STUDY OF SEDIMENT TRANSPORT IN THE RIVER GORAI-MODHUMOTI

Submitted by
WALIUZZAMAN KHAN

In partial fulfilment of the requirements for the degree of Master of Science in Water Resources Engineering, Bangladesh University of Engineering & Technology, Dhaka.

September, 1986

CERTIFICATE

This is to certify that this thesis work has been done by me and neither this thesis nor any part thereof has been submitted elsewhere for the award of any degree or diploma.

Countersigned

Dr. Monowar Hossain
Supervisor

Signature

Waliuzzaman Khan
Candidate

BANGLADESH UNIVERSITY OF ENGINEERING & TECHNOLOGY

September, 1986.

WE HEREBY RECOMMEND THAT THE THESIS PREPARED BY WALIUZZAMAN KHAN ENTITLED STUDY OF SEDIMENT TRANSPORT IN THE RIVER GORAI-MODHUMOTI BE ACCEPTED AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN WATER RESOURCES ENGINEERING

Chairman of the Committee	DR. Monowar Hossain
Member	Dr. A. Nishat
Member	DR. A. Hannan
Member (External)	Mr. Nurul Huda
Head of the Department	DR. A. Nishat

ABSTRACT

An attempt has been made to study the sediment transport in the river Gorai-Modhumoti by using available data from Bangladesh Water Development Board (BWDB). Comparison of five recent and well known formulae for total load prediction namely, Engelund-Hansen (1967), Yang (1976), Ackers-white (1973), Mantz (1983) and Strathclyde formula (1984) have been made against two hundred and ninteen (219) sets of data collected by BWDB. No attempt has been made to obtain a new formula but a correlation between observed and calculated sediment concentration have been found out.

It is revealed from this study that about 13.27 million tons of suspended sediment pass through the Gorai-Railway Bridge of Gorai-Modhumoti river annually. The total load as computed by the Engelund-Hansen and Strathclyde formula were respectively found to be 30.4% and 18.3% higher than the measured suspended load. Other equations provided very unrealistic prediction.

Attempt has also been made to establish the hydrogeometric relations, and rating curves have been developed for practical uses.

Finally, suggestions are made for possible extention of this work.

ACKNOWLEDGEMENT

The author acknowledges his indebtedness to Dr. Monowar Hossain, Assistant Professor, Department of Water Resources Engineering for his kind supervision, encouragement and guidance during the course of studies and research. It was great privilege for the investigator to work with Dr. Monowar Hossain, whose constant guidance made this work possible.

The author is greatly indebted to Dr. A. Nishat, Professor and Head of the Department of Water Resources Engineering for his keen interest, constant insistence and affectionate guidance during the course of the research. Friendly co-operation and useful suggestions of Mr. Liaquat Ali Khan, Asstt. Professor of Water Resources Engineering Department, is also appreciated.

Gratitude is expressed to Mr. Zahurul Islam, Director, Soil Mechanics & Material Division, Bangladesh Water Development Board (BWDB), Mr. A.I.M. Dara, Deputy Director, Surface Water Hydrology (SWH)-II, BWDB, Mr. Kazi Siddiqur Rahman, Technical Officer, SWH-II,BWDB for their help and cooperation during data collection for this study.

Sincer appreciations are expressed to Mr. Hassan Ali for typing the manuscript and Mr. Shahiuddin for preparing the figures.

CONTENTS

LIST OF TABLES

List of Figures

LIST OF SYMBOLS

Symbol	Definition
A	Cross-sectional area of flow, ft^2, m^2
B	Channel width, ft
C_a	Concentration of sediment with fall velocity ω at level y=a above the bed, ppm
C_b	Concentration of the same size of bed material at the bed, ppm
C_c	Calculated total sediment concentration, ppm
C_{md}	Reference concentration at $y=\frac{d}{2}$, ppm
C_T	Observed total sediment concentration, ppm
D	Particle size, mm
D_s	Diameter of sediment, mm
D_{50}, D_m	Median sediment size, mm
d, H	Depth of flow, ft, m
e_b	Bed load transport efficiency
g	Acceleration due to gravity, ft/sec^2
g_s	Sediment discharge, lbs/sec-ft.
i_{BW}	Fraction of the total sediment load for a given sediment size D
i_T	Specific percentage of total load
i_b	Size distribution of the bed material at the cross-section
i_{bw}	Fraction of bed sediment of a given sediment size D
k	Correction factor
k_b, k_T	Constants

Symbol	Defination
M	Sand modulus
Q	Total water discharge, ft^3/sec, m^3/sec.
Q_s	Suspended sediment discharge
Q_T	Total sediment discharge
q	Water discharge per unit width
q_T	Bed material discharge per unit width per unit time
q_b	Bed load discharge per unit width and time
q_c	Discharge per unit width of channel slope S_c where sediment transport begins
q_s	Suspended load discharge per unit width per unit time
q_{sv}	Volume transport rate of the bed load per unit width
q_{bw}	Bed load discharge by weight per unit width per unit time
q_{sv}	volume of suspended sediment load discharge per unit time and width
q_{sw}	Weight of suspended sediment load discharge per unit time and width
$q_{sui}, q_{smi}, q_{sLi}$	Suspended load discharge per unit width in upper, middle and lower zones for sediment of size D
R	Hydraulic radius, ft, m
S	Energy slope
S_c	Channel slope where sediment transport begins.
S_r, S_k	Parameters defined by Meyer-peter Muller
S_s	Specific gravity of sediment
T	Water temperature, ^oC

Symbol	Defination
U,V	Average flow velocity, ft/sec.
U_*	Shear velocity
X	Sediment transport mass flux rate per unit mass flow.
y	Distance above bed
z	Exponent of suspended distribution
ω	Fall velocity, cm/sec.
γ	Specific weight of water, kg/m^3, lb/ft^3
γ_s	Specific weight of sediment, kg/m^3,lb/ft^3
γ_s'	Submerged specific weight of sediment, kg/m^3,lb/ft^3
ρ	Density of water, $lb\text{-}sec/ft^4$
ρ_s	Density of sediment $lb\text{-}sec/ft^4$
ν	Kinematic viscosity of water, cm^2/sec.
τ_o	Shear stress at the boundary lb/ft^2
τ_c	Critical shear stress, lb/ft^2
ϕ_*	Intensity of bed load transport
ψ_*	Intensity of shear for individual grain size.

CHAPTER - 1
INTRODUCTION

1.1 Background of the Study

Sediment transport plays an important role in the regulation and control of rivers. The mechanism of sediment transport has been the subject of study for centuries due to its importance in the design and operation of various water resources project. A number of relationships have been developed to compute the amount of sediment discharge as a function of the various flow parameters. The very number of such formulae have served to emphasise the unsatisfactory nature of these relationships. None of the available equations for the calculation of sediment discharge has gained universal acceptance in confidently predicting sediment transport rate. The calculated sediment from various equations often differ drastically from each other for a given set of observed data. This is due to inclusion of so many variables that influence sediment transport like the size of sediment, the fall velocity, specific weight,cohesion, porosity of particles etc. Moreover, river parameters paly a leading role in alluvial channel formula.

Engineers thus must have to choose the best sediment transport formula to solve a river problem. But it is very difficult to recommend the best sediment transport formula as so many river parameters have been considered by different investigator to establish their formula. To ease this difficulty many of the most commonly used transport formulae have been tested by various researchers over a wide range of field and laboratory data. Among these the followings are very prominent.

i) The ASCE Task committee (Vanoni 1975)

ii) HRS study (White et.al. (1975)

iii) Yang (1977)

iv) Gole et. el. (1973)

v) Ranga Raju et al (ASCE, 1981)

The ASCE Task committee have studied thirteen formulae and observed that Colby (1964), Tofaletti (1969) and Engelund-Hansen (1967) formula gave consistently better results than others.

On the otherhand, 19 transport formulae against about 1000 laboratory and 270 field measurements have been tested at Hydrauulic Research Station, Wallingford, U.K. It was found that out of 19 theories, the Ackers-White (1973), Engelund-Hansen (1967) total load and Rottner (1959) bed load formulae is the most reliable.

1.2 Importance of the Study

The study of sediment transport of Gorai-Modhumoti at present is very necessary as it is the main branch of the Ganges river.

In every year large discharge and heavy sediment load in the rainy season cause the river Gorai to be extremely unstable and the channels are constantly migrating laterally. Due to heavy siltation water level in the river Gorai is falling gradually. This gradual falling of water level will have tremendous adverse effect on six on going projects like chenchuri beel Irrigation and drainage project (Narail), Baliakandi Irrigation Project (Rajbari), Gorai Irrigation Project (Magura), Barasia Irrigation Project (Gopalganj), Magura-I Irrigation Project (Magura) and old river Resuscitation Project (Magura) on both sides of Gorai-Modhumoti (Fig.1.1). The above projects have a net benifited area of about 470,450 acres and are fully dependent on the water of Gorai-Modhumoti. Moreover, siltation problem arises at intake of the river Gorai. Insufficient flow rates at Gorai-Modhumoti cause saline water intrusion in Khulna Industrial Zone. This saline water intrusion has adverse effect on the processing of the industrial projects. Thus, it appears to be of paramount necessity to study the various hydro-geological aspect of Gorai-Modhumoti river, including the sediment aspect for planning any further water resources development projects.

The sediment transport formulae available in the literature have been derived primarily based on laboratory flume and natural channel data collected mostly from U.S.A., U.K., and Canada. A very few data from the main rivers of Bangladesh like the Ganges, the Bhramaputra and the Meghna have been employed to test them. So, available data of this river will be an additional testing of the validity of the formula and recommending the best one as far as the sediment transport aspect of Gorai-Modhumoti river is concerned.

Chang (1980) selected DuBoys (1879), Einstein-Brown (1950) and Engelund-Hansen (1967) formula for verification with field data. He found that Engelund-Hansen formula produced the best results when tested against Indian and American canal data.

Yee (1976, after Hossain 1985) tested DuBoys, shields, Meyer-Peter-Muller, Einstein-Brown, Bagnold, Yalin, Engelund-Hansen and Ackers-White formula. He tested these equations against laboratory flume data of trapezoidal channel section. He found that Ackers-white, Bagnold and Engelund-Hansen formulae gave the best results in predicting the rates of sediment transport.

Hossain, (1970) studied about the sediment characteristics of the Teesta river. He applied the then available field data of the Teesta river in modified Einstein method. The predicted sediment flow showed very poor correlation with the measured sediment load.

Bari (1978) applied five sediment transport formulae, namely Colby's equation (1964), Engelund-Hansen formula (1967), Ackers-white formula (1973) and Yang's formula (1976) against the data of the Ganges and Jamuna. He compared the sediment load predicted by these equations with the measured sediment flow and suggested that Colby and Engelund-Hansen formulae give better prediction.

Hossain (1984) selected nine well known sediment transport equation and tested them with 4260 sets of flume and field data. He concluded that for $Q \leqslant 1.0$ cumecs the Ackers-white equation (1973) was found to hold supremacy over the other eight equations. He further added that for $Q > 1.0$ comecs the Strathclyde equation was found to be the most satisfactory. There followed the Engelund-Hansen (1967), the Ackers-white (1973) the Einstein Brown (1950), the Yang (1973) and Toffaleti (1969), in this order and then the other equations.

Considering the above facts and availability of the field data the following five formulae have been chosen to test with 219 sets of data of the river Gorai-Modhumoti. The formulae considered in this test are:

i) Strathclyde formula (Hossain 1984)

ii) Engelund-Hansen formula (1967)

iii) Yang formula (1976)

iv) Ackers-white formula (1973)

v) Mantz formula (1983)

1.3 The Objectives of the Study

The objectives of the present study are:

i) to quantify the amount of sediment movement of the river Gorai-Modhumoti by applying different sediment transport formulae.

ii) to comapre the results and suggest the best transport formula

iii) to find out the efficacy of various transport formulae with the data of the river Gorai-Modhumoti.

iv) to establish sediment rating curves and to formulate simplified empirical equations for practical use.

CHAPTER - 2
SEDIMENT PROPERTIES AND ITS SOURCE

2.1 Introduction

An equlibrium channel under the existing circumstances balances its sediment transporting ability to the available sediment loads. The sediment loads greatly influence its morphology and pattern. In the following section a brief review of sediment properties, its source and characteristics commonly used will be made.

2.2 Physical Properties of Sediment

Sediments are broadly classified as cohesive and non-cohesive. With cohesive sediment the resistance to erosion depends on the strength of cohesive bond binding the particles. Cohesion may far outweigh the influence of the physical characteristics of the individual particles. However, once erosion has taken place, cohesive material may become non-cohesive with respect to transport. Also sediment characteristics may change through chemical or physical reactions. On the other hand, the non-cohesive sediments generally consist of large discrete particles than the cohesive soils. Non-cohesive sediment particles react to fluid forces and their movement is affected by the physical properties of the particles such as size, shape and density.

2.2.1 Size of the Sediment Particle

Of the various sediment properties, size has the greatest significance to the hydraulic engineer, not only because size is important and the most readily measured property, but also because other properties such as shape and specific gravity tend to vary with particle size. Particle size may be defined by volume, diameter, weight, fall velocity, sieve size and by intercepts through the particle. With the exception of volume, the definitions are generally influenced by the shape and density of the particle.

Size may be measured by calipers, by optical methods, by photographic methods, by seiving or by sedimentation methods. The size of an individual particle is not a primary importance in river mechanics or sedimentation studies, but the size distribution of the sediment that forms the bed and banks of a stream or reservoir are of great importance.

A size classification recommended by the Sub-committee on Sediment Terminology of the Committee on Dynamic of streams of the American Geophysical Union (Lane and others, 1947), contains consecutive size classes; boulders, cobbles, gravel, sand, silt and clay as given in table - 2.1

2.2.2 Shape of the Particle

Generally speaking, shape refers to the overall geometric form of a particle regardless of size or composition. Two particles have the same shape if the ratio of their intercepts and the angles between the intercepts are the same for the both particles. Particles of very different geometrical shape, but of the same volume and density, may behave the same in fluids. Hence the shape may be defined in terms of dynamic behaviour.

In sediment analysis one of the most pertinent shape parameters is "sphericity" which has defined by Wadell (1932) as the ratio of surface area of a sphere of the same volume as the particle to the actual surface area of the particle. The primary role of sphericity is to help describe the relative motion between the falling particle and the fluid.

In contrast to the sphericity of a particle, is its "roundness", which is defined as the ratio of the average of the corners and edges of a particle to the radius of a circle inscribed in the maximum projected area of the particle. Roundness is thus geometrically independent of sphericity. Studies show that roundness has essentially a negligible effect on the hydrodynamic behaviour of particles, but it is of first order importance in abrasion studies. Both sphericity,roundness are dimensionless and tend to decrease with decreasing size of particle, but sphericity depends upon mineral composition as well.

2.2.3 Density (ρ)

The density of a solid is the mass which it possesses per unit volume. Again the density of a sediment particle is a function of its mineral composition. Water borne sediments are mainly quartz and felspathic minerals with a specific gravity of 2.65. For this reason the specific gravity of water borne sediments is often assumed to be 2.65. It would not be reasonable to use 2.65 for waste from a coal field or volcanic area.

2.2.4 Fall Velocity or Settling Velocity (ω)

The fall velocity is the average terminal settling velocity of a particle falling alone in quiscent, distilled water of infinite extent and at a temperature of 24^{o}C. When a sediment grain moves through water, it experiences considerable resistance, which is a function of the Reynolds number. When the particle moves downward, a velocity known as terminal velocity is reached at which the resistance equals the weight of the grain in water.

Several approaches are adopted in determining fall velocity. For laminar and turbulent flow around the grain, the settling velocity for spherical grains is given by stokes law as follows:

$$\text{For laminar flow, } \omega = (S_s - 1)\,\frac{g}{18\nu}\,D_s^2 \qquad (2.1)$$

$$\text{For turbulent flow, } \omega = \sqrt{(S_s - 1)\,\frac{4}{3}\,\frac{gD_s}{C_D}} \qquad (2.2)$$

Where ω is the fall velocity in cm/sec., D is the grain diameter in cm, g is the acceleration due to gravity in cm/sec^2, ν is the kinematic viscosity of fluid in square cm per sec. and C_D is a co-efficient and equal to, $C_D = \frac{24}{R\omega}$. Where R_ω is the

Reynolds number.

Rubey (1933) gave a formula for determining fall velocity which has the advantage of being suitable for similitude analysis. His formula is

$$\omega^2 = \frac{A_3'^2 \; D_s \; g\gamma_s'}{A_3^2 \gamma}$$

Where A_3' and A_3 are constants. Here A_3 = 1.225 for quartz particles greater than 1mm.

For smaller grains Rubey gave the following equation,

$$\omega = \frac{\sqrt{\frac{2}{3} g \; (S_s - 1) \; D_s^3 + 36v^2} - 6v}{D_s}$$

Where ν is the kinematic viscosity. Other terms have been defined previously.

Raudkivi in 1967 gave a relationship between fall velocity and sediment size as shown in fig. 2.1.

