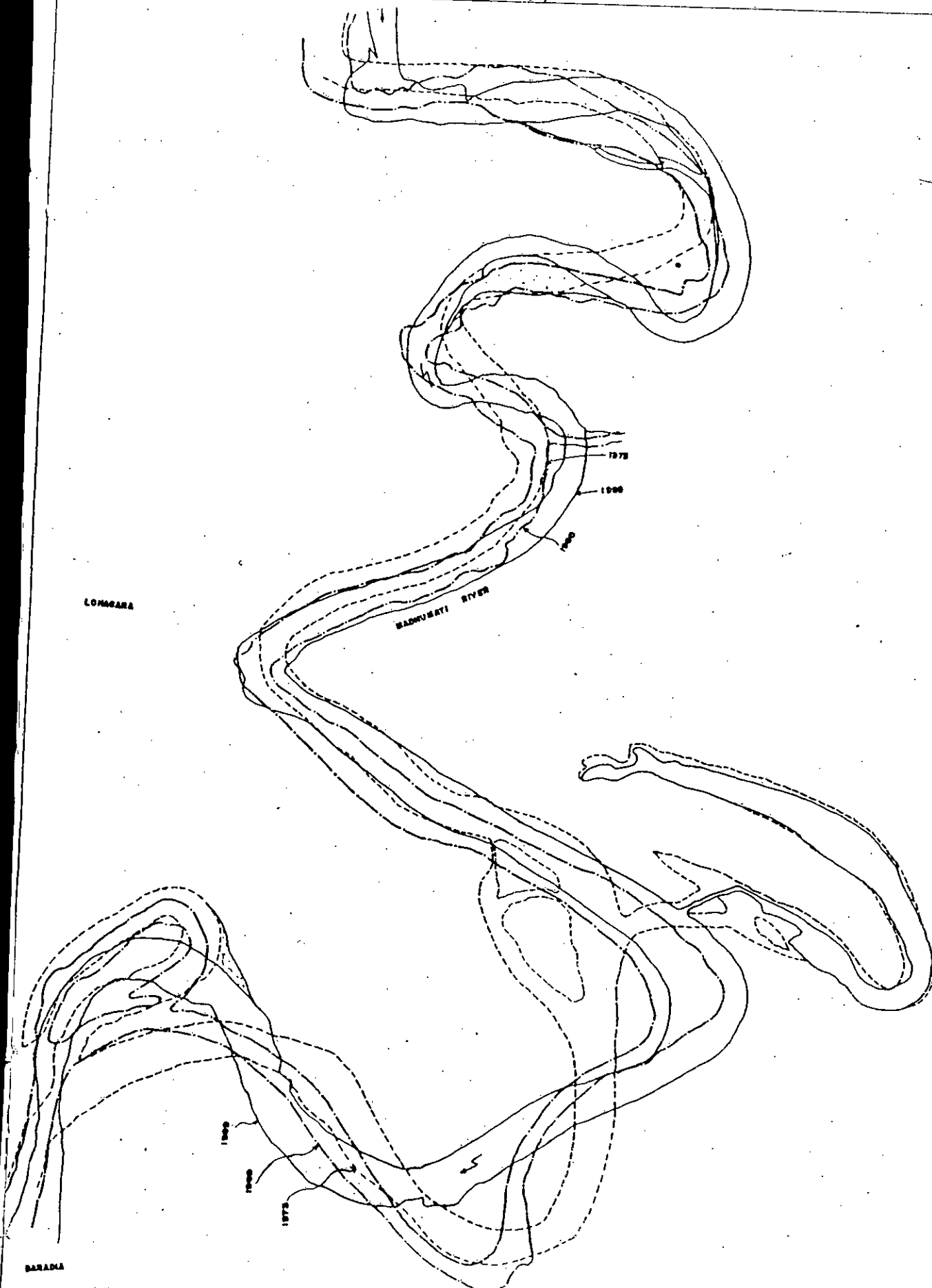


Fig4.52A reach of the Gorai-Madhumati river in 1973, 1980 and 1989  
Scale : 1 : 78,200

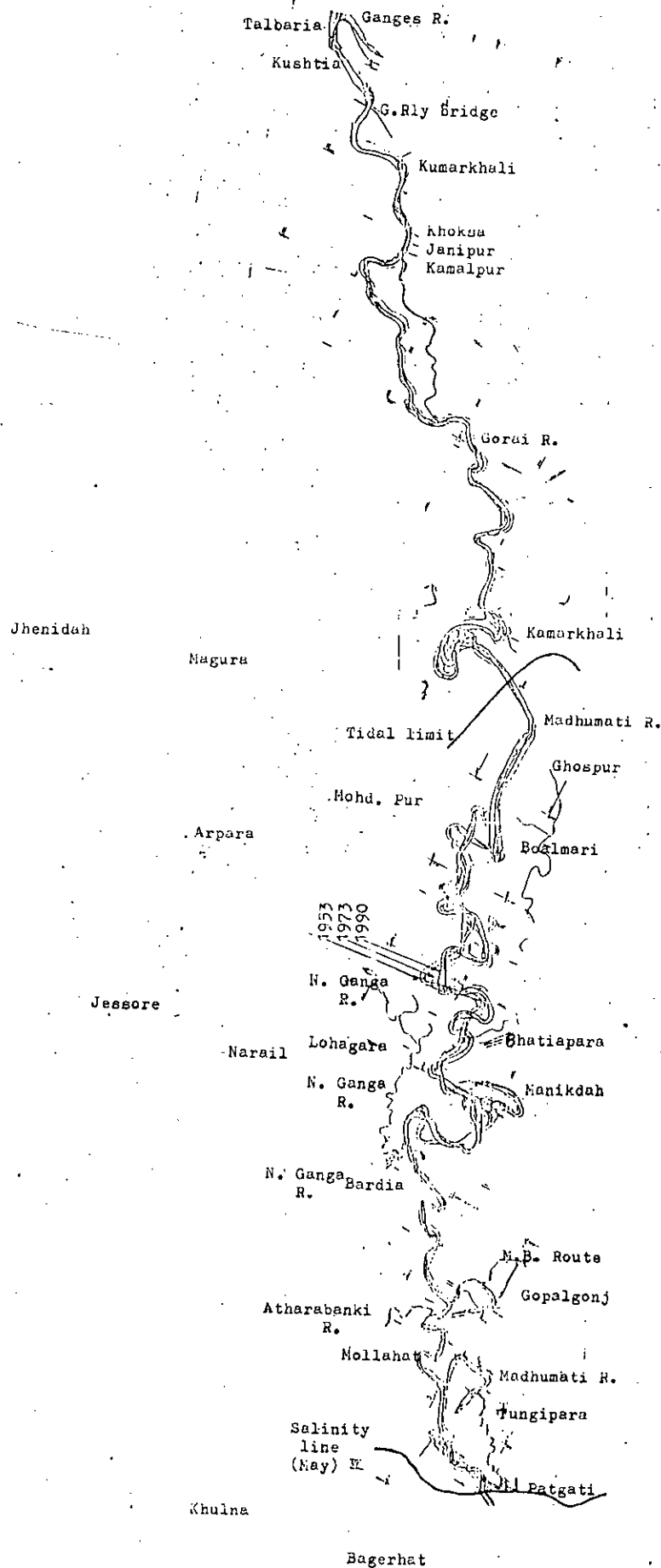