Dietrich (1982) has reported that no theory based on the physics of flow over irregular objects exist to predict the settling velocity of natural particles. In its absence researchers have proposed empirical curves based on laboratory experiments (Graf, 1971, Baba and Komar, 1981). Dietrich (1982) has developed a set of empirical equations in terms of non-dimensional parameters, for the effect of size, shape, density and roughness on settling velocity and claimed its superiority over others. But, since some of the parameters in his complicated equations are to be read from graphical representations supplied, its computer application against large body of data is impracticable.

Considering all these facts and its reliability, Raudkivi's graphical representations has been chosen for settling velocity computation in this study.

2.2.4.1 Factors Affecting the Fall Velocity of a Particle

The primary variables defining the intraction of sediment transport with the bed, banks or suspended in the fluid is the fall velocity of sediment particles. It has been shown that the bed configuration in a sand channel may change when the fall velocity of the bed material change. The variables affecting the velocity of a particle falling in quiescent, distilled water of infinite extent are:

$$\phi_1 \, (\omega, \rho_f, \rho_s, \mu, D_s, S_p, f, S_r, F) = 0 \qquad (2.3)$$

Where ω is the fall velocity, ρ_f is the density of fluid, ρ_s is the density of the particle, μ is the dynamic viscosity of fluid, D_s is the particle diameter, S_p is the shape factor of particle, f is the frequency of oscillation, S_r is the surface roughness, F is the buoyant weight of the particle.

2.2.5 Size Frequency Distribution

The most commonly used method to determine size frequency is mechanical or seive analysis. In general, the results are presented as cumulative size frequency curves. The fraction or percentage by weight of a sediment that is smaller or longer than a given size is plotted against particle size. From the size frequency curve it is possible to obtain representative grain size of sediment mixture. Physical evidence does not conclusively fix the size that represents a given sediment mixture. The choice is rather arbitrary and varies from researchers to researchers. For usual practice D_{35}, D_{50}, D_{65}, D_{85}, D_{90} are commonly used.

D_{35} indicates the size of sediment for which 35 percent of the sample is finer. D_{50} represents median diameter of the sediment mixture and indicates 50 percent sample is finer. In the same way D_{65}, D_{85}, D_{90} represents that 65 percent, 85 percent and 90 percent sample is finer respectively.

Dm is the mean diameter given by

$$Dm = \frac{\Sigma \Delta_i \; D_i}{100}$$

Where Δ_i represents any portion of the percentages shown on the y axis of Fig.2.2 and D_i represents the mean value of the sizes established by the extreme values of the interval Δ_i. According to Fig.2.2 Dm is computed as follows:-

$$Dm = \frac{\Delta_1 D_1 + \Delta_2 D_2 + \Delta_3 D_3 + \dots\dots\dots + \Delta_6 D_6}{100}$$

and Dm represents the mean size of the sample. It should not be assumed that the particle measure Dm represents the hydraulic properties of the sediment mixture.

2.3 Sources of Sediment

The followings are the principal sources of stream-borne sediment,

i) Sheet erosion by surface runoff from precipitation on agricultural, forest and waste land-sheet erosion being defined by soil conservationists as the removal of surface soil by overland flow without the formation of channels of sufficient depth to prevent cultivation or crossing by form machinery.

ii) Stream-channel erosion, including bank cutting and degradation of formerly well-defined channels.

iii) Mass movements of soil landslides, slumps and soil creep.

iv) Gullying, or the cutting of channel in soil or unconsolidated geologic formations by concentrated runoff.

v) Flood erosion, or the removal of surface soil by flood flows sweeping across flood plains.

vi) Mining, industrial, and sewage wastes discharged into stream.

vii) Erosion due to cultural developments, including roads, railroads, power lines and industrial projects.

Of these the first two are most important and major sources of sediment supply into streams.

2.4 Modes of Sediment Transport

Water flowing over a bed of sediment exerts forces on the grains. These forces tend to move or entrain them. The forces that resist the entraining action of the flowing water differ depending on the properties of bed material. For coarse sediments such as sand and gravels the resisting forces mainly relate to the weight of the particles. When the hydrodynamic forces acting on a grain of sediment have reached a value that, if increased even slightly the grain will move, critical condition is said to have been reached.

After achieving this critical condition the fine sediments first start to move and then coarser particles are in motion. Sediment particles are transported by flow in one or a combination of the following modes:

i) Sliding or Rolling along the bed. According to this mode the sediment particles are transported along the bed. If the particles are relatively round, it is more likely that they will roll along the bed. On the otherhand, if they are angular or cubical , they will overturn or slide.

ii) Saltation. In this mode, the particles move forward in small jumps. Saltation depends upon the reaction of the particles from the bed. This causes necessary impulsive force required for the jumping phenomenon.

iii) Movement in suspension. According to this mode the sediments remain in suspension. The particles do not come in contact with the bed. The particles are supported and kept in suspension by the turbulent eddies.

There is no sharp distinction between saltation and suspension. However this distinction is important for it serves to delimit two methods of hydraulic transportation which follow different laws i.e., fraction and suspension. Again sediments may be transported partialy as saltation and then suddenly be caught by the flow turbulence and transported in suspension. Sediments which move as saltation are supported by the bed, are called bed load and sediments which are suspended and supported by the flow are called suspended load.

2.5 Distribution of Velocity and Sediment Concentration

From experiments it is found that the vertical distributions of velocity and concentration vary with depth. Generally concentration is higher near the bed, while velocity is maximum near the water surface and about to zero near the bed as shown in Fig.2.3.

After several experiments Colby (1963) found that concentration varies with depth and particle size. This is shown in Fig. 2.4. Fig. 2.5 shows the lateral variation of particle and concentration. Material finer than about 0.062 mm is distributed uniformly with respect to vertical and lateral position. This finer material generally is called the wash load and is transported at the rate at which it is made available from the watershed and from the stream bed and banks. The size distribution of finer material in suspension bears no relation to the size distribution of the material comprising the channel bed, and the quantity transported usually is not related to the properties of flow. The transport of coarse sediments that are found in appreciable quantities in the bed, on the otherhand, usually is related at least roughly to the flow.

Toffaleti (1969) suggested a relation of sediment concentration as a function of velocity of flow. This is shown in Fig. 2.6. He divided the total depth of flow into four zones. The sediment concentration distribution of each size fraction is given by a power relation for each of the three upper zones as shown in Fig. 2.6. The velocity profile is represented by the power relation.

$$u = (1 + n_v)\, U \left(\frac{y}{d}\right)^{n_v}$$

Where n_v is given by the emperical relation

$$n_v = 0.1198 + 0.00048T$$

in which T = water temperature in degrees Ferhenhite.

2.6 Factors Affecting Sediment Transportation

The large number of variables which affect the sediment transportation are interdependent. Some of the variables change with the flow conditions and alter their roles from dependent to independent variables. It is difficult, especially in field studies, to differentiate between independent and dependent variables. A list of variables which are responsible in sediment transportation are given below:

i) Geometric properties of stream channel: depth, width, form and alignment.

ii) Hydraulic properties of stream channel: slope, roughness, hydraulic radius, discharge, velocity, turbulence, tractive force, fluid properties and uniformity of discharge.

CHAPTER - 3

HISTORICAL ASPECTS OF SEDIMENT TRANSPORTATION

3.1 A Short History of Sediment Transport

A study of previous works reveals that the importance of the problem of sediment transport has been realized by the hydraulicians only in recent years. Hydraulics, in general, and the first advancements in the art of sediment transport apparently developed in china some 4000 years before the beginning of the modern era. History tells us that similar knowledge simultaneously developed in Mesopota ia and Egypt. This observation seems paradoxical, if by hypothesis the beginning of these activities were in china and recognizing that transfer of knowledge takes time. One could argue that engineering applications of hydraulics in china developed in an even earliar period. Ten centuries were necessary to transfer new techniques to the people living in the western regions of the world. During this period some hydraulic constructions are still in evidence today in Yugoslavia, Bulgaria, Turkey, Africa and in the Middle East. Advancements in the science of sediment transportation were significant by the start of the second half of the current century (Simons and Senturk,1977).

An insight investigation into the history reveals that the most advances in river hydraulics have taken place as the result of specific problems that required immediate solutions. The first advancements were developed by a man of great talent, Leonardo da vinici (1452-1519). He was not only an artist but is among the greatest of engineers. He also observed and studied sediment movement. Since then Domenico Guglielmini (1655-1710), Frizi (1770), P. Dubuat (1734-1809), Enler (1707-1783), D. Bernouilli (1700-1782), A. Chezy (1718-1798) made their contribution to channel hydraulics and sediment transportation (Simons and Senturk 1977). A. Chezy's contribution to channel hydraulics includes the well-known Chezy uniform flow formula used to estimate the average velocity of flow in open channels. This formula relates average velocity, U, the slope of energy gradient, s, and hydraulic radius, R.

$$U = c\sqrt{Rs}$$

where c is a resistance factor that varies with channel characteristics and boundary conditions.

Dupuit (1804-1866, after Simons & Senturk, 1977) was the first person to give serious consideration to the transportation of sediment in suspension. He oberved that the transportation of sediment in suspension was due to the excess of velocity on the upper side of the particle as compared with that on the lower side. He also conceived that the sediment concentration near the bottom of the channel was greater than that near the surface.

DuBoys (1879, after Simons and Senturk, 1977) presented his theory of "tractive force" which has been widely accepted and has been extensively used in studying bed-load movements. He stated that the amount of material carried by streams was dependent first on the slope and then on the depth. He also stated that there was a critical tractive force for each kind of material depending on the size of the material. The magnitude of this force was larger for larger sizes. Also he believed that the bed material moved to a considerable depth.

Deacon (1894,after Simons and Senturk, 1977)presented a very complete discription of the interaction between flowing water and a mobile alluvial bed. His experiments showed that the weight of material transported was proportional to the fifth power of the surface velocity or possibly a little more. He presented two curves, one relates the surface velocity to the discharge of sand, other relates the surface velocity to the velocity of the crests of the sand ripples.

The laboratory study of river and channel problems by Engels (1854-1945, after Simons and Senturk, 1977), added a new dimension to the knowledge of transport of sediments and channel response.

Kennedy (1895, after Simons and Senturk, 1977) first studied in this subcontinent about the sediment problems encountered in the design and operation of Irrigation canals. He reported his conclusions in a paper entitled "The Prevention of Silting in Irrigation Canals". This was the first quantitative study that related transport to channel shape. He proposed the velocity depth relation for which a channel would neither erode nor deposit any sediment. His work was the forerunner of studies by Lindley (1929), Lacey (1929) Inglis (1947), Blench (1970), Nishat (1981) and others that resulted in the so called "Regime Theory".

Hooker (1896, after Simons and Senturk, 1977) presented an important paper titled "The suspension of solids in flowing water" in which he gave an excellent summary of practically all of the related literature up to that date. This paper provides detailed in formation regarding the early development of transport concepts and theories.

3.2 Development of Sediment Transport Formulae

The most difficult problem related to sediment transport in alluvial streams probably consists of determining the rate of transport. Several different approaches have been adopted in determining the transport rate.

a) Theoretical approach: It is derived on the basis of some assumptions.

b) Laboratory observation approach: It is performed in course of which observations were made on the rate of transport, on the hydraulic characteristics of flow and the observed rate were related to each other.

c) Measurement made in the field: The values thus obtained in the field have been used for deriving empirical relations between the transport rate and the hydraulic characteristics of the stream.

Using the above mentioned three approaches several sediment transport formulae have been derived which, however, yield drastically different results. The sediment transport formulae can be classified to a number of aspects. In some of the formulae the rate of sediment transport is expressed in terms of the tractive force or in terms of discharge and slope whereas in several others the velocity of flow is introduced.

3.3 Bed Load Transport Formulae

There are slightly different approaches to the problem of bed load discharge computations. They are,

a) The DuBoys type equations deriving from a shearing stress relationship.

b) The Schoklitsch type equation derived on the basis of discharge relationship.

c) The Einstein type equations based upon statistical consideration of lift forces on bed materials.

3.3.1 DuBoys Type Equation

Much of the early developments in the analysis of bed load was influenced by the work of DuBoys (1879). He assumed that the bed materials move in layers and that the mean velocity of the successive layers increase linearly towards the bed surface. His formula is a relationship between the sediment discharge and the difference between actual and critical shear stress on the sediment bed. This can be expressed as:

$$q_b = k\ \tau_o\ (\ \tau_o - \tau_c\ ) \tag{3.1}$$

where q_b is the volumetric discharge of bed load per unit time and width, τ_o is the unit tractive force exerted by the flow on the bed, τ_c is the shear stress and k is a constant of proportionality.

Straub (1935) suggested average values of k and τ_c for sediment sizes. However his work has been criticised mainly because all the data he utilised were obtained using small scale laboratory flumes. O'Brien and Rindlaub (1934) generalised DuBoys equation as:

$$q_b = k'\ (\ \tau_o - \tau_c\ )^m \tag{3.2}$$

Analysis of Gilberts' (1914) data showed that the new parameter k' and m are a function of the median sediment size. It was found that for $0.025 < D_s < 0.560$ mm, the values of m are confined to narrow range i.e., $1.5 < m < 1.8$.

Shields (1936) proposed from his experimental results, a dimentionally homogenous transport function of the form

$$\frac{q_b \gamma_s}{q^s \gamma} = 10 \frac{\tau_o - \tau_c}{(\gamma_s - \gamma) D_s} \qquad (3.3)$$

where q is the actual water discharge per unit width, D_s is the representative sediment size. The above equation was derived from data using flume width 40 cm and 80 cm, $1.06 < \frac{\gamma_s}{\gamma} < 4.25$ and $1.50 < D_s < 2.47$ mm.

By considering turbulent fluctuations of the flow, Kalinske (1942) developed a bed load equation. He assumed that the velocity of a sediment grain moving on the bed is given by

$$U_s = b\ (U - U_c) \qquad (3.4)$$

where U is the instantaneous fluid velocity at the particle level, U_c is the critical velocity for incipient motion of the particle and b is a constant close to unity. By applying turbulent flow theory, Kalinske demonstrated that,

$$\frac{q_b}{U_* D_s} = f\left(\frac{\tau_c}{\tau_o}\right) \qquad (3.5)$$

and $\tau_c = 0.12\ \gamma'_s \cdot D_s$

where U_* is the shear velocity, γ'_s is the specific weight of submerged sediment.

Chang, Simons and Richardsons (1967) has developed a bed load discharge equation which can be expressed as follows:

$$q_b = k_T V (\tau_o - \tau_c) \qquad (3.6)$$

where k_T is a constant varies between 0.27 to 1.10 applied to the Colorado, Middle Loup and Niobrara Rivers. The equation represents the discharge of sediment in pounds per foot on a dry weight basis.

3.3.2 Schoklitsch Type Equation

In 1930 Schoklitsch, independent from DuBoys, but not in too strong a contradiction suggested an equation based on laboratory experiments, such as

$$q_b = k'' S^k (q - q_c) \qquad (3.7)$$

where k" is a new characteristics sediment co-efficient and q_c is the water discharge at which materials begin to move. Bed load equation with this general form and equations that utilize average velocities have been proposed by various researchers. Schoklitsch in 1934 suggested the relation as follows:

$$q_b = 2500\ S^{3/2}\ (\ q - q_c\ ) \tag{3.8}$$

where, $q_c = 0.26\ (\ \frac{\gamma'_s}{\gamma}\ )^{5/3}\ (\ \frac{D^{3/2}}{S^{7/6}})$

MacDougall (1934) derived a similar kind of equation that can be written as

$$q_b = AS^B\ (\ Sq - k) \tag{3.9}$$

where A and B are constants dependent upon the specific gravity and mechanical composition of sand and

$$k = Sq_c = \frac{100}{6}\ D_{50}\ (\ \gamma_s - \gamma)/M \tag{3.10}$$

in which M is a sand modulus and it is obtained by refering to the arithmatic size distribution curve and dividing the area betow 50 percent line by the area above the 50 percent line. The value of A ranges from 100 to 1000 and B from 0.25 to 1.0 when using in empirical units.

Barekyan (1962) proposed a bed load equation using average velocity,

$$q_{bw} = 0.187\ \gamma\,(\ \frac{\gamma_s}{\gamma_s - \gamma}\ )\ \ qS\ (\ \frac{V - V_c}{V_c}\ ) \tag{3.11}$$

where, q_{bw} is expressed as the bed load rate by weight per unit width.

Meyer- Peter in 1934 gave the following formula

$$g_s^{\frac{2}{3}} = 39.25\ q^{\frac{2}{3}} - 9.95\ D_{50} \tag{3.12}$$

where g_s is the bed load discharge by weight per unit width per unit time and q is the water discharge per unit width.

3.3.3 Einstein's Bed Load Function

Einstein (1942,1950) departed from the mean tractive force concept. The starting point of his argument is that in turbulent flow, the fluid forces acting on the particle vary with respect to both time and space. Thus the movement of any particle depends upon the probability, P, that at a particular time and place, the applied forces exceed the resisting forces.