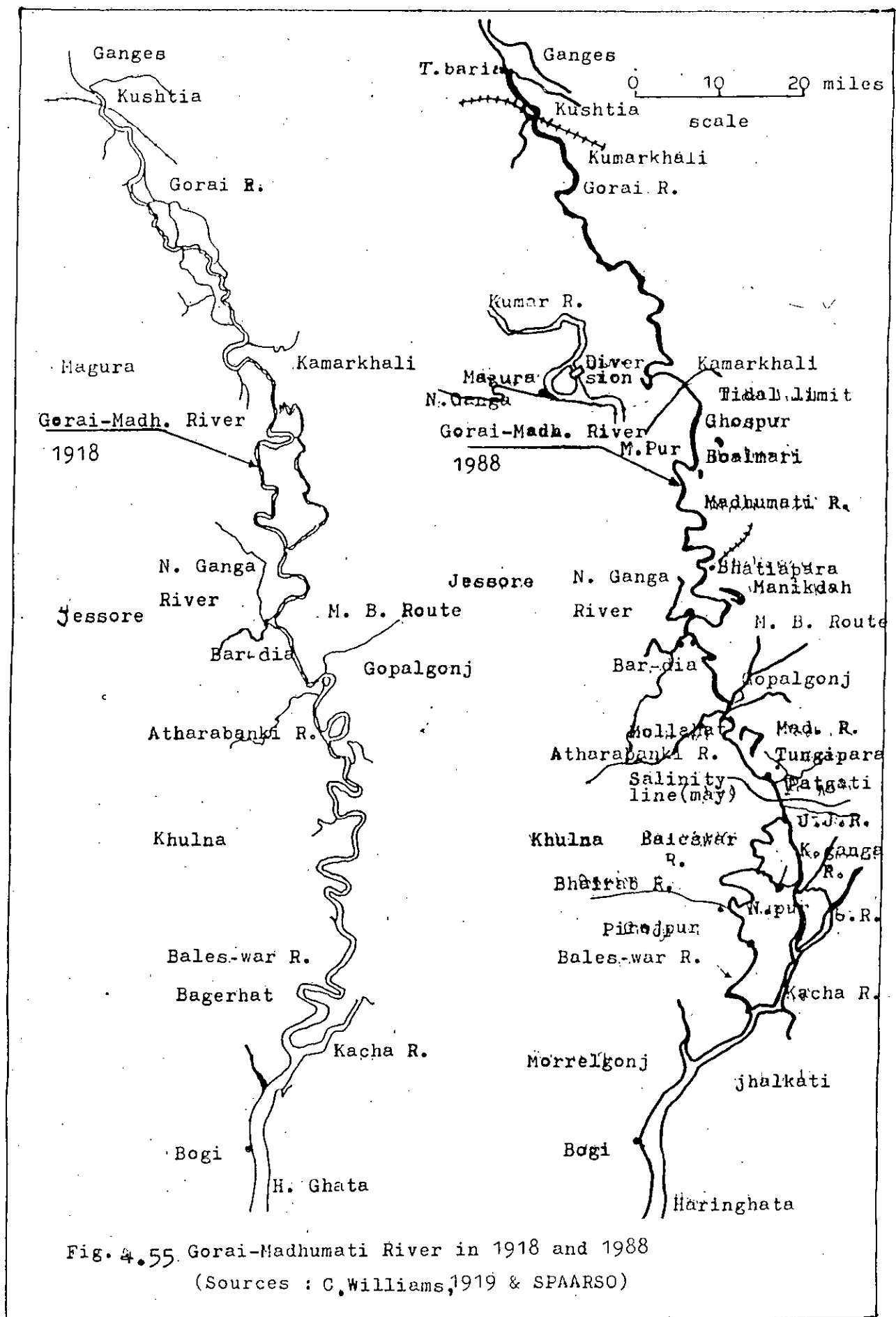