The probability of movement of a particle is expressed in terms of weight rate of sediment transport, the size and immerged weight of particles and a characteristic time which is a function of particle size/Fall velocity ratio. Einstein assumed that a given particle moves in a series of steps and that a given particle does not stay in motion continuously. From these considerations a transport function was developed by him as follows:

$$\phi = \frac{g_b}{g\rho_s} \sqrt{\frac{\rho}{\rho_s - \rho} \frac{1}{gD^3}} = \frac{q_b}{\sqrt{\{(S_s-1)\ gD^3\}}} \tag{3.13}$$

where, g_b is the weight rate of bed load transport per unit width. The probability is interpreted as the fraction of the bed on which, at any given time, the lift on a given particle is sufficient to cause motion. The flow parameters are based on the logarithmic velocity distribution. The expression derived is,

$$\psi = \frac{\rho_s - \rho}{\rho} \frac{D}{R'_b S} \qquad (3.14)$$

The total drag (the pull of flowing water on the bed sediment particles), is subdivided into form and surface drag and R'_b is the fraction of the hydraulic mean radius appropriate to surface drag. The resulting relationship is expressed by

$$\phi = f(\psi) \qquad (3.15)$$

The probability bed load intensity relationship is written in the following form

$$\frac{p}{1-p} = A_* \left(\frac{i_B}{i_b} \right) = A_* \phi_* \qquad (3.16)$$

where A_* is a constant and is determined by experiments, the i_B is a fraction of bed load in a given grain size and i_b is fraction of bed material is given grain size. Einstein also concluded that D_{35} was the most satisfactory grain size to use in these calculation. The above equation was further developed by Einstein (1950) in the following way:

$$1 - \frac{1}{\sqrt{\pi}} \int_{-B_* \psi_* - \frac{1}{\eta_0}}^{B_* \psi_* - 1/\eta_0} e^{-\varepsilon^2} dt = P = \frac{A_* \phi_*}{1 + A_* \phi_*} \qquad (3.17)$$

where A_*, B_* and n_o are universal constants to be determined from the experimental data and it is found that

$$A_* = 43.5$$

$$B_* = \frac{1}{7}$$

$$n_o = 0.5$$

3.3.3.1 Bed Load Equations Similar to Einstein's Bed Load Equations

A. Einstein- Brown Formula

Brown (1950) gave a modification of Einstein (1942) Bed load formula which is as follows

$$\phi = f\left(\frac{1}{\psi}\right) \tag{3.18}$$

where $$\phi = \frac{q_b}{\nu_s F_1 \sqrt{g\left(\frac{\gamma_s}{\gamma} - 1\right) D_s^3}}$$

$$\frac{1}{\psi} = \frac{\tau}{(\gamma_s - \gamma)\, D_s}$$

and: $$F_1 = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gD_s^3\left(\frac{\gamma_s}{\gamma} - 1\right)}} - \sqrt{\frac{36\nu^2}{gD_s^3\left(\frac{\gamma_s}{\gamma} - 1\right)}}$$

The quantity F_1 for fall velocity ω of sediment size D_s is given by

$$\omega = F_1 \sqrt{\left(\frac{\gamma_s}{\gamma} - 1\right) g D_s} \qquad (3.19)$$

where ν = kinematic viscosity. The bed load discharge q_b is given as volume per unit time.

B. Toffaleti Formula

Toffaleti (1969) presented a procedure for the determination of sediment transport based on the concept of Einstein (1950). In his method, he replaced the actual channel for which the sediment discharge is to be calculated, by an equivalent two-dimentional channel of width B, equal to that of the real stream and of depth R, equal to the hydraulic radius of the real stream. Then he devided the depth into four zones as shown in Fig. 2.6. These are (1) the zone of relative thickness $y/H = 2D_i/H$ (2) the lower zone extending from $y/H = 2D_i/H$ to $y/H = 1/11.24$ (3) the middle zone extending from $y/H = 1/11.24$ to $y/H = 1/2.5$ and (4) the upper zone extending from $y/H = 1/2.5$ to the stream surface. The velocity profile is represented by the power relation,

$$u = (1 + N_v)\, V\, (y/H)^{N_v}$$

where u is the instantaneous fluid velocity, N_v is given by the empirical relation

$$N_v = 0.1198 + 0.00048T$$

where T is the water temperature in degree Ferenheit. The sediment concentration distribution for each size fraction is given by a power relation for each of three upper zones.

For upper zone concentration $C_i = C_{ui} \left(\frac{y}{d} \right)^{-1.5z_i}$

For middle zone concentration $C_i = C_{mi} \left(\frac{y}{d} \right)^{-z_i}$

For lower zone concentration $C_i = C_{Li} \left(\frac{y}{d} \right)^{0.756\, z_i}$

The exponent of above three equations are given as,

$$z_i = \frac{\omega_i V}{C_z HS}$$

where ω_i is the fall velocity of the sediment of size D_i in water temperature T, S is the slope of real stream, and C_z is given by

$$C_z = 260.67 - 0.667T$$

3.4 Suspended Sediment Transport Formulae

A part of the sediment transported by the flow in stream is suspended in the flow. The weight of these sediment particles is continuously supported by the fluid. Turbulance is the most important factor in the suspension of sediment. Owing to the weight of the particles, there is settling which, however, is counter-balanced by the irregular motion of the fluid particles introduced by the turbulent velocity components. Many researchers made their contributions in this field to find out a good suspended sediment transport formula. Some of the important theories in this light is discussed below.

3.4.1 Lane and Kalinske's Approach

In 1941, Lane and Kalinske gave the following relation for suspended bed material discharge

$$C = C_a \cdot e^{-\frac{15\omega}{U_*}\left(\frac{y-a}{d}\right)}$$

where C is the suspended bed material discharge in lb/ft. sec., C_a is the concentration by weight, (y-a) is the level of reference above the bed, ω is the fall velocity in in/sec, U_* is the shear velocity, d is the depth in inch.

3.4.2 Einstein's Approach

One of the most widely recognised methods used to compute suspended sediment load is that proposed by Einstein in 1950.

He gave the equation for suspended load discharge for each size fraction $i_{sw}\, q_{sw}$.

$$i_{sw}\, q_{sw} = \left[2.303 \log \left(\frac{30.2d}{\Delta} \right) I_1 + I_2 \right] i_{Bw}\, q_{bw}$$

$$= i_{Bw}\, q_{bw} \left[P_E\, I_1 + I_2 \right] \qquad (3.20)$$

where, $I_1 = 0.216 \dfrac{E^{z-1}}{(1-E)^z} \int_E^1 \left(\dfrac{1-y}{y} \right)^z dy$

$$I_2 = 0.216 \frac{E^{z-1}}{(1-E)^z} \int_E^1 \left(\frac{1-y}{y} \right)^z \log y \, dy$$

and $E = \dfrac{a}{d}$; $Z = \dfrac{\omega}{0.4U_*'}$; $P_E = 2.303 \log \dfrac{30.2d}{\Delta}$

The values of I_1 and I_2 in terms of E for values of Z can be obtained from graphs presented by Einstein (1950), in which q_{sw} = weight of suspended sediment discharge per unit time and width; q_{bw} is the bed load discharge by weight per unit time and width; i_{sw} is the fraction of suspended sediment of given, sediment size; i_{Bw} is the fraction of bed load in a given grain size; Δ is the correction factor; 'a' is the level of reference above bed; U_*' is the shear velocity due to grain only.

3.4.3 Brooks Approach

In 1963 Brooks obtained the following equation that is similar to Einstein's (1950) relation

$$\frac{q_{sw}}{C_{md}q} = T_B \left(k \frac{U}{U_*} , Z_1 \right) \tag{3.21}$$

where q is the discharge per unit width, C_{md} is the reference concentration at $y = \frac{d}{2}$; T_B is the transport function. For known values of $k \frac{U}{U_*}$ and Z_1, $\frac{q_{sw}}{qC_{md}}$ can be obtained from a graph given by Brooks.

3.4.4 Chang et al. Approach

Chang et al. obtained the following expression for the suspended load discharge in 1967.

$$q_{sw} = dC_a \left(UI_1 - \frac{2U_*}{k} I_2 \right) \tag{3.22}$$

where I_1 and I_2 are integrals that can be evaluated from graphical plots.

3.4.5 Bagnold's Stream Power Approach

Bagnold in 1966 gave the following formula for suspended load.

$$\left(\frac{\gamma_s - \gamma}{\gamma}\right) q_{sw} = 0.01\ \tau_o \frac{U^2}{\omega} \tag{3.23}$$

where q_{sw} is the suspended load discharge expressed as dry weight per unit time and U is the mean velocity.

3.4.6 Velikanov's Gravitational Theory

It 1954, Velikanov obtained a transport equation based on gravitational theory which is as follows:

$$q_{sv} = \frac{\gamma}{\gamma_s - \gamma} \frac{\tau_o U^2}{\omega} - \frac{\gamma}{\gamma_s - \gamma} \frac{bU^4}{g\omega} \tag{3.24}$$

where q_{sv} is the volume rate of discharge and 'b' is an experimentally determined co-efficient.

3.5 Total Load Formulae

The total load is obtained by addition of the bed load and the suspended load. Besides, this somehow indirect approach of the addition of the two fractions, there exist more direct approaches. In these cases, researchers establish a relationship which is immediately compared with measurements of the total load.

A more correct name for the total load is, actually, bed material load.

3.5.1 Colby's Approach

After investigating the effect of mean flow velocity, shear, shear velocity, stream power, flow depth, viscosity, water temperature and concentration of fine sediment, Colby (1964) developed four graphical relations for determining the bed material discharge.

The true sediment discharge q_T, corrected for the effect of water temperature, presence of fine suspended sediment and sediment size is given by

$$q_T = \left[1 + (k_1 k_2 - 1)\ 0.01 k_3\right]\ q_{Ti} \qquad (3.25)$$

where k_1, k_2 and k_3 are the correction factors found from graphs. $k_1 = 1$, when the temperature is 60°F. $k_2 = 1$ when the concentration of fine sediment is negligible and $k_3 = 100$ for D_{50} lies between 0.2 mm to 0.3 mm. q_{Ti} = Incorrect sediment discharge.

3.5.2 Engelund - Hansen Formula

Engelund and Hansen (1967) developed a sediment discharge formula which was based heavily on data from experiments in a specific series of tests in a large flume. The sediments used in this flume had a median fall diameter of 0.19 mm, 0.27 mm, 0.45 mm and 0.93 mm. The equation can be written as,

$$q_b = 0.05\ \gamma_s V^2 \sqrt{\frac{D_{50}}{g(\frac{\gamma_s}{\gamma} - 1)}} \left[\frac{\tau_o}{(\gamma_s - \gamma) D_{50}}\right]^{\frac{3}{2}} \qquad (3.26)$$

All the variables in previous equations have already been discussed. Since the equation is dimensionally homogeneous it can be used with any consistent set of units.

3.5.3 Inglis-Lacey Formula

Inglis and Lacey developed the following formula basing the data from large scale irrigation canal.

$$g_s = 0.562\ \frac{(\nu g)^{\frac{1}{3}}}{\omega}\ \frac{v^2}{gH}\ \frac{\gamma v^3}{g} \qquad (3.27)$$

in which ω is the fall velocity of a characteristic sediment particle which is assumed to be particle having the median size of the bed material. g_s is the sediment discharge in lbs/sec-ft. The other quantities have been defined previously. Since the equation is dimensionally homogenous it can be used with any consistent set of units.

3.5.4 Ackers White Formula

Based on dimentional analysis and physical considerations, a general function was developed by Ackers and white in 1973. The various co-efficients were derived using a wide range of flume data. The general function is

$$G_{gr} = C \left(\frac{F_{gr}}{A} - 1 \right)^m \tag{3.28}$$

where

$$G_{gr} = \frac{XH}{S_s D} \left(\frac{V_*}{V} \right)^n \tag{3.29}$$

$$F_{gr} = \frac{V_*^n}{\sqrt{gD(S_s-1)}} \left[\frac{V}{\sqrt{32} \log \left(\frac{10H}{D}\right)} \right]^{1-n} \tag{3.30}$$

in which m, C and A are given in terms of D_{gr}, the dimentionless particle size and is defined as

$$D_{gr} = D \left[\frac{g(S_s-1)}{\nu^2} \right]^{\frac{1}{3}} , \tag{3.31}$$

Here X is the sediment transport, mass flux per unit mass flow rate.

For coarse sediment, $D_{gr} > 60$

$n = 0.0 \qquad A = 0.17$

$m = 1.50 \qquad C = 0.025$

For transtional sizes $1 < D_{gr} \leqslant 60$

$n = 1.0 - 0.56 \log D_{gr}$

$$A = \frac{0.23}{\sqrt{D_{gr}}} + 0.14$$

$$m = \frac{9.66}{D_{gr}} + 1.34$$

$$\log C = 2.86 \log D_{gr} - \log (D_{gr})^{2} - 3.53$$

The general function of Ackers and white is based on flume data with sediment size in the range 0.4 mm <D <4.0 mm. A limitation of Froude number of <0.8 is imposed pending further investigation.

3.5.5 Shen and Hung's Approach

Shen and Hung (1971) recomanded the use of a regression equation based on available data for immediate engineering purpose. The disadvantage of this approach is that the final flow parameter can be made dimentional. The formula proposed by Shen and Hung can be expressed as:

$$\text{Log } C = -107404.459 + 324214.747 X - 326309.589 X^{2} + 109503.872 X^{3} \quad \ldots \quad (3.32)$$

Where $X = \left[\frac{VS^{0.57}}{\omega^{0.32}}\right]^{0.0075}$

A major limitation of the relation is its independence of depth of flow caused by the analysis of data covering a limited depth range.

3.5.6 Yang Formula

The approach proposed by Yang (1973) has provided one of the more recent formula for evaluating the total sediment transport rate. He approached the total transport from the energy expenditure point of view and related the transport rate to stream power. By using available data, most of which is of laboratory origin and with the help of multiple regression techniques, the following expression was obtained.

$$\log C = 5.453 - 0.286 \log \frac{\omega D}{\nu} - 0.457 \log \frac{U_*}{\omega} + \left(1.799 - 0.409 \log \frac{\omega D}{\nu} - 0.314 \log \frac{U_*}{\omega}\right) \log \left(\frac{VS}{\omega} - \frac{V_{cr}S}{\omega}\right) \qquad (3.33)$$

The value of $\frac{V_{cr}}{\omega}$ is given by

$$\frac{V_{cr}}{\omega} = \frac{2.5}{\log \frac{U_* D}{\nu} - 0.06} + 0.66; \qquad 1.2 < U_*D < 70$$

and

$$\frac{V_{cr}}{\omega} = 2.05; \quad 70 \leqslant \frac{U_* D}{\nu}$$

3.5.7 Holtorff's Equation

Based on the concept that the total power of fluid and sediment flow is constant i.e., as the power of the sediment flow increases, the power of the fluid decreases, Holtorff (1983) suggested the following expressions to compute the total sediment discharge in alluvial stream.

For plane bed, $$q_{bp} = \frac{(\tau'_o - \tau_c)V}{3(1-n')\,\rho g(S_s-1)\,\tan\phi} \qquad (3.34)$$

where n' is the grain coefficient = 1 + 0.3 log R_b in which R_b is the boundary Reynold's number and is given by the following expression

$$R_b = 0.195\,\frac{VD}{\nu}$$

For deformed bed (dune) the expression is

$$q_b = q_{bp}\,(1-i) \qquad (3.35)$$

where $i = \dfrac{0.285\,k\Delta}{(f'-f_c)\,(fR_e)^{\frac{1}{2}}}$ (i varies 0 to 0.5)

For antidune, $i = 0$; in which $k = \dfrac{2\pi}{L}$

and, L = wave length

R_e = Renolds number of flow

f = Dorcy - Weisbach factor and f' and f'_c are related to

$$f' = \frac{8\tau'_o}{V^2\rho} \quad ; \qquad f_c = \frac{8\tau_c}{V^2\rho}$$

and $\tau_c = 0.047\; g\rho(S_s-1)\,D_{50}$

3.5.8 Mantz Equation

Mantz in 1983 suggested a semi - empirical equation to calculate sediment flow for both fine and coarse materials. His formula is expressed as follows

For Bed Load Discharge

$$q_b = \frac{2.57 \times 10^{-4} (P'_\omega)^{2.90}}{H\sqrt{D_b}} \qquad (3.36)$$

For D_b = 0.01 mm to 0.2 mm and

H = 0.03 m to 0.12 m

$$q_b = \frac{6.17 \times 10^{-4} (P'_\omega)^{1.50}}{H\sqrt{D_b}} \qquad (3.37)$$

For D_b = 0.2 mm to 3.00 mm

and H = 0.12 m to 12 m

For Suspended Load discharge

$$q_s = 1.26 \times 10^{-2} (P'_\omega)^{1.03} \frac{V}{\omega} \qquad (3.38)$$

where P'_ω is the excess stream power and expressed as

$P'_\omega = (P_\omega - P_{\omega c})$; Here $P_{\omega c}$ is the critical stream power. The total sediment discharge is obtained by summing up the above two equation i.e., bed load discharge+ suspended load discharge.