Fig. 4.54 Gorai-Madhumati river in 1953, 1973 and 1990  
Scale 1 : 440,14





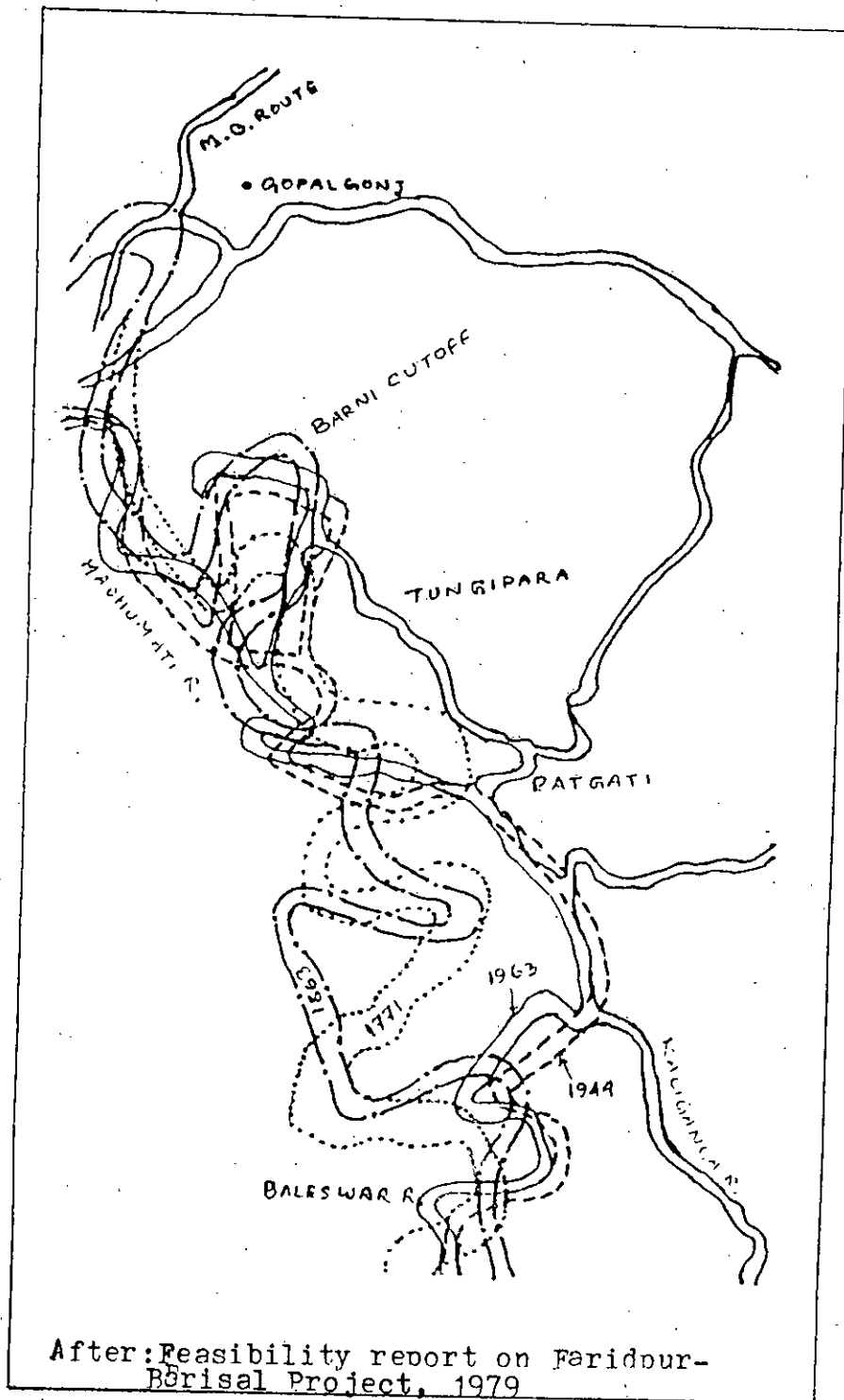


Fig.4.56 A reach of Madhumati-Baleswar river in 1771, 1863, 1944 & 1963

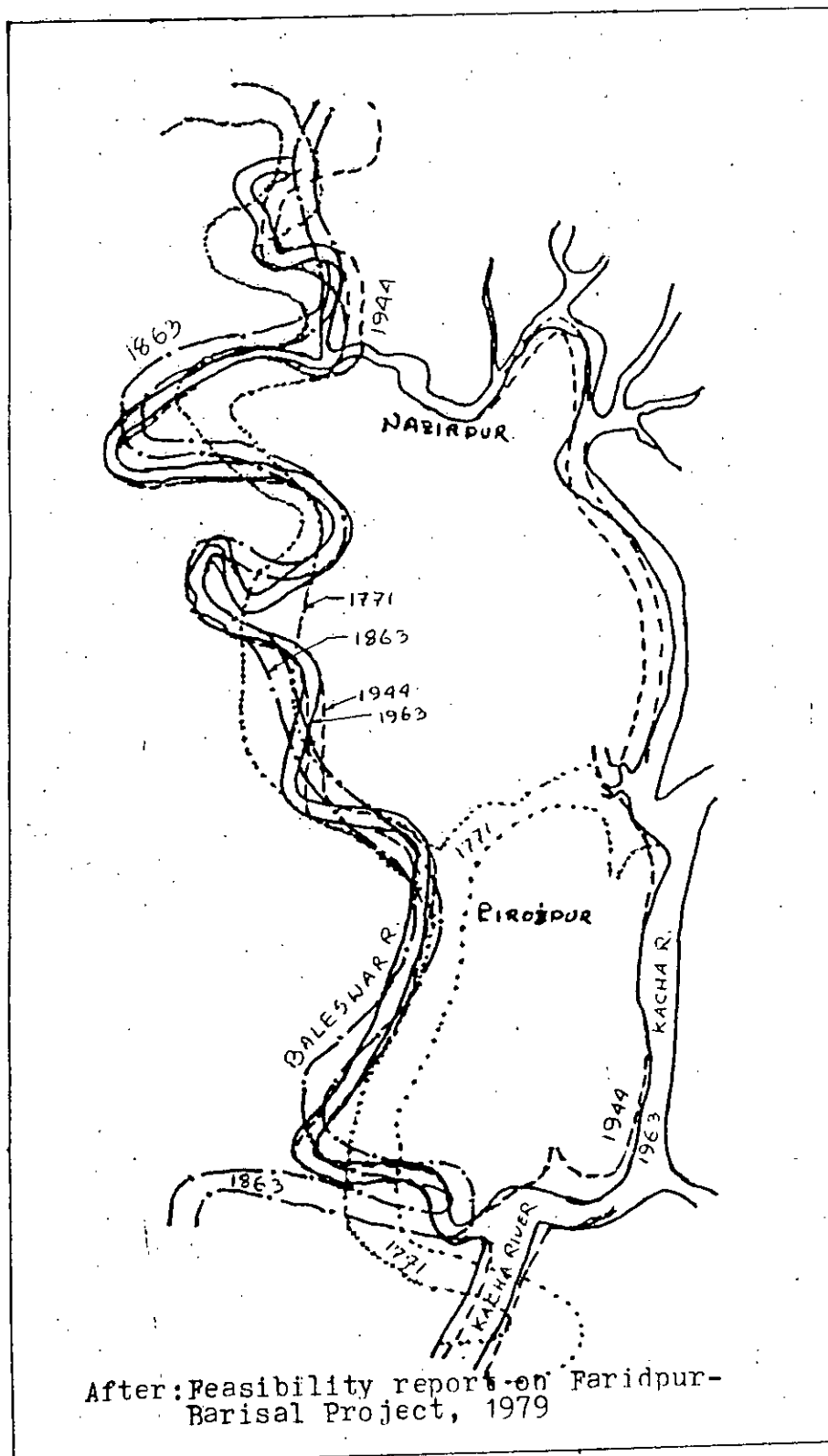


Fig.4.57 A reach of Baleswar river in 1771, 1863, 1944 & 1963