3.5.9 Strathclyde (Hossain,1984) Formula

In 1984, Dr. Hossain suggested a sediment transport formula of semi empirical nature as follows:

$$C = A \left[X^a \; Y^b \; Z^c \right] \tag{3.39}$$

where :

C = the sediment concentration in ppm.

$A = 0.845 \times 10^5$ for $Q \leqslant 1.0$ cumecs

$= 6.946 \times 10^5$ for $Q > 1.0$ cumecs

$X = \dfrac{VS}{\sqrt{gH}}$ $a = 0.745$

$Y = \dfrac{\omega r}{\omega}$ $b = 0.633$

$Z = \dfrac{Q}{Q_c}$ $c = 0.50$

ω_r = settling velocity for a representative sediment size, for which D_{50} = 0.15 mm at ambient temperature

ω = settling velocity of the sediment load

Q_c = assessed water discharge and is calculated as

$$= \left[\left(2.15 + 0.205 \frac{B}{H} \right) H \; (gs)^{1/5} \right]^{5/2} \tag{3.40}$$

It should be noted here that the sediment transport function developed represent a median condition of correlation.

CHAPTER - 4

THE RIVER GORAI-MODHUMOTI-ITS BEHAVIOUR, SEDIMENT CHARACTERISTICS AND FLOW CONDITION

4.1 Physiographical Description of Gorai-Modhumoti Basin

The Gorai-Modhumoti basin physiographically falls under the category of deltaic plain. Again owing to physical feature and drainage pattern, this deltaic basin can further be divided into the Moribound delta, the central delta and the mature delta (Fig. 4.1; Ahmed, N., 1968).

The upstream of Gorai-Modhumoti falls under moribound delta plain. If an imaginary line is drawn from north of the Faridpur town in a South-Westerly direction to south of Satkhira, then in general all the area north and west of this line would lie in the moribound delta plain, an area of dead and decaying rivers. This part is somewhat higher and free from regular annual inundations. No tidal water reaches the river and it generally remains confined within its high banks, except during the rainy season. A part of the Ganges water flows in the river Gorai-Modhumoti. The distributaries of the river Gorai have deteriorated and become chocked with vegetation and weeds. From the air it present a fantastic picture of dirty, a green bands, creeping into the country side (Hossain, L., 1974).

The central part of this basin falls under the central deltaic plain. This area is commonly known as Faridpur bill area and it is about 900 square miles. The cause of these extensive bills is due to the absence of rapid deposition by the active distributaries (which flow towards its east) coupled with steady subsidence due to warping by torsional forces (Haroun Er Rashid, 1967).

The tail end of Gorai-Modhumoti basin falls under mature deltaic plain. If an imaginary line is drawn from east to west of Faridpur bill area, the southern part of this line falls under Mature Deltaic plain. The land is slightly higher. Tidal excursion are experimented in this zone. The rise of tide has led to rapid deposition of the silt carried by the rivers. The formation of this ledge has proceeded with compaction of the deposits, which made the depression along the east-west line to the north of it (Hossain,L., 1974).

4.2 Origin and Course of the River

The gorai is the main distributary of the river Ganges. It originates from the right bank of the river Ganges near Talbaria of Kushtia district. In the downstream of Kamarkhali the river is renamed as Modhumoti and the Gorai-Modhumoti course flows in the South-east direction. The main river bifurcates and rejoins several times as it flows south-east to Muhammadpur Upazilla of Magura district, from there flowing South- southwest direction. The Kumar, Nabaganga and Chitra join it through several channels south of Mollahat Upazila (Fig. 4.2). There the name changes to Baleswar, which in turn take the name of Haringhata from the Bogi forest out post of the Sunderbans.

4.3 Present Flow Condition of Gorai-Modhumoti

During the last three decades, diversion of Ganges water to the Gorai has been highly variable due to moving sandbars which periodically seriously obstruct flow into Gorai. Flows into the Gorai were completely cut-off in 1976, but this stopage was largely due to the histroic low discharges and water levels in the Ganges river (IECO,1981).

In 1981, the maximum flood discharge of the river Gorai-Modhumoti at Gorai Rly. bridge water measurement station was about 2,34,000 cusecs and in the winter it was as low as 157cfs table 4.1. Actually, in winter no water of the Ganges falls into the Gorai and hence the upstream remains almost dry. But in the rainy season huge amount of water from the Ganges river pass through the Gorai river.

4.4. Causes and Effect of Low Discharge at Gorai

The water discharge at Gorai offtake largely depends on the flow condition of the Ganges, the channel geometry, the flow in the Brahmaputra river (causing backwater effects at the Gorai offtake) and sand bars formation at the upstream of the river Gorai.

The minimum discharge in Gorai at Gorai railway bridge water measurement station (8 miles downstream of Gorai offtake) in 1981 was recorded 157 cusecs and day by day this discharge is being lowered. This low discharge effects Khulna industrial area where insufficient flow rates causes saline water intrusion. Besides, the Gorai Irrigation project, Magura-I Irrigation Project, Chenchuri beel irrigation project, Old river Resuscitation project, Baliakandi and Barasia Irrigation project are adversely affected due to lowering of water levels at Gorai offtake.

It is estimated by the special studies, BWDB (March,1981) that the minimum flow in Gorai for extreme low water condition would be approximately 6,730 cusec and the corresponding minimum discharge at Ganges at Hardinge Bridge measurement station would be 22,200 cusecs, to save the aforsaid projects.

4.5 <u>River Section at Gorai Rly. Bridge</u>

The section at Gorai Rly. Bridge is about 8 miles downstream of Gorai offtake. At this section the river is meandering. Water level and discharge data are available in Hydrology department of Bangladesh Water Development Board since 1964. BWDB is also collecting sediment data since 1964 but not regularly. It is also observed that the river at this section is eroding its left bank.

4.6 Sediment Discharge and Hydrographs

It is frequently observed that the sediment discharge and water discharge do not always increase or decrease simultaneously. Jarocki (1963) suggests that this is owing to the difference in the cause of two effects. Intensive sediment transport from the river basin does not, necessarily, coincide with the occurance of maximum flow rates. For small and homogeneous watersheds, the two peaks usually coincide, since the runoff or rain is responsible for both of them. For large rivers it is often reported that the peak sediment discharge depends on the hydrologic system of the watershed and the water velocity. The observations by Einstein et alia (1940) for the Enoree River, by Jarocki (1963) for the Vistula and Volga, and by Nordin et al. (1963) for the Rio Grande, all exhibit this trend.

The discharge hydrograph and sediment concentration of Gorai-Modhumoti are shown in Fig. 4.3, 4.4 and 4.5. Figures illustrate that the peak sediment discharge generally occurs a little later than the peak of the water discharge. Figure 4.6, 4.7, 4.8 and 4.9 shows the stage discharge curve for this river.

CHAPTER - 5

DATA COLLECTION, ANALYSIS, RESULTS AND DISCUSSION

5.1 Source of Data

The basic data have been collected from Bangladesh Water Development Board (BWDB). Water discharge and other associated flow measurement data have been collected from the Directorate of Surface Water Hydrology, BWDB, Dhaka. The suspended sediment concentration have also been collected from the Surface Water Hydrology Directorate, BWDB. The bed material size have been obtained from River Research Institute, BWDB, Dhaka. All the data have been collected from Gorai Railway bridge station (Fig.4.2). Besides, the water level data at Kamarkhali station have been collected to determine the water surface slope. But a considerable amount of time have been spent in the assimilation and compilation of the data scattered in different stations of BWDB.

The above data were available at BWDB since 1964 and only data from 1976 to 1984 have been considered in this study. Further more, sediment concentration data were not collected by BWDB on a regular basis and were often not in a usable form. All data have been arranged systematically and brought into proper form in a manner shown in table 5.1.

5.2 Field Measurement Procedure

The BWDB has been collected sediment concentration and flow data for Gorai river at Gorai Railway bridge station. Water flow and sediment concentration data and their various parameters were calculated in the following way:

i) Water discharge (q): For each sub-section delineated by two adjacent verticals the discharge is obtained as the product of the average of velocities at the two verticals and the area of the sub-section. The total flow discharge (Q) for the entire section is obtained by summing up the discharges for all the sub-sections.

ii) Water Surface width (B): It is the sum of the widths of all the sub sections.

iii) Cross-Sectional area (A): From the measured depth at the verticals, the distance between verticals and water surface elevation, the transverse bed profile is plotted, keeping the left bank at the left edge of the paper. The top boundary of the cross-section is the horizontal line forming the water surface width while the rest of it is the river bed profile. The cross-sectional area is then calculated by summing up the areas of all the sub-sections.

iv) Mean Velocity (V): Mean velocity has been obtained dividing the water discharge by cross-sectional area.

v) Mean depth (H): Mean depth of flow has been determined from equivalent rectangular channel section whose top lateral dimension equals the water surface width. Thus the water section area divided by the water surface width gives the mean depth.

vi) Water Surface Slope (S): Water surface slope has been determined from simultaneous staff Gauss readings at Gorai railway bridge station and those situated downstream of Kamarkhali. The difference of the two readings is divided by the total length of water course yields the water surface slope.

The length of water course between above two Gauge Stations was recorded 67.1 km by the Directorate of special studies, BWDB.

5.3 Sediment Concentration and its Measuring Instrument

The sediment concentration has been determined by BWDB with a sampler known as Binkley silt sampler which consists of two brass made hollow pipes mounted at the two ends of a rubber sleeve. The rubber sleeve again have two valves which can manually be operated. During operation the instrument is lowered into the water at a certain depth and suddenly the inlet and outlet valves of the sleeve is stopped manually. The samples collected in the manner is then transfered to a plastic container for computation of sediment concentration.

The suspended sediment concentration has been taken by BWDB at 0.2 and 0.8 depths of each sub-section and average of the two is the mean concentration for each sub-section. The average sediment concentration for a section is then calculated by summing up all the concentrations divided by the number of sub-sections.

5.4 Other Sediment and Water Flow Parameters

Besides, the data as described above the following parameters are required to compare the performance of various sediment transport formulae.

i) Effective sediment size (D_{50}): The grain size analysis of bed material is available in BWDB only for the year 1964. From grain size analysis (Fig. 5.1) the median grain size (D_{50}) is obtained as 0.048 mm. Again from table-I the above sediment size falls under the category of coarse silt.

ii) Water temperature (T): The water temperature data were not collected in the field. Two standard average values of temperature i.e., $30^{o}C$ (April to October) and $20^{o}C$ (November to March) is assumed in this study.

iii) Fall velocity (ω): As discussed earlier, many formulae and graphical relationships are used in computation of fall velocity. The graphical relationship as suggested by Raudkivi (Fig. 2.1) has been adopted in present study for computation of fall velocity. From this relationship two values of fall velocity, ω_{30} = 0.3 cm/sec. and ω_{20} = 0.21 cm/sec. are considered in this study at $30^{o}C$ and $20^{o}C$ respectively.

iv) Kinematic viscosity (ν): The kinematic viscosity is the ratio of the absolute viscosity to density; $\nu = \frac{\mu}{\rho}$ and has units of square meters per second. The standard value of kinematic viscosity at $30^{o}C$ and $20^{o}C$ is assumed as 8.59×10^{-7} m^2/sec and 1.01×10^{-6} m^2/sec respectively in this study.

v) Specific weight (γ): The specific weight is computed by the formula, $\gamma = \rho g$; where ρ is the density and g is the acceleration due to gravity. The standard value of density of water (ρ) and the density of sediment (ρ_s) is considered in this study as 1.0 and 2.65 respectively.

5.5. Performance of the Formulae

5.5.1 Strathclyde Formula (Hossain 1984)

Figure 5.2 shows the graphical comparison of the calculated and measured sediment discharges. The best fit line has been drawn by regression analysis. The lines of perfect agreement have also been shown in the figure. The scatter in the figures may be due to the appreciable quantity of bed and wash load present in the measured total load.

The ratio of calculated to the observed sediment discharge (Descripency ratio) varies between 0.52 and 16.6 with a mean value of 2.77. From the table 5.2 it is apparent that about 60% of the data fall within the range of descripency ratio from 0.5 to 2.0 values. This indicates the closeness of the data about the mean line is better. Hence, it may be inferred that the results obtained by Strathclyde formula are consistently good.

5.5.2 Mantz Equation (1983)

Results obtained by Mantz formula provides relatively a better correlation. Figure 5.3 shows the comparison between the calculated and observed sediment discharge for the Gorai river. In table 5.2 there has been depicted that about three-fourths of the data have a range of descripency ratio from 0.5 to 2.0. This indicates that most of the calculated values lie nearer to the mean line. So the predictive performance of this formula against the field data of the river Gorai appears to be acceptable. Again, the range of descripency ratio varies from 0.37 to 6.71. The scatter of the points about the mean line may be attributed to the presence of bed and wash load. In addition to that the equation has been derived on the data obtained from laboratory channels that have a particular depth of flow and a given range of sediment size.

5.5.3 Engelund-Hansen Formula (1967)

The performance of the formula against the present sets of data is shown in figure 5.4. The ratio of calculated sediment discharge to the observed sediment discharge varies from 0.14 to 5.0 with a mean value of 1.26. About 56% data have a range of descripency ratio from 0.5 to 2.0. This indicates that about 56% of the calculated values of sediment flow fall within $\pm$ 100 percent region of perfect agreement line. The predicted values fluctuate from seven times lower to five times greater than the measured ones which indicates a moderate performance of the formula.

5.5.4 Yang Formula (1976)

This formula yields a reasonable performance against the data in flood season but during very low discharge in the winter it provides very unrealistic prediction as shown in table 5.3 and 5.4 As such, about 44% data were not found suitable to use in the study. Originally this formula has been derived on the basis of the study of the data of sand bed flumes and rivers having fine and coarse sand. But the bed material size of the present study falls under the category of coarse silt 0.048 mm). These may be attributed to the poor correlation of this formula.

About 32% of the data have a range of descripency ratio from 0.5 to 2.0. This indicates the poor coverage of data about the mean line, as shown in figure 5.5.

5.5.5 Ackers-White Formula (1973)

The performance of this formula against present sets of data is shown in table 5.3 & 5.4.This prediction is quite erratic. This may due to the fact that the sediment transport rates largely depends on the significant particle size. This formula is well defined for coarse and transitional sediment sizes but for fine sediments like the present study, this formula states nothing.

About ninty percent data falls outside the range of descripency ratio from 0.125 to 8.0. The maximum descripency ratio is as high as about 450 times, whereas the minimum descripency ratio is as low as about to zero. This warrant the recommendation of this equation with the present sets of data.

5.6 Comparative Study

The formulae to be accepted as suitable for the river Gorai-Modhumoti have been selected on the basis of five criteria viz,(i) Maximum data coverage for the descripency range of 0.5 to 2.0. (ii) The range between the highest and lowest descripency ratio (iii) The percentage of data applicable to the formulae. (iv) Standard deviation and (v) Coefficient of correlation.

Of the five formulae examined in this study, the Mantz equation predicts transport rates which are comparatively closer to the observed values. About 77% data lies within the descripency ratio range from 0.5 to 2.0 (Fig. 5.6). The minimum and maximum descripency ratio is 0.37 and 6.71 respectively. Again it is found that about 25% of the predicted value lies within the range of 85% accuracy of the observed values. Considering the above facts, the Mantz Equation shows the best performance of all the five formulae against the data of present study.

About 60% data calculated by the Strathclyde formula have a descripency ratio range from 0.5 to 2.0 (Fig. 5.7). The maximum and minimum descripency ratio is 16.6 and 0.52 respectively. It is apparent that about 5% of the data have a very high descripency ratio (above 8.0). Besides, about 20% of the predicted value lies within range of the 80% accuracy of the observed values. From these considerations the Strathclyde formula may be regarded as a good one for this study.

The maximum and minimum value of descripency ratio for the Engelund-Hansen formula is 0.14 and 5.0 respectively. Moreover about 56% data fall within the descripency ratio of 0.5 to 2.0 (Fig. 5.8). This also proves the formula to be a stable one for the present sets of data of the Gorai river.

As far as the Acker-white formula is concerned, there exist no consistent trend of variation of predicted rates. At times the formula under estimates and at times it abruptly over estimates the transport rates. Thus it appears from this study that the formula lacks confidence in the prediction against the field data of present study.

Although the predicted values of Yang formula during high flow as compared to the observed values are good, the formula does not respond during low flow condition. So, it is apparent that the Yang formula has shown poor performance against the data of this study (Fig. 5.9). Though the results obtained by Yang formula are not good, this seems to be better compared to the results obtained by the Ackers-white formula (table-5.2).

In this study, as it appears that the Mantz and the Strathclyde formula has provided comparatively better prediction against the present sets of data. This is then followed by the Engelund-Hansen formula, the Yang formula and the Ackers-white formula. This is shown in table-5.3 and table-5.4. The performance of the formulae in prediction of total load, as compared to the observed suspended load is shown in Fig. 5.10 and 5.11.