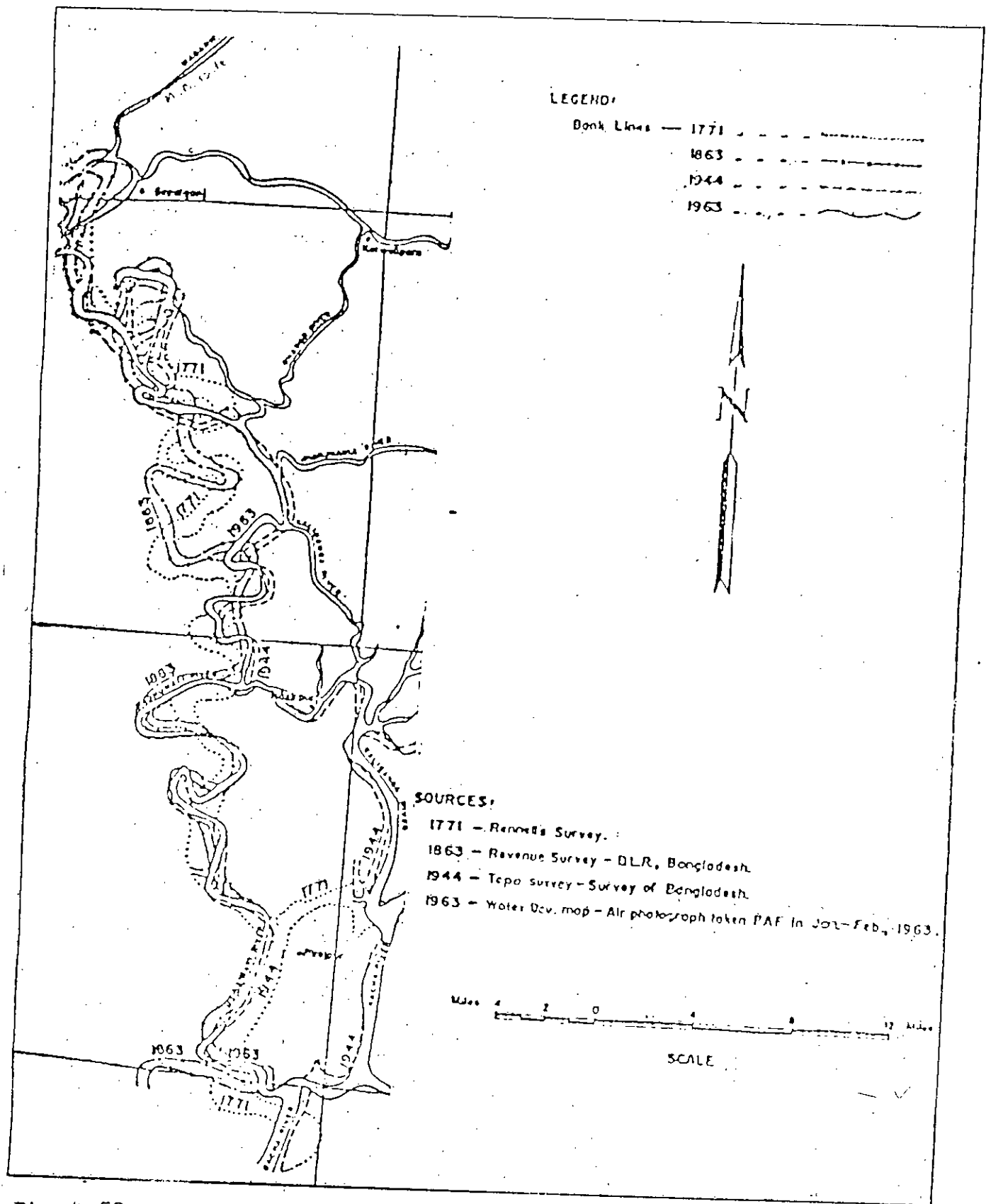


Fig. 4.58 A reach of Madhumati-Baleswar river in 1771, 1863, 1944 & 1963  
 After : Feasibility report on Faridpur-Barisal project, 1979