5.7 The Regression Equation

These are the equations relating the observed suspended sediment rates to the total predicted sediment rates (Fig. 5.2 to 5.5). The data have been fitted by regression analysis of the power form;

$$Y = kx^{\eta} \tag{5.1}$$

where η is exponent and k is a constant. The equations obtained in this study are:

$$C_c = 78.52\ C_T^{0.235} \quad \text{(Strathclyde; Hossain 1984 formula)} \tag{5.2}$$

$$C_c = 4.315\ C_T^{0.691} \quad \text{(Engelund-Hansen formula)} \tag{5.3}$$

$$C_c = 2.455\ C_T^{0.70} \quad \text{(Yang formula)} \tag{5.4}$$

$$C_c = 19.91\ C_T^{0.406} \quad \text{(Mantz Equation)} \tag{5.5}$$

Here C_c and C_T are calculated and observed sediment concentration in ppm respectively.

5.8 Sediment Rating Curves

These are the curves relating between the (i) observed suspended sediment discharge and the water discharge (fig. 5.12). (ii) Observed suspended sediment discharge and average velocity (fig.5.13). (iii) Observed suspended sediment rate and water area (fig. 5.14) and (iv) Observed suspended sediment discharge and the average depth (fig.5.15). The equations found in this study by regression analysis are as follows:-

$$C_T = 4.571\ Q^{0.325} \tag{5.6}$$

$$C_T = 20.4\ V^{1.68} \tag{5.7}$$

$$C_T = 1.584\ A^{0.471} \tag{5.8}$$

$$C_T = 19.95\ d^{0.73} \tag{5.9}$$

These curves could not be compared with the curves of other investigators since similar curves were not available to the author. However, it may be used for future comparison.

5.9 Hydrogeometric Relations

At a particular section the average velocity (v) depth(d), water area (A) continuously change as water discharge (Q) changes. An increase in discharge causes an increase in each of the variables. This article shows how they vary with change of discharge.

Figures 5.16, 5.17 and 5.18 show the plots for discharge versus average velocity, water area and depth respectively. The following equations were obtained by plotting the best fit curves with regression equation analysis. These are

$$Q = 407.4\ V^{4.08} \tag{5.10}$$

$$Q = 0.42\ A^{1.21} \tag{5.11}$$

$$Q = 225.4\ d^{1.97} \tag{5.12}$$

According to Leopold and Maddock (1953), upto the bankful stage in a natural river section the relations of velocity and depth to discharge are in the mathematical form of simple power functions. The relative rate of increase of velocity and depth are determined by the shape of the channel, the slope of water surface and the roughness of the wetted perimeter. This justifies the different values of co-efficients and exponents obtained in eqn. 5.10 to 5.12. Again it may be mentioned here that the rivers flowing under different climatic and geologic conditions will change their cross sectional dimensions differently with change of discharge. This is due to the difference in the bed and bank material characteristics, the quantity and quality of sediment load, magnitude and variation of discharge and other parameters from one river to another.

5.10 Discussion

For the present sets of data the median size of bed material is 0.048 mm, the mean depth ranges from 1.31 ft. to 32.61 ft. and discharge from 157 cfs. to 2,62,800 cfs. for the river Gorai.

While judging from the above angles, especially in terms of magnitude and range of flow it may be inferred that all the formulae studied herein lack confidence in varying degrees. The recently developed Mantz Equation (1983) gives comparatively better compliance with these data. He suggestes a semi-empirical equation to sediment flow for both fine and coarse materials. About three-fourths of the data fell between the range of descripency ratio from 0.5 to 2.0.

Relatively greater advantage of the Strathclyde formula is that is simple and easy to apply. It incorporates the coefficients based on the experiments and study of about 4,500 sets of flume and field data. Considering the characteristics of the river Gorai and the availability of various geometric data, the performance of the Strathclyde formula seems to be more acceptable.

The Engelund-Hansen formula has also the advantage for its simplicity in form. It properly takes into account the effect of bed sediments on sediment transport. In the present study the performance of the formula has been found to be good. It is worth mentioning that the formula has been extensively tested by different investigators at different times and proved one of the best (Task committee, ASCE, 1971, W.R. white et el.,1973, and C.V. Gole et el., 1973).

Performance of the Yang's formula against the present sets of data is not very encouraging. This is due to the fact that the Gorai river has a large fluctuation of discharge throughout the year. Again the co-efficients of the formula are influnced mostly by flume data of Gilbert, its application in river like the Gorai possibly needs modification.

The application of the Ackers-white formula in present study appears to be doubtful as observed from its performance. Again, the formula does not take into account the bed form characteristics properly. The Ackers-white formula involves laborious calculation and thus put severe limitations for practical use (table 4.3).

CHAPTER - 6

CONCLUSION AND RECOMMENDATION FOR FUTURE STUDY

6.1 Conclusion

Five equations to compute the total sediment load in alluvial rivers have been used against the data of the Gorai-Modhumoti river. The data were obtained from Bangladesh Water Development Board for various years. Based on the study as delineated in the previous chapter, the following conclusions may be drawn:

i) The Mantz, the Strathclyde and the Engelund-Hansen formulae yield comparatively a better prediction. Values obtained by the Mantz equation have been found to be in closer agreement with the observed values while Strathclyde and Engelund-Hansen formula give consistent results in terms data coverage between the descripency ratio of 0.5 and 2.0.

ii) The relationship between observed sediment concentration and computed sediment concentration and the water discharge of the Gorai river may be correlated to give simple power equations (eqns. 5.2,5.3,5.4,5.5 and 5.6).

iii) The hydrogeometric relationship between water discharge and mean velocity, average depth and cross-sectional area of flow for the Gorai river at Gorai Rly. bridge station may be correlated to give simple rating curves (eqns. 5.10, 5.11 and 5.12).

iv) The relationship between observed sediment discharge rates and mean velocity, average depth and cross-sectional area of flow may also be correlated with the help of simple rating curves (eqns. 5.7, 5.8 and 5.9).

v) It is revealed from this study that about 13.27 million tons of suspended sediment pass through the Gorai rly. bridge of Gorai-Modhumoti river annually (table 6.1).

vi) The hydrographs of observed suspended sediment discharge and water discharge (Fig. 4.3, 4.4 and 4.5) show that the peaks of sediment hydrograph preceeds the peaks of the water hydrograph. Similar relationships were also found by Jarocki (1963) for the Vistula and Volga river.

6.2 Limitation of Present Study

The collection and analysis of data should be based on recent standard practices. The necessary data like concentration, discharge, depth, width, velocity of flow, sediment size, temperature etc. for this study were not available throughout the year and hence detail investigation pertaining to sediment movement could not done properly. Moreover, due to the lack of water temperature data, evaluation of computed sediment rates of the river Gorai was dependent upon standard value of temperature. Thus systematic approach and methodology should be developed for the collection of various geometric and hydraulic parameters.

6.3 Suggestion for Further Investigation

In view of the limitations cited in the previous article, the following suggestions are made for future study:

1) More data should be collected and compiled spreading over a large number of years to study their correlation.

2) More sediment transport equations should be considered to validify their efficacy against this river.

3) Data for more than one station and covering a wide range of time should be taken to study erosion-deposition phenomena.

4) Measurements of bed load should be undertaken. However, very good bed load sampling technique still does not exist. Possibility of determination of bed load by tracer techniques may be explored.

5) Similar investigations may be carried out on other smaller rivers of Bangladesh to formulate the relationships among the variables that influence sediment movement.

REFERENCES

1. Ackers, P., 1972, "Sediment Transport in Channels, An Alternative Approach", Report INT 102, Hydraulics Research Station, Wallingford, England, March.

2. Ackers, P., and White W.R., 1973, "Sediment Transport New Approach and Analysis", Journal of the Hydraulics Division, ASCE, Vol. 98, No. HY 11, November pp 2041-2060.

3. Allen, J.R.L. 1965, "A Review of the Origin and Characteristics of Recent Alluvial Sediments, Sedimentology, Vol. 5, No. 2, pp- 89 to 191.

4. Alam, M.K., 1977, "Studies of Hydraulic Factors Controlling the Geometry of Non-Tidal and Tidal Water Coarses", Ph.D. Dissertation, Department of Civil Engineering, University of Strathclyde, Glassgow, U.K., January.

5. Anderson, A.G. 1953, "The Characteristics of Sediment Waves Formed by Flow in Open Channels", Proceedings, Third Midwestern Conference on Fluid Machanics, University of Minnesota, Mineopolis, June, pp. 379-395.

6. Ahamed, N., 1968, "An Economic Geography of East Pakistan (Bangladesh)", Oxford University, London.

7. Banton, J.K. and Lin Pin-Nam, 1955, "A Study of Sediment Transport in Alluvial Channels", Report No. 55 JRB2, Colorado Agricultural and Mechanical College, Civil Engineering Department, March.

8. Bari, M.F., 1978, "Applicability of Sediment Transport Formulas For the Ganges and the Jamuna", M.Sc., Dissertation. Department of Civil Engineering, Bangladesh University of Engineering & Technology, Dhaka.

9. Blench, T., 1966, "Mobile Bed Fluviology", Dept. of Technical Services, University of Alverta, Canada.

10. Bogardi, J.L. 1958, "The Total Sediment Load of Stream, A Discussion", Journal of Hydraulics Division, ASCE, Vol.84 No. HY6.

11. Chang H.H., 1980 "Stable Alluvial Channel Design", Journal of the Hydraulics Division, ASCE, Vol. 106, No. HY5, Proc. Paper 15420, May, PP 873-891.

12. Colby, B.R. and Hubbel, D.W., 1961, "Simplified Methods for Computing Total Sediment Discharge with Modified Einstein Procedure", Water Supply paper No. 1593, United States Geological Survey, Washington D.C.

13. Colby, B.R., and Hembree, C.H., 1955 "Computations of Total Sediment Discharge, Niobrara River near Cody, Nebraska," U.S. Geological Survey Water Supply Paper 1357, U.S. Government Printing Office, Washington. D.C.

14. Coleman, J.M., 1969, "Brahmaputra River Channel Process and Sedimentation", International Journal of Applied and Regional Sedimentology, Vol. 3, No. 2/3, Amsterdam, The Netherlands, August.

15. Dawdy, D.R., 1961, "Depth- Discharge Relations of Alluvial Streams- Discontinuous Rating Curves", Water supply paper 1948-C, United States Geological Survey, Washington D.C.

16. Einstein, H.A., 1950, "The bed load Function for sediment Transportation in open channels, Technical Bulletein 1026, United States Department of Agriculture, Soil conservation Service, Washington, D.C.

17. Engelund, F., and Hansen, E., 1967, "A monograph on sediment Transport in Alluvial Streams, Teknist Vorlag, Copenhagen, Denmark.

18. Engelund, F., 1969, "Dispersion of Floating particles in Uniform channel Flow", Journal of the Hydraulics Division, ASCE, Vol. 95, No. HY4, July, pp, 1149-1162.

19. Gole, C.V., Tarapore, Z.S. and Dixit, J.G., 1973, "Applicability of sediment transport formulae to natural streams", Proc., 5th Congress of the IAHR, Istanbul, Vol. 1, P. 429-436.

20. Grade, R.J., and Albertson, M.L., 1958, "The Total sediment load of streams," Journal of the Hydraulics Division, ASCE, vol. 84, No. HY6.

21. Graf, W.H., 1971, Hydraulics of Sediment Transport, McGraw Hill Book Company, Inc., New York.

22. Griffith, W.M., 1939, "A theory of silt Transportation", Transactions, ASCE, vol. 104, paper No. 2052, pp 1733-1748.

23. Hossain, L., 1974, "Water Resources Planning for regional development of Bangladesh", M.Sc. Dissertation, Dept. of WRE, BUET,Dhaka.

24. Hossain, M.M., 1984, "Development of a Sediment Transport Formula", Ph.D., Dissertation, Department of Civil Engineering, University of Strathclyde, Glassgow, United Kingdom.

25. Hossain. M.S., 1970, "Sediment Transport Characteristics of the Teesta River", M.Sc. Dissertation, Department of Civil Engineering, Bangladesh University of Engineering & Technology, Dhaka.

26. Hydraulics Research Laboratory, 1971, "Sediment Testing Report: Analysis of Bed Material and Suspended Sediment Samples of River Jamuna and Ganges" BWDB, Dhaka Report No. sed-79, December.

27. Hydraulics Research Laboratory, 1974, "Sediment Testing Report: Analysis of Bed Material and Suspended Sediment Samples of River Jamuna and Ganges", BWDB, Dhaka Report No. sed-85, June.

28. Hubbel, D.W. and Matejka, D.Q., 1959, "Investigations of Sediment Transportation, Middle Loup River at Dunning, Nebraska", Water supply paper No. 1476, U.S. Geological survey, Washington, D.C.

29. Hunt, J.N., 1954, "The Turbulent Transport of Suspended Sediment in Open Channels", Proceddings, Royal Society of London, series-A, Vol. 224, No. 1158, pp. 322-335.

30. Ippen, A.T., 1971, "A New Look at Sedimentation in Turbulent Streams", Journal of the Boston Society of Civil Engineers, Vol. 58, No. 3, July, pp-131-163.

31. Inman, D.L., 1952, "Measures for Describing the size Distribution of sediments", Journal of sedimentary perology, vol. 22, No. 3, pp. 125-145.

32. Jeffries, H., 1929, "Transport of sediments by streams", Proceddings, cambridge Phylosophical Society, vol. 25, Cambridge, England.

33. Jobson, H.E., and Sayre, W.W., 1970, "Predicting concentration profiles in open channels", Journal of the Hydraulic Division, ASCE, vol. 96. No. HY10, October, Paper 7618, pp. 1983-1986.

34. Kalinske, A.A., and Hsia, C.H., 1945, "Study of Transportation of fine sediment by flowing water", Iowa University, Studies in Engineering, Bulletin No. 29.

35. Lane E.W., et al., 1947, "Report of the sub-committee on sediment Terminology", Transactions, American Geophys. Union, vol 28, No. 6 pp. 936-938.

36. Laursen. E.M., 1958,"The total sediment load of Streams", Journal of the Hydraulics Division, ASCE, vol. 84, No. HY-1, February pp. 1-36.

37. Mantz, P.A., 1983, "Semi-empirical correlations for fine and coarse cohesionless sediment transport", Proc. ICE, Part 2, Vol. 75, P. 1-33.

38. Maddock, Thomas, Jr. 1973, "A role of sediment Transport in alluvial channels", Journal of the Hydraulics Division, vol. 99 No. HY-11, November, pp. 1915-1931.

39. Meyer- Peter, E. and Muller, R., 1948, "Formulas for bed load Transport", REport of 2nd meeting of I.A.H.R., Stockholm.

40. Nichools, M., and Poor, G., 1967, "sediment Transport in a coastal plain Estuary", Journal of the waterways and Harbour Divn. ASCE, vol. 93, No. WW4, Proc. paper 5571, November, pp. 83-95.

41. Nordin, C.F. Jr., 1964, " Aspects of Flow Resistance and Sediment Transport: Rio Grande near Bernalillo, New Mexico", water supply paper 1498-H, United states Geological survey, washington, D.C.

42. Novak, P., and Nalluri, C., 1975, "Sediment Transport in smooth Fixed Bed channels", Journal of the Hydraulic Division, ASCE, vol. 101, No. HY-9, September, pp. 1193-1154.

43. Paintal, A.S., and Grade, R.J., 1964, "Discussion of sediment Transportation Mechanics: Suspension of sediment", by the Task Committee on preparation of sedimentation Mannual, Journal of Hydraulics Divn. ASCE vol. 90 No. HY-4 proc. paper 3980, July pp. 257-265.

44. Raudkivi, A. J. 1971, discussion of "Sediment Transportation Mechanics: Hydraulics Relations for Alluvial streams", by the Task Committee on preparation of sedimentation Mannual, Journal of the Hydraulics Divn. ASCE, vol 97, No. HY-12, Proc. paper 8552, December, pp. 2089-2093.

45. Rashid, H., 1967, "A systematic Regional Geography and Its Development Planning Aspects", (Lahore: Sh. Ghulam Ali and Sons.).

46 . Ranga Raju, K.G.; R.J. Grade and R.C. Bhardwaj, 1981, "Total Load Transport in Alluvial channels", JHD, Proc. ASCE, Vol. 107, No. HY-2, February.

47. Shen, H.W., Ed., 1971, River Mechanics, vol. 1, Fort Collins, Colo.

48. Shen, H.W., and Hung, C.S., 1971, "An Engineering Approach to Total Bed Material Load by Regression Analysis", Sedimentation symposium to Honour Prof. H.A. Einstein H.W. Shen, ed., Fort collins, Colo.

49. Simons, D.B., and Senturk, F., 1977, sediment Transport Technology Water Resources Publications, Fort Collins, Colo, U.S.A.

50. Sutherland, A.J., 1967, "Proposed Mechanism for sediment Entrainment by Turbulent Flows", Journal of Geophysical Research vol. 72, No. 24, pp. 6183-6194.

51. Task Committe for Preparation of Sedimentation Manual, 1971, "Sediment Transport Mechanics: H. sediment Discharge Formulas", Journal of the Hydraulics Divn. ASCE, vol. 97, No. HY-4, Proc. Paper 7786, April, pp. 523-567.

52. Tisen, L.J., 1953, "Studies of the Critical Tractive Forces for the Entertainment of "Bed Materials" Proceddings, Minneotu International Hydraulics Conference, Minneapoils, Minn., pp. 21-35.

53. Toffaleti, F.B. 1969, "Definitve Computations of Sand Discharge in Rivers", Journal of the Hydraulics Division, ASCE, vol 95, HY-1, January, pp. 225-246.

54. Turtle, R., 1952, "The Hydraulic Conveying of Granular Materials", thesis presented to London University, at London, England in partial fulfilment of the requirements for the degree of Doctor of Phylosopy.

55. United States Bureu of Reclamation, 1958, "Total Sediment Transport Program, Lower Colorado River Basin, Denver, Colorado", Interim REport, January.

56. Vanoni, V.A., 1946, "Transportation of Suspended sediment by Water", Transactions, American Society of Civil Engineers, vol III, pp. 67-133.

57. Vanoni, V.A., 1953, "Some effects of suspended sediment on flow characteristics", Proc. Fifth Hydraulic Conference, Bulletin 34, State University of Iowa, Iowa City, pp. 137-158.

58. White, W.R., Milli, H., and Crabbe, A.D., 1975, "Sediment Transport Theories: A Review", Proceedings, Institution of Civil Engineers, London, Paper No. 7783, vol 59, June, pp. 265-292.

59. White, W.R., Milli, H., and Crabbe, A.D., 1973, "Sediment transport: An Appraisal of Available Methods", Report INT 119, Hydraulics Research station, Wallingford, England, December.

60. Wilson, K.C., 1966, "Bed Load Transport at High shear stress", Journal of the Hydraulic Division, ASCE, vol. 92 No. HY-6, Proc. paper 4968, Nov. pp. 49-59.

61. Worster, R.C., 1952, "The Hydraulic Transport of Solids", Proceddings of a Colloquium on the Hydraulic Transport of Coal, National Coal Board, London, England, November,5.

62. Yang C.T., 1972, "Unit stream power and sediment Transport", Journal of the Hydraulics Division, ASCE, vol. 98, No. HY-10, Proc. paper 9295, October, pp. 1805-1926.

63. Yang, C.T. 1977, "The movement of sediment in rivers" Geophysical Surveys, Vol. 3, Part 1, P.39-68.

64. Yang. C.T., 1976, " Minimum Unit stream power and Fluvial Hydraulics", Journal of the Hydraulics Division, ASCE, vol. 102, No. HY-7, Proc. paper 12238, pp. 919-934.

65. Zernial, G.A., and Laurson, E.M., 1963, "Sediment Transporting Characteristics of streams", Journal of Hydraulics Divn. ASCE, vol. 89, No. HY-1, Proc. paper 3396, Jan. pp. 117-137.

APPENDIX- A

TABLE

Table - 2.1

Sediment Grade Scale

(Subcommittee on Sediment Technology, A.G.U.)

Size in millimeters	Microns	Inches	Class
4000 - 2000		160 -80	Very large boulders
2000 - 1000		80- 40	Large boulders
1000 - 500		40 - 20	Medium boulders
500 - 250		20 - 10	Small boulders
250 - 130		10 - 5	Large cobbles
132 - 64		5 - 2.5	Small cobbles
64 - 32		2.5 - 1.3	Very large gravel
32 - 16		1.3 - 0.6	Coarse gravel
16 - 8		0.6 - 0.3	Medium gravel
8 - 4		0.3- 0.16	Fine gravel
4 - 2		0.16-0.08	Very fine gravel
2 - 1	2000-1000		Very coarse sand
1 - 0.5	1000- 500		Coarse sand
0.5- 0.25	500- 250		Medium sand
0.25-0.125	250- 125		Fine sand
0.125-0.062	125 - 62		Very fine sand
0.062-0.031	62- 31		Coarse silt
0.031-0.016	31 - 16		Medium silt
0.016-0.008	16 - 8		Fine silt
0.008-0.004	8 - 4		Very fine silt
0.004-0.002	4 - 2		Coarse clay
0.002-0.001	2 - 1		Medium clay
0.001-0.0005	1 -0.5		Fine clay
0.0005-0.00024	0.5-0.24		Very fine clay

Table - 4.1

A brief summary of the river Gorai-Modhumoti

Station: Gorai Railway bridge Year: 1981

Length of the river (miles)	Maximum discharge (cfs)	Minimum discharge (cfs)	Maximum average depth (ft)	Minimum average depth (ft)	Maximum average width (ft)	Minimum average width (ft)	Maximum average velocity (ft/sec)	Minimum average velocity (ft/sec)
1	2	3	4	5	6	7	8	9
148	2,34,080	157	30.71	4.30	1549	203	5.41	0.58

TABLE - 5.1

BASIC DATA

RIVER: GORAI-MODHUMOTI Station: Gorai-Railway Bridge

Date	Width (ft)	Depth (ft)	Average velocity (ft/sec)	Cross-Sec.Area (ft^2)	Discharge (ft^3/sec)	Slope in 10^{-5}	Sediment concentration (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	
7-6-76	503	3.28	2.64	2030	5370	4.46	-	
14-6-76	1153	5.87	3.93	6100	24000	5.00	-	
21-6-76	1243	9.78	3.93	9300	36600	4.53	-	
28-6-76	1198	7.45	3.25	8400	27400	4.82	198	
5-7-76	1332	13.58	3.95	16500	65200	5.17	152	
13-7-76	1376	15.29	3.61	19100	69000	5.03	138	
19-7-76	1399	15.72	4.29	21200	91300	5.12	70	
26-7-76	1400	16.18	4.00	22500	89931	4.94	94	
2-8-76	1448	16.44	3.74	24300	90700	4.90	134	
9-8-76	1416	21.79	4.20	28900	121700	5.08	280	D_{50} = 0.048 D_{35} = 0.033 D_{65} = 0.051
16-8-76	1536	25.07	4.73	32400	153000	5.46	312	
23-8-76	1542	26.18	5.34	33900	181000	5.82	-	
30-8-76	1753	27.30	5.28	38400	202100	5.96	-	
6-9-76	1494	25.20	4.98	31900	159000	5.53	-	
13-9-76	1490	27.00	3.86	34200	131900	5.53	108	
20-9-76	1598	30.38	5.31	40800	216900	6.13	-	
24-9-76	1655	31.89	5.74	42700	245500	6.48	-	
28-9-76	1493	28.48	4.19	37400	158900	5.90	-	

Table-5.1contd.

Date	Width (ft)	Depth (ft)	Average velocity (ft/sec)	Cross-Sec.Area (ft^2)	Discharge (ft^3/sec)	Slope in 10^{-5}	Sediment concentrtion(ppm)	Bed-material size (mm)
1	2	3	4	5	6	7	8	
4-10-76	1424	23.23	3.21	27800	89300	5.26	-	
11-10-76	1414	20.44	2.45	24500	60200	4.91	-	
18-10-76	1447	17.52	2.25	18600	45800	4.64	-	
25-10-76	1169	14.86	2.37	14600	34500	4.40	-	
1-11-76	1097	13.42	2.72	10500	28700	4.38	-	
16-10-78	1394	25.95	2.73	30200	82200	4.79	-	
23-10-78	1383	23.13	2.44	24694	60200	4.47	-	
30-10-78	1276	20.67	2.35	19100	45000	4.47	-	
6-11-78	1187	18.87	2.26	14810	36930	4.12	-	
14-11-78	1148	16.93	2.46	12410	30500	3.98	-	
27-11-78	973	14.11	2.54	8444	21400	3.67	-	D_{50} = 0.048
4-12-78	911	12.60	2.66	6630	17700	3.42	-	D_{35} = 0.033
11-12-78	898	12.14	2.54	6598	16753	3.42	-	D_{65} = 0.051
25-12-78	903	10.96	2.99	4603	13800	3.23	-	
1-1-79	893	10.40	1.80	3593	6470	3.16	-	
8-1-79	893	9.65	1.44	3878	5590	3.18	-	
15-1-79	880	8.92	2.06	3970	8190	3.19	-	
22-1-79	878	8.96	2.17	4030	8730	3.36	-	
29-1-79	902	8.50	2.09	3740	7830	3.27	-	
5-2-79	916	8.89	2.12	4040	8550	3.39	-	
12-2-79	931	9.29	2.19	4460	9760	3.36	-	
19-2-79	968	9.74	2.18	5050	11000	3.45	-	

Table -5.1contd.

Date	Width (ft)	Depth (ft)	Average velocity (ft/sec)	Cross-Sec. Area (ft^2)	Discharge (ft^3/sec)	Slope in 10^{-5}	Sediment concentration (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	
26-2-79	951	9.22	2.26	4375	9880	3.46	-	
5-3-79	951	8.40	2.13	3780	8070	3.32	-	
12-3-79	973	8.99	2.10	4550	9550	3.50	-	
19-3-79	972	8.33	1.99	3850	7680	3.23	-	
26-3-79	891	7.71	2.06	3155	6500	3.20	-	
2-4-79	898	7.48	2.19	3040	6660	3.12	-	
9-4-79	906	7.35	2.17	2970	6440	3.320	106	
17-4-79	914	6.92	2.10	3495	7350	3.20	-	
23-4-79	922	8.04	2.24	3530	7890	3.29	104	
2-5-79	1013	7.51	2.16	3735	8060	3.23	148	
7-5-79	877	7.90	2.10	3860	8090	3.34	92	$D_{50} = 0.048$ $D_{35} = 0.033$ $D_{65} = 0.051$
14-5-79	1010	7.74	2.12	4035	8560	3.27	70	
22-5-79	1024	6.23	2.20	4630	10200	3.31	-	
4-6-79	1038	8.01	2.08	4760	9910	3.34	-	
11-6-79	1064	6.20	2.12	3460	7350	3.41	-	
18-6-79	1094	6.89	1.95	3910	7630	3.20	-	
25-6-79	1122	7.45	1.85	6045	11200	3.53	182	
2-7-79	1142	8.86	2.23	9090	20200	3.98	98	
9-7-79	1337	15.32	3.13	18000	56400	4.66	198	
16-7-79	1284	18.28	2.91	18365	53400	4.56	144	
23-7-79	1432	21.26	3.90	26140	102000	5.26	210	
30-7-79	1486	25.95	4.21	33580	141000	5.45	312	

Table-5.1 contd.

Date	Width (ft)	Depth (ft)	Average velocity (ft/sec)	Cross-Sec Area (ft^2)	Discharge (ft^3/sec)	Slope in 10^{-5}	Sediment concentration (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	
6-8-79	1482	27.82	3.88	36805	143000	3.92	192	
13-8-79	1433	27.66	3.47	31240	108000	4.93	-	
20-8-79	1480	27.53	3.68	36235	133000	3.26	398	
27-8-79	1483	32.61	3.56	41050	146000	5.24	396	
3-9-79	1421	28.41	3.19	32910	105000	4.92	246	
10-9-79	1421	27.27	3.32	32085	107000	4.93	120	
17-9-79	1420	23.98	2.89	29500	85200	4.72	84	
24-9-79	1371	19.23	2.26	21450	48600	4.26	146	
2-10-79	1334	17.68	2.35	18960	44500	4.30	94	
8-10-79	1394	19.59	2.52	21445	54100	4.43	120	
15-10-79	1387	19.88	2.44	20870	50900	4.36	120	$D_{50} = 0.048$ $D_{35} = 0.033$ $D_{65} = 0.051$
22-10-79	1187	16.18	2.19	16230	35600	3.88	120	
29-10-79	1060	13.55	2.17	12520	27200	3.64	106	
5-11-79	1026	8.79	2.37	8060	19100	3.38	114	
12-11-79	994	6.66	2.75	5500	15200	3.16	88	
15-11-79	964	6.23	2.96	4830	14300	3.23	-	
19-11-79	938	5.84	2.55	4690	12700	3.15	116	
26-11-79	801	6.37	3.00	3515	10540	3.09	96	
3-12-79	798	4.72	2.70	3555	9600	3.15	134	
10-12-79	809	5.54	2.60	4125	10700	3.16	120	
17-12-79	791	5.12	2.44	3810	9280	3.21	102	
24-12-79	805	8.04	2.17	4020	8710	3.14	124	
31-12-79	789	5.94	2.37	3170	7510	3.24	64	

Table-5.1 contd.

Date	Width (ft)	Depth (ft)	Average velocity (ft/sec)	Cross-Sec. Area ft^2	Discharge (ft^3/sec)	Slope in 10^{-5}	Sediment concentration (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	9
7-1-80	819	5.77	2.57	2705	6970	3.27	162	
14-1-80	786	5.64	2.10	2672	5619	3.23	166	
21-1-80	730	5.68	2.00	2375	4760	3.12	96	
27-1-80	664	5.09	1.97	1950	3840	3.12	218	
1-2-80	601	4.99	2.01	1930	3880	3.15	238	
6-4-80	408	4.00	1.86	1310	2430	3.01	20	
13-4-80	410	4.27	1.84	1380	2530	3.03	14	
20-4-80	403	4.56	1.84	1480	2710	2.97	20	
27-4-80	434	4.72	1.95	1460	2850	3.04	18	
4-5-80	819	6.50	2.17	3140	6810	3.46	46	D_{50} = 0.048 D_{35} = 0.033 D_{65} = 0.051
11-5-80	856	6.76	2.18	3620	7910	3.24	122	
18-5-80	873	7.02	2.44	3680	8970	3.22	104	
25-5-80	913	7.55	2.42	4310	10400	3.35	126	
2-6-80	963	8.76	2.16	4640	10000	3.32	120	
9-6-80	1008	9.97	2.13	5380	11500	3.32	148	
16-6-80	1120	11.19	3.09	8460	26100	4.30	172	
30-6-80	1214	12.40	2.81	14200	40100	4.35	30	
7-7-80	1380	15.75	3.20	17600	56300	4.78	126	
14-7-80	1412	21.52	4.16	26700	111000	4.90	146	
22-7-80	1518	24.31	5.60	33400	187000	5.47	476	
28-7-80	1524	26.40	4.70	36500	171800	5.58	550	
4-8-80	1488	29.96	4.00	41900	167700	5.53	330	

Table-5.1 contd.

Date	Width (ft)	Depth (ft)	Average velocity (ft/sec)	Cross-Sec. Area ft^2	Discharge (ft^3/sec)	Slope in 10^{-5}	Sediment concentration (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	9
19-8-80	1625	30.94	5.46	43000	235000	6.28	-	
1-9-80	1608	31.92	4.95	39600	196200	5.87	408	
8-9-80	1625	29.33	5.41	48600	262800	6.28	-	
22-9-80	1458	26.74	3.71	33900	126000	5.15	282	
29-9-80	1430	25.43	3.63	33300	120800	5.18	108	
6-10-80	1407	22.28	3.15	27000	85000	4.74	130	
13-10-80	1406	18.63	2.58	21500	55500	4.35	134	
28-10-80	1339	11.71	2.60	13200	34300	4.13	114	
4-11-80	1245	11.35	2.45	11900	29100	4.05	114	
10-11-80	1144	9.12	2.49	8960	22300	3.71	72	D_{50} = 0.048, D_{35} = 0.033, D_{65} = 0.051
17-11-80	1099	10.43	2.36	8740	20700	3.49	-	
25-11-80	893	9.09	2.24	7150	16000	3.44	-	
1-12-80	895	7.81	2.21	5880	13000	3.33	28	
8-12-80	902	6.17	2.21	4860	10700	3.24	56	
15-12-80	876	5.61	2.20	4310	9470	3.18	54	
22-12-80	879	4.63	2.04	3350	6830	3.03	38	
29-12-80	826	5.54	2.11	2780	5860	3.10	46	
4-1-81	839	5.12	1.94	2090	4050	3.01	70	
14-1-81	591	4.43	2.02	1410	2850	2.90	50	
18-1-81	410	4.82	1.85	1220	2260	2.85	24	
25-1-81	300	4.39	1.85	1010	1860	2.85	18	
3-2-81	343	4.00	1.68	1000	1690	2.91	-	

Table-5.1 contd.

Date	Width (ft)	Depth (ft)	Average velocity (ft/sec)	Cross-Sec. Area (ft^2)	Discharge (ft^3/sec)	Slope in 10^{-5}	Sediment concentration (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	9
8-2-81	303	4.37	1.73	1060	1840	3.05	-	
15-2-81	289	3.67	1.56	837	1310	2.86	-	
22-2-81	256	3.08	1.49	670	1000	2.81	-	
1-3-81	233	2.30	1.00	500	500	2.72	-	
8-3-81	223	2.00	0.97	400	385	2.57	-	
15-3-81	223	1.80	0.82	375	306	2.64	-	$D_{50} = 0.048$ $D_{35} = 0.033$ $D_{65} = 0.051$
22-3-81	217	1.71	0.78	363	281	2.68	-	
7-4-81	210	1.48	0.73	304	222	2.22	-	
12-4-81	203	1.31	0.58	273	157	2.47	-	
21-4-81	278	3.28	1.51	784	1189	2.39	-	
26-4-81	305	4.10	1.88	750	1409	2.77	-	

Table-5.1 contd.