# GRADATION CURVE OF BED MATERIALS

RIVER: GORAI

STATION: RLY. BRIDGE

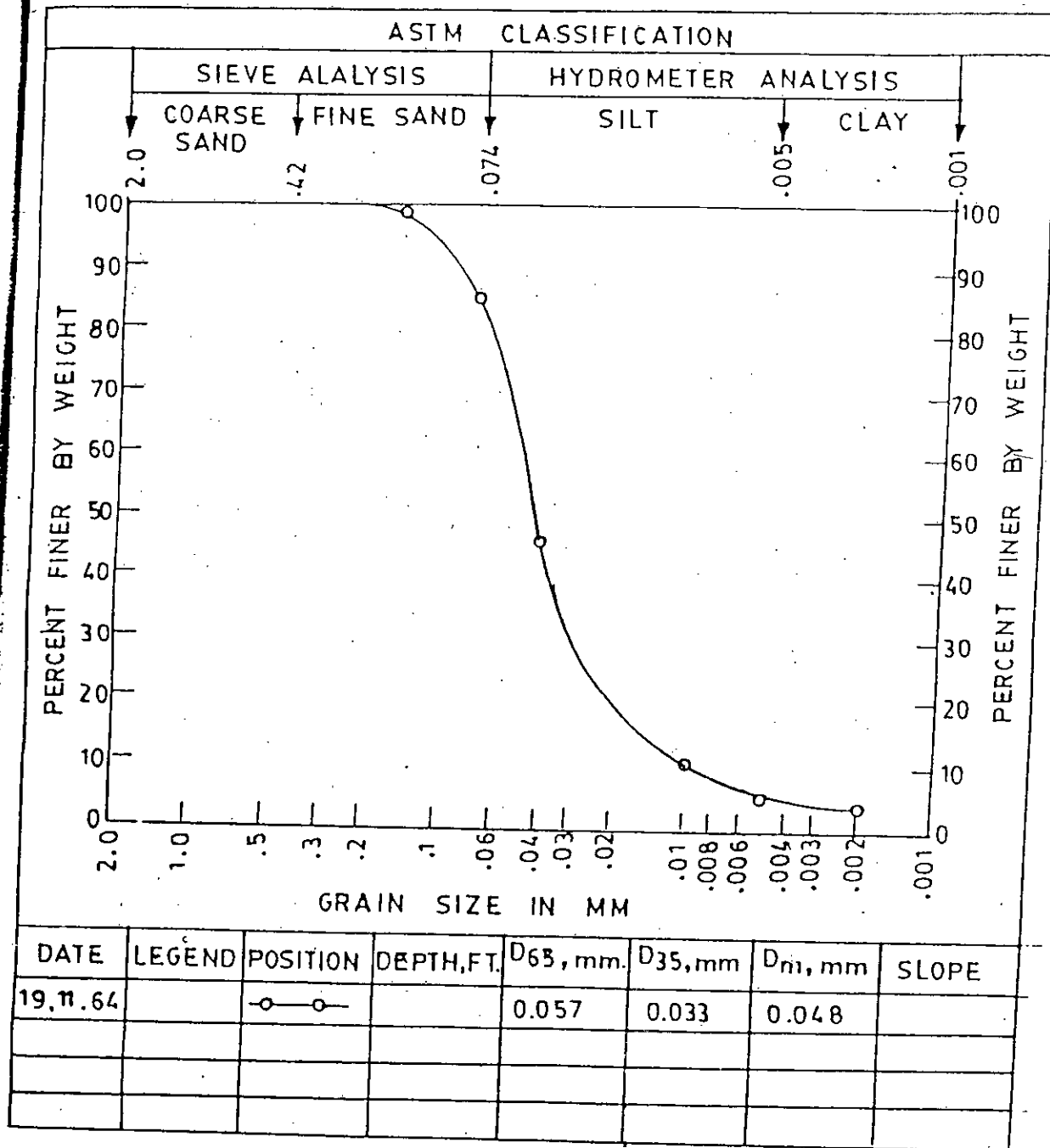


Fig.4.59 Gradation Curve of Bed Material of Gorai River at Gorai Rly. Bridge Station  
( Source : Walliuzzaman, M. 1986)