Date	Width (m)	Max. depth (m)	Average velocity (m/sec)	X-Sec. at area (m^2)	Discharge m^3/sec.	Slope in 10^{-5}	Sediment conc. (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	9
10-5-81	111	1.42	0.600	98	59	2.94	-	
18-5-81	124	1.60	0.628	111	69	3.15	-	
24-5-81	226	1.90	0.640	221	142	3.25	-	
8-6-81	297	2.80	0.698	375	262	3.12	-	
15-6-81	290	2.54	0663	313	208	3.30	-	
22-6-81	304	2.98	0.773	428	331	3.59	-	
29-6-81	322	3.26	0.779	493	384	3.73	-	
6-7-81	422	5.09	1.223	1820	2220	5.23	-	
13-7-81	435	6.06	1.277	2240	2860	5.06	376	D_{50} = 0.048 D_{35} = 0.033 D_{65} = 0.051
20-7-81	441	6.32	1.435	2490	3570	5.29	230	
28-7-81	458	7.47	1.622	3110	5050	5.43	272	
5-8-81	472	8.35	1.640	3790	6210	5.64	192	
11-8-81	472	7.75	1.540	3370	5190	5.52	230	
17-8-81	469	7.38	1.485	3370	5020	5.35	232	
24-8-81	463	7.52	1.270	3220	4090	5.29	306	
31-8-81	469	9.36	1.650	4040	6650	5.73	296	
8-9-81	458	8.05	1.330	3790	5030	5.08	-	
14-9-81	444	8.62	1.350	3730	5050	5.67	-	
22-9-81	422	7.96	1.185	3420	4060	4.97	-	
28-9-81	450	7.79	0.900	2900	2610	4.85	-	
4-10-81	449	7.63	0.925	3050	2820	5.20	164	
20-10-81	441	5.67	0.650	2190	1430	4.30	-	
27-10-81	359	3.53	0.675	1300	885	3.88	-	
2-11-81	340	2.89	0.590	1045	617	3.73	-	

Table-5.1 contd.

Date	Width (m)	Max. depth (m)	Average velocity (m/sec)	X-Sec. at area (m^2)	Discharge m^3/sec	Slope in 10^{-5}	Sediment conc (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	9
10-11-81	287	2.33	0.650	670	435	3.62	-	
17-11-81	282	2.16	0.730	550	401	3.55	284	
30-11-81	259	1.66	0.930	292	272	3.30	194	
4-4-83	38	0.65	0.3241	13.67	4.43	3.43	-	
11-4-83	33	0.68	0.3461	12.40	4.29	3.49	-	
18-4-83	34	0.68	0.3482	14.49	5.05	3.48	-	
25-4-83	59	0.90	0.3352	28.34	9.50	3.87	-	
2-5-83	82	1.10	0.4872	34.24	18.63	3.35	-	
9-5-83	108	1.60	0.4167	77.30	32.21	3.77	-	
16-5-83	152	1.80	0.5516	143.69	79.26	3.95	-	D_{50} = 0.048 D_{35} = 0.033 D_{65} = 0.051
23-5-83	202.54	1.70	0.5742	163.31	93.78	3.99	-	
30-5-83	265.00	1.89	0.6490	243.30	157.89	4.08	-	
6-6-83	206.52	1.70	0.6672	204.60	136.50	3.77	-	
13-6-83	304.96	2.08	0.7633	250.83	191.46	3.98	-	
20-6-83	248.66	1.81	0.6225	236.31	147.10	3.71	-	
27-6-83	399.54	2.65	0.7206	542.72	391.11	3.91	-	
4-7-83	414.03	2.64	0.9167	689.11	631.60	4.07	-	
11-7-83	425.36	5.49	1.3620	1891.05	2515.52	5.14	-	
18-7-83	421.83	5.18	1.2367	1688.73	2088.38	4.61	-	
25-7-83	426.82	7.01	0.9841	2150.19	2116.00	4.77	-	
1-8-83	441.34	9.27	1.3281	2580.15	3428.15	5.24	-	
8-8-83	442.09	9.47	1.4090	2957.00	4166.27	5.49	-	
17-8-83	438.03	10.66	0.8461	2983.24	2524.15	5.05	-	

Table-5.1 contd.

Date	Width (m)	depth (m)	Average velocity (m/sec)	X-sec. area (m^2)	Discharge (m^3/sec)	Slope in 10^{-5}	Sediment conc. (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	9
24-8-83	439.65	10.45	1.0847	3216.65	3489.19	5.11	-	
31-8-83	451.83	11.77	1.4410	3634.85	5242.10	5.65	-	
7-9-83	442.23	11.35	1.2809	3500.12	4401.71	5.53	-	
14-9-83	478.50	13.22	1.6311	3124.29	6495.82	6.21	-	
21-9-83	506.46	12.25	1.9324	3864.62	7468.05	6.76	-	
29-9-83	458.57	13.01	1.3830	3552.26	4912.86	6.01	-	
5-10-83	430.92	12.80	1.2141	3349.34	4066.30	5.67	-	
12-10-83	441.48	11.34	0.9997	3143.64	3142.77	4.84	-	
19-10-83	403.53	11.80	1.0127	2718.54	2753.12	5.16	-	
26-10-83	427.98	10.22	0.9423	2335.58	2200.53	4.62	-	
2-11-83	428.88	9.96	0.7397	1944.04	1438.02	4.64	-	D_{50} = 0.048, D_{35} = 0.033, D_{65} = 0.051
9-11-83	330.81	7.34	0.7218	1448.00	1045.23	4.38	-	
16-11-83	303.62	5.97	0.9048	957.35	866.18	4.07	-	
23-11-83	303.70	5.09	1.0454	645.70	675.03	3.90	-	
30-11-83	308.95	4.28	0.9369	528.44	495.10	3.76	-	
7-12-83	330.64	2.60	1.1304	421.94	476.97	3.63	-	
14-12-83	345.40	2.66	1.0063	466.81	469.75	3.65	-	
21-12-83	310.38	3.13	0.8146	324.23	264.13	3.53	-	
29-12-83	317.44	2.15	0.7670	291.10	223.28	3.52	-	
4-1-84	293.23	2.17	0.7990	301.86	241.18	3.49	-	
11-1-84	302.32	2.06	0.7298	284.15	207.37	3.59	-	
18-1-84	245.03	2.13	0.7231	231.29	167.25	3.68	-	
25-1-84	185.23	2.72	0.6777	242.62	164.13	3.77	-	

Table-5.1 contd.

Date	Width (m)	Depth (m)	Average velocity (m/sec)	X-sec. area (m^2)	Discharge (m^3/sec)	Slope in 10^{-5}	Sediment conc. (ppm)	Bed material size (mm)
1	2	3	4	5	6	7	8	9
1-2-84	234.56	2.56	0.5949	271.83	161.71	3.79	-	
8-2-84	183.13	3.00	0.6204	256.34	159.09	3.74	-	
15-2-84	145.57	2.40	0.6249	203.14	126.95	3.80	-	
22-2-84	159.06	2.90	0.4949	233.24	115.43	3.75	-	
29-2-84	158.10	2.80	0.4768	210.91	100.56	3.75	-	
7-3-84	153.55	2.69	0.6210	138.35	85.92	3.69	-	
14-3-84	153.23	2.65	0.3755	187.80	70.51	3.63	-	D_{50} = 0.048 D_{35} = 0.033 D_{65} = 0.051
21-3-84	151.33	2.35	0.4304	121.46	52.28	3.51	-	
28-3-84	151.61	2.00	0.4675	105.61	49.37	3.58	-	

Table: 5.2

Descripancy Ratio (D.R) of different formulae

Sl. No.	Formula	Percentage of data coverage between descripancy ratio of						Mean of D.R.	Standard deviation of D.R.
		1/8 to 1/4	1/4 to 1/2	1/2 to 1	1 to 2	2 to 4	4 to 8		
	1	2	3	4	5	6	7	8	9
1	STRATHCLYDE (1984)	0	0	10.75	49.46	27.95	4.30	2.77	2.99
2	ENGELUND-HANSEN (1967)	4.30	21.50	23.65	32.26	16.13	2.15	1.26	0.92
3	YANG (1976)	0	19.35	22.58	9.67	1.07	0	0.66	0.45
4	MANTZ (1983)	0	6.45	38.70	38.70	10.75	5.37	1.42	1.07
5	ACKERS-WHITE (1973)	2.15	0	1.07	1.07	3.22	2.15	2.44	-

$$\text{Descripancy Ratio (D.R)} = \frac{\text{Calculated total sediment transport rate in ppm}}{\text{Observed suspended sediment transport rate in ppm}}$$

TABLE - 5.3

Evalution of Total Load

River: Gorai-Modhumoti Station: Gorai Rly. Bridge

Date	Discharge (cfs)	Observed sediment discharge (tons/ day)	Predicted total sediment discharge in Tons/day				
			Mantz formula	Strathclyde formula	Engelund- Hansen formula	Yang Formula	Ackers-White Formula
1	2	3	4	5	6	7	8
28-6-76	27,400	14,943	12,105	21,117	10,626	5716	32,212
5-7-76	65,200	33,353	39,208	62,093	45,449	23,364	18,73,077
13-7-76	69,000	16,439	36,920	56,566	44,542	20,154	29,39,035
19-7-76	91,300	15,782	59,609	95,689	74,848	37,648	83,05,595
26-7-76	89,931	25,741	52,976	85,866	67,545	30,682	80,40,432
2-8-76	90,700	38,762	49,644	78,586	64,338	27,160	84,46,381
9-8-76	1,21,700	91,068	78,262	1,20,082	1,12,316	48,391	-
16-8-76	1,53,000	1,12,000	1,20,547	1,62,608	1,81,009	84,496	-
13-9-76	1,31,900	36,260	85,793	1,11,369	1,35,327	52,771	-
2-4-79	6,660	1,800	1,128	2,125	637	-	-
17-4-79	7,350	1,000	1,244	2,453	739	-	-
23-4-79	7,890	2,010	1,471	2,672	890	-	-
2-5-79	8,060	2,060	1,404	2,473	830	-	-
7-5-79	8,090	2,300	1,469	2,859	893	-	39
14-5-79	8,560	1,470	1,512	2,479	903	-	21
25-6-79	11,200	5,900	1,924	2,639	1,347	329	220
2-7-79	20,200	4,300	5,007	8,974	4,313	1,437	8,675
9-7-79	56,400	28,100	24,087	37,377	27,686	11,074	11,63,235
16-7-79	53,400	21,000	20,708	30,801	24,379	8,913	11,54,536

Table-5.3 Contd.

Date	Discharge (cfs)	Observed sediment discharge (tons/day)	Predicted total sediment discharge in Tons/day				
			Mantz formula	Strathclyde formula	Engelund- Hansen Formula	Yang Formula	Ackers-White Formula
1	2	3	4	5	6	7	8
23-7-79	1,02,000	51,700	62,839	88,877	87,125	37,553	-
30-7-79	1,41,000	1,06,100	98,633	1,33,934	1,53,314	63,679	-
6-8-79	1,43,000	76,300	65,284	1,05,298	91,258	31,589	-
20-8-79	1,33,000	1,30,800	48,966	83,570	63,656	18,933	-
27-8-79	1,46,000	1,54,800	75,736	97,608	1,27,314	46,228	-
3-9-79	1,05,000	68,300	49,482	66,234	74,996	24,483	-
10-9-79	1,07,000	30,600	52,526	71,961	78,526	26,788	-
17-9-79	85,200	18,800	34,714	48,516	49,143	15,475	-
24-9-79	84,600	17,500	23,672	33,847	28,240	9,552	13,09,850
2-10-79	44,500	12,900	12,997	18,896	14,963	4,587	4,96,725
8-10-79	54,100	19,100	17,660	24,565	21,245	6,639	8,20,472
15-10-79	50,900	16,600	15,491	21,613	18,739	5,747	8,35,014
22-10-79	35,600	11,800	8,650	14,592	9,349	2,621	1,82,785
29-10-79	27,200	8,500	6,075	11,616	6,008	1,669	50,535
5-11-79	19,100	6,100	6,000	10,501	3,328	1,734	1,125
12-11-79	15,200	3,590	4,999	9,476	2,350	-	74
19-11-79	12,700	3,880	3,584	7,730	1,620	-	-
26-11-79	10,540	2,300	3,621	7,657	1,526	-	-
3-12-79	9,600	3,450	3,015	7,092	1,295	-	-
10-12-79	10,700	3,660	3,309	7,432	1,549	-	-
17-12-79	9,280	3,040	2,710	6,127	1,230	-	-
24-12-79	8,710	2,520	2,223	3,998	1,004	-	-

Table -5.3 Contd.

Date	Discharge (cfs)	Observed sediment discharge (tons/day)	Predicted total sediment discharge in Tons/day				
			Mantz formula	Strathclyde formula	Engelund Hansen formula	Yang formula	Ackers-White formula
1	2	3	4	5	6	7	8
31-12-79	7,510	1,320	2,101	4,110	884	-	-
7-1-80	6,970	2,430	2,070	3,815	821	-	-
14-1-80	5,619	1,590	1,351	2,496	538	-	-
21-1-80	6,760	1,133	1,476	2,854	564	-	-
27-1-80	3,840	1,890	810	1,687	301	-	-
1-2-80	3,880	1,750	857	1,981	333	-	-
6-4-80	2,430	492	578	1,354	179	-	-
13-4-80	2,530	48	583	1,372	192	-	-
20-4-80	2,710	97	366	1,496	213	-	-
27-4-80	2,850	101	413	1,539	231	-	-
4-5-80	6,810	680	1,304	2,724	802	-	-
11-5-80	7,910	2,381	1,436	3,087	873	-	- 39
18-5-80	8,970	1,870	1,805	3,875	1,100	-	44
25-5-80	10,400	2,636	2,220	4,518	1,429	255	102
2-6-80	10,000	2,850	1,890	3,412	1,227	196	98
9-6-80	11,500	3,186	2,145	3,726	1,468	339	197
16-6-80	26,100	11,160	9,801	16,015	8,456	4,036	22,614
30-6-80	40,100	2,789	14,075	24,704	14,763	5,511	2,62,402
7-7-80	56,300	17,500	25,150	36,482	28,605	11,884	10,57,140
14-7-80	1,11,000	38,600	68,112	1,03,258	92,905	40,867	-
22-7-80	1,87,000	2,36,800	1,75,793	2,57,035	2,68,051	1,38,615	-
28-7-80	1,71,800	2,46,300	1,37,890	1,89,335	2,21,805	97,830	

Table 5.3 Contd.

Date	Discharge (cfs)	Observed sediment discharge (Ton/day)	Predicted total sediment discharge (Tons/day)				
			Mantz formula	Strathclyde formula	Engelund Hansen formula	Yang formula	Ackers-White formula
1	2	3	4	5	6	7	8
4-8-80	1,67,700	1,47,100	1,13,607	1,54,769	1,97,165	71,210	-
1-9-80	1,96,200	2,03,100	1,75,292	2,10,447	2,91,833	1,33,395	-
22-9-80	1,26,800	95,400	72,677	1,00,511	1,11,954	41,441	-
29-9-80	1,20,800	36,220	68,788	97,549	1,06,741	38,545	-
6-10-80	85,000	30,400	37,762	54,453	51,532	17,942	-
13-10-80	55,500	20,200	18,118	26,155	21,660	6,811	10,68,268
28-10-80	34,300	9,600	10,439	16,585	10,018	3,704	67,435
4-11-80	29,100	8,300	8,214	15,642	7,714	4,500	18,642
10-11-80	22,300	4,080	8,155	12,425	4,707	2,791	2,572
1-12-80	13,000	910	3,701	6,860	1,914	606	191
8-12-80	10,700	1,122	2,889	5,594	1,392	-	26
15-12-80	9,470	1,256	2,464	4,974	1,115	-	-
22-12-80	6,830	626	1,508	3,051	603	-	-
29-12-80	5,840	667	735	2,488	532	-	-
4-1-81	4,050	706	775	1,352	278	-	-
14-1-81	2,850	356	538	1,294	181	-	-
18-1-81	2,260	133	399	1,237	149	-	-
25-1-81	1,860	70	337	1,365	127	-	-
13-7-81	1,00,662	95,470	63,998	93,650	84,013	39,041	-
20-7-81	1,25,664	75,500	94,692	1,40,341	1,32,321	65,081	-
28-7-81	1,77,760	1,17,800	1,57,072	2,29,500	2,40,407	1,19,549	-
5-8-81	2,18,592	1,15,300	2,04,955	2,92,411	3,44,990	1,60,959	-

Table 5.3 Contd.

Date	Discharge (cfs)	Observed sediment discharge Tons/day	Predicted total sediment discharge (Tons/day)				
			Mantz formula	Strathclyde formula	Engelund Hansen formula	Yang formula	Ackers-White formula
1	2	3	4	5	6	7	8
11-8-81	1,82,688	97,660	1,56,045	2,19,271	2,47,521	1,15,689	-
17-8-81	1,76,704	1,01,450	1,40,525	2,07,318	2,19,462	99,755	-
24-8-81	1,43,968	1,06,680	96,469	1,35,693	1,48,768	60,779	-
31-8-81	2,34,080	1,54,990	2,25,798	3,09,107	3,94,140	1,78,684	-
4-10-81	99,264	44,180	47,025	62,131	71,634	23,390	-
17-11-81	14,152	9,900	4,689	8,580	2,466	1,112	312
30-11-81	9,574	3,554	3,501	6,862	1,433	-	-

TABLE - 5.4

Comparation of observed and calculated sediment concentration

River: Gorai-Modhumoti Station: Gorai Railway Bridge

Date	Observed sediment concentration (ppm)	Total sediment conc. by STRATHCLYDE FORMULA (ppm)	Total sediment conc. by ENGELUND-HANSEN FORMULA (ppm)	Total sediment calculate by YANG'S FORMULA FORMULA (ppm)	Total sediment calculated by MANTZ'S FORMULA (ppm)	Total sediment calculated by ACKERS-WHITE FORMULA (ppm)
1	2	3	4	5	6	7
28-6-76	198	314	158	85	180	479
5-7-76	152	388	284	146	245	13759
13-7-76	138	334	263	119	218	17354
19-7-76	70	427	334	168	266	37063
26-7-76	94	389	306	139	240	36426
2-8-76	134	353	289	122	223	37941
9-8-76	280	402	376	162	262	—
16-8-76	312	433	482	225	321	—
13-9-76	108	344	418	163	265	—
2-4-79	104	130	39	*	69	—
9-4-79	106	128	39	*	70	—
17-4-79	56	136	41	*	69	-
23-4-79	104	138	46	*	76	-
2-5-79	148	125	42	*	71	-
7-5-79	92	144	45	*	74	02
14-5-79	70	118	43	*	72	01
25-6-79	182	96	49	12	70	08
2-7-79	98	181	87	29	101	175
9-7-79	198	270	200	80	174	8403
16-7-79	144	235	186	68	158	8809
23-7-79	210	355	348	150	251	79261

Table 5.4 contd.

Date	Observed sediment concentration (ppm)	Total sediment conc. by STRATHCLYDE FORMULA (ppm)	Total sediment conc. by ENGLU.—HANSEN FORMULA (ppm)	Total sediment calculated by YANG's FORMULA (ppm)	Total sediment calculated by MANTZ FORMULA (ppm)	Total sediment calculated by ACKERS-WHITE FORMULA (ppm)
1	2	3	4	5	6	7
30-7-79	312	387	443	184	285	—
6-8-79	192	300	260	90	186	82816
20-8-79	398	256	195	58	150	31947
27-8-79	396	299	390	129	232	*
3-9-79	246	257	291	95	192	—
10-9-79	120	274	299	102	200	—
17-9-79	84	232	235	74	166	52452
24-9-79	146	163	136	46	114	6308
2-10-79	94	173	137	42	119	4548
8-10-79	120	185	160	50	133	8396
15-10-79	120	173	150	46	124	6684
22-10-79	120	167	107	30	99	2092
29-10-79	106	174	90	25	91	757
5-11-79	114	224	71	37	128	24
12-11-79	88	254	63	*	134	02
19-11-79	116	248	52	*	115	–
26-11-79	96	296	59	*	140	–
3-12-79	134	301	55	*	128	–
10-12-79	120	283	59	*	126	–
17-12-79	102	269	54	*	119	–
24-12-79	124	187	47	*	104	–
31-12-79	64	223	48	*	114	–
7-1-80	162	223	48	*	121	–
14-1-80	166	181	39	*	98	–
21-1-80	96	172	34	*	89	–
27-1-80	218	179	32	*	86	–
1-2-80	238	208	35	*	90	–
6-4-80	20	227	30	*	97	–

Table5.4contd.

Date	Observed sediment concentration (ppm)	Total sediment calculated by STRATHCLYDE FORMULA (ppm)	Total sediment calculated by ENGELUND-HANSEN FORMLA (ppm)	Total sediment calculated by YANG'S FORMULA (ppm)	Total sediment calculated by MANTZ FORMULA (ppm)	Total sediment calculated by ACKERS-WHITE FORMULA (ppm)
1	2	3	4	5	6	7
13-4-80	14	221	31	*	94	-
20-4-80	20	225	32	*	55	-
27-4-80	18	220	33	*	59	-
4-5-80	46	163	48	*	78	-
11-5-80	122	159	45	*	74	02
18-5-80	104	176	50	*	82	02
25-5-80	126	177	56	10	87	04
2-6-80	120	139	50	08	77	04
9-6-80	148	132	52	12	76	07
16-6-80	172	250	132	63	153	353
30-6-80	30	251	150	56	143	2666
7-7-80	126	264	207	86	182	7650
14-7-80	146	379	341	150	250	-
22-7-80	476	560	584	302	383	-
28-7-80	550	449	526	232	327	-
4-8-80	330	376	479	173	276	-
1-9-80	408	437	606	277	364	-
22-9-80	282	325	362	134	235	-
29-9-80	108	329	360	130	232	-
6-10-80	130	261	247	86	181	43831
13-10-80	134	192	159	50	133	7842
28-10-80	114	197	119	44	124	801
4-11-80	114	219	108	63	115	261
10-11-80	72	227	86	51	149	47
1-12-80	28	215	60	19	116	06
8-12-80	56	213	53	*	110	01
15-12-80	54	214	48	*	106	-
22-12-80	38	182	36	*	90	-
29-12-80	46	173	37	*	94	-
4-1-81	70	136	28	*	78	-
14-1-81	50	185	26	*	77	-

Table 5.4 contd.

Date	observed sediment concentration (ppm)	Total sediment calculated by STRATHCLYDE FORMULA (ppm)	Total sediment calculated by ENGELUND-HANSEN (ppm)	Total sediment calculated by YANG'S FORMULA (ppm)	Total sediment calculated by MANTZ'S FORMULA (ppm)	Total sediment calculated by ANCKERS-WHITE FORMULA (ppm)
1	2	3	4	5	6	7
18-1-81	24	223	27	*	72	-
25-1-81	18	299	28	*	74	-
13-7-81	376	379	340	158	259	54131
20-7-81	230	455	429	211	307	-
28-7-81	272	526	551	274	360	-
5-8-81	192	545	643	300	382	-
11-8-81	230	489	552	258	348	-
17-8-81	232	478	506	230	324	-
24-8-81	306	384	421	172	273	-
31-8-81	296	538	686	311	393	*
4-10-81	164	255	294	96	193	-
17-11-81	284	247	71	32	135	09
30-11-81	194	292	61	*	149	-

(-) & * Indicates results did not found in the formula

TABLE 6.1

SEDIMENT FLOW RATES CALCULATED BY DIFFERENT FORMULAE

RIVER:- Gorai Station: Gorai Rly.Bridge Year:1980

Unit	Observed sediment flow rates	Predicted sediment flow rates				
		Mantz	Strathclyde	Enge.-Hansen	Yang	Ackers-White
1	2	3	4	5	6	7
Tons/day	36,261	30,629	42,895	47,279	19,310	4,68,852
Mill.Tons/Yr.	13.27	11.21	15.70	17.30	7.06	171.67

APPENDIX - B

FIGURE

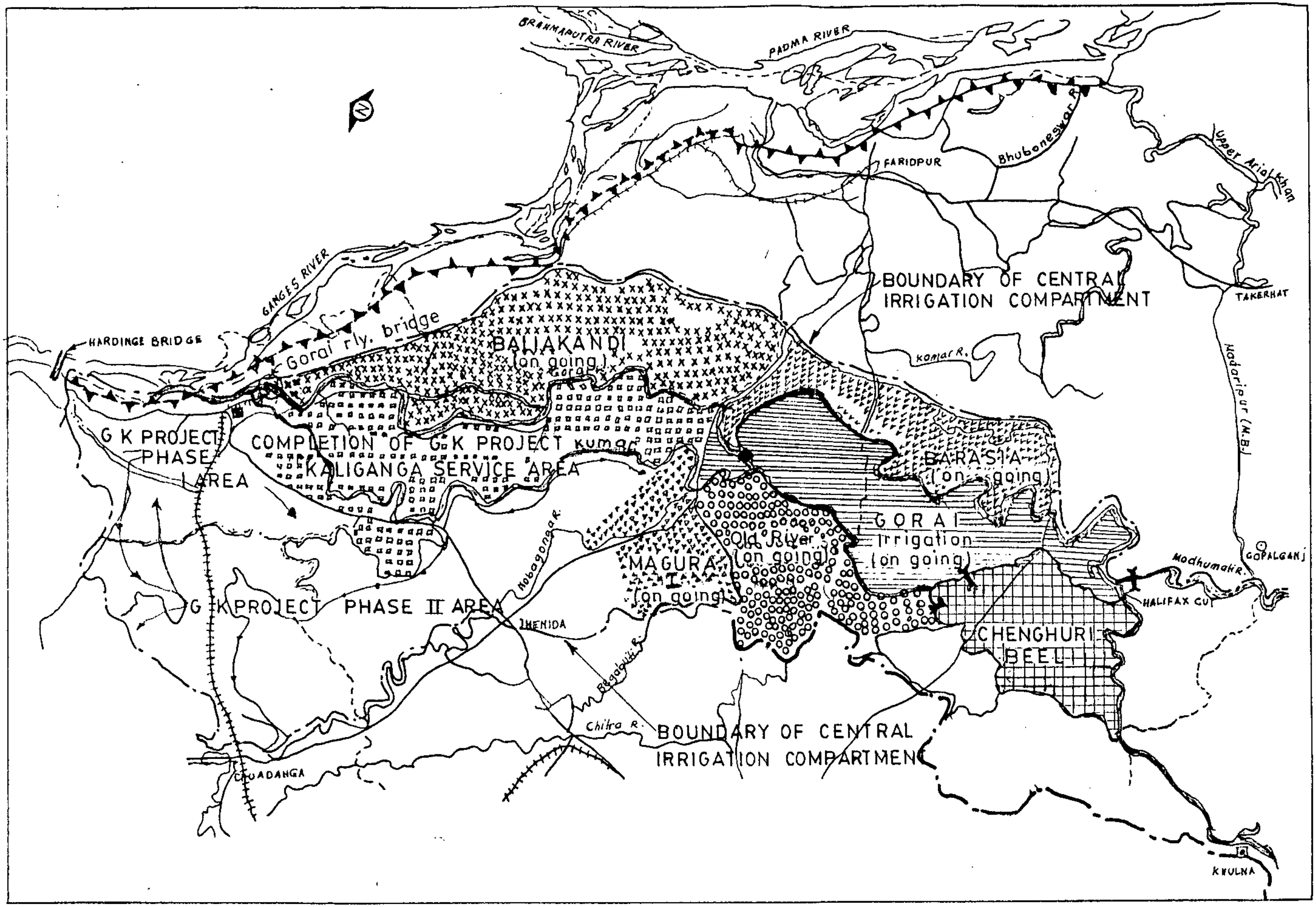


Fig. 1.1 Present and ongoing projects in the Gorai river valley. (after IECO, 1981)

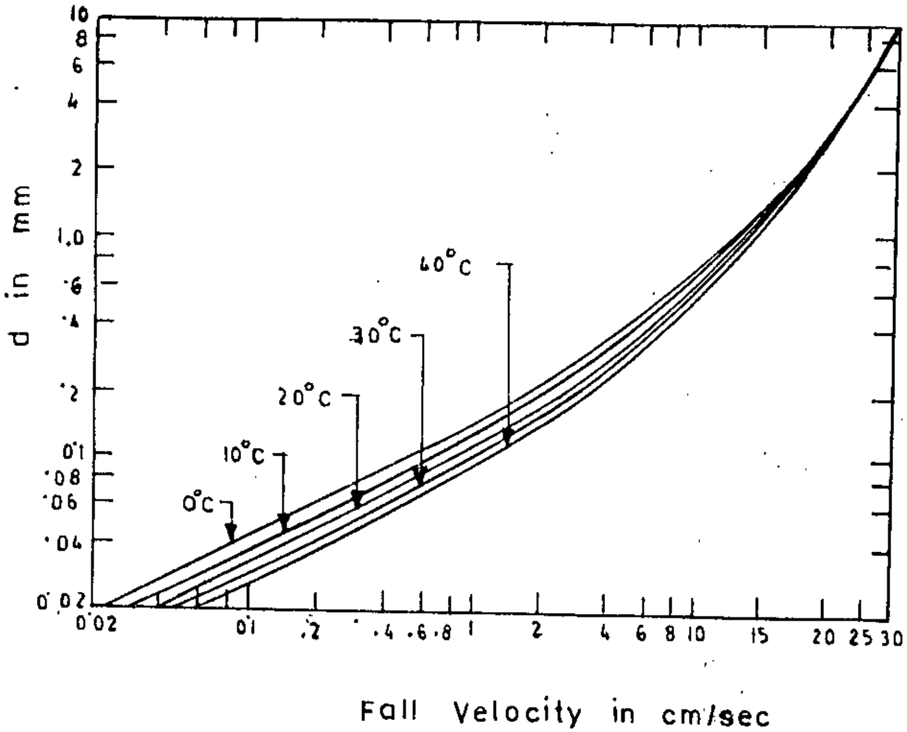


Fig. 2.1 Fall Velocity of Spherical Particles (Relative Density=2.65) in Water. (after Raudkivi 1967)

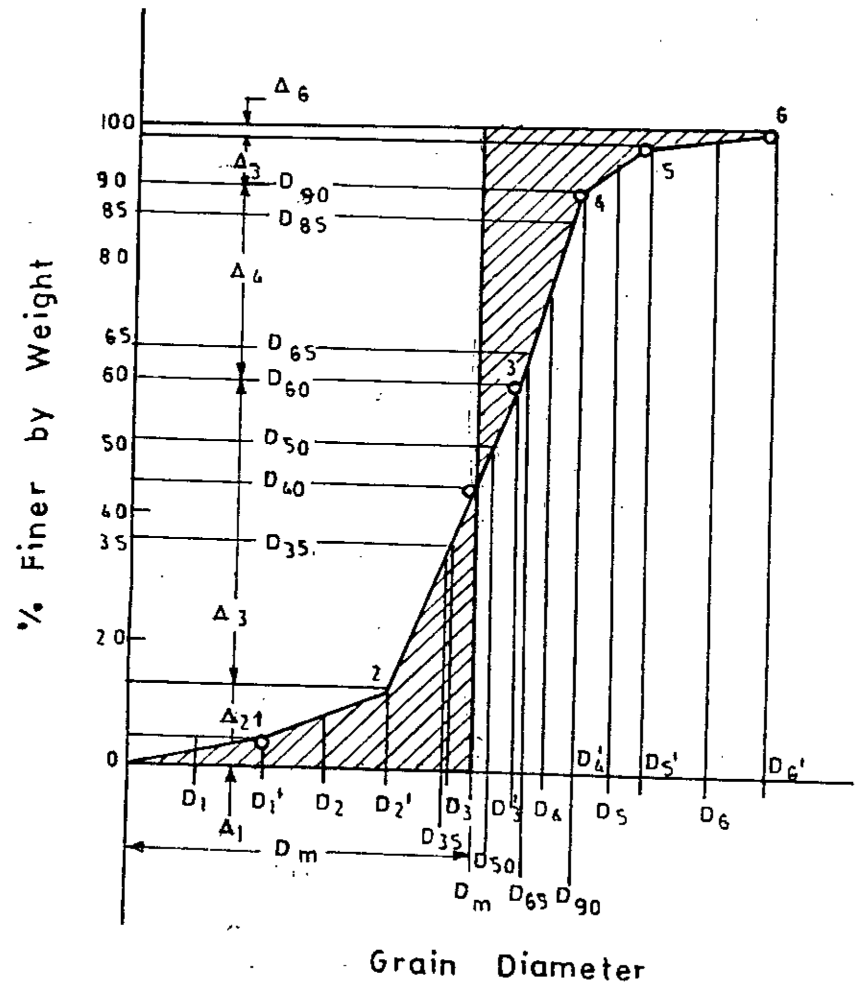


Fig. 2.2 Size frequency distribution showing D_m, D_{35}, D_{50}, D_{65}, D_{85} and D_{90} (after Simons)

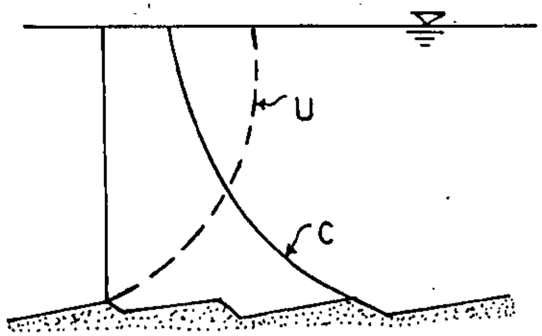


Fig. 2.3 Idealized velocity and concentration distribution (after Nordin and Demster 1963)

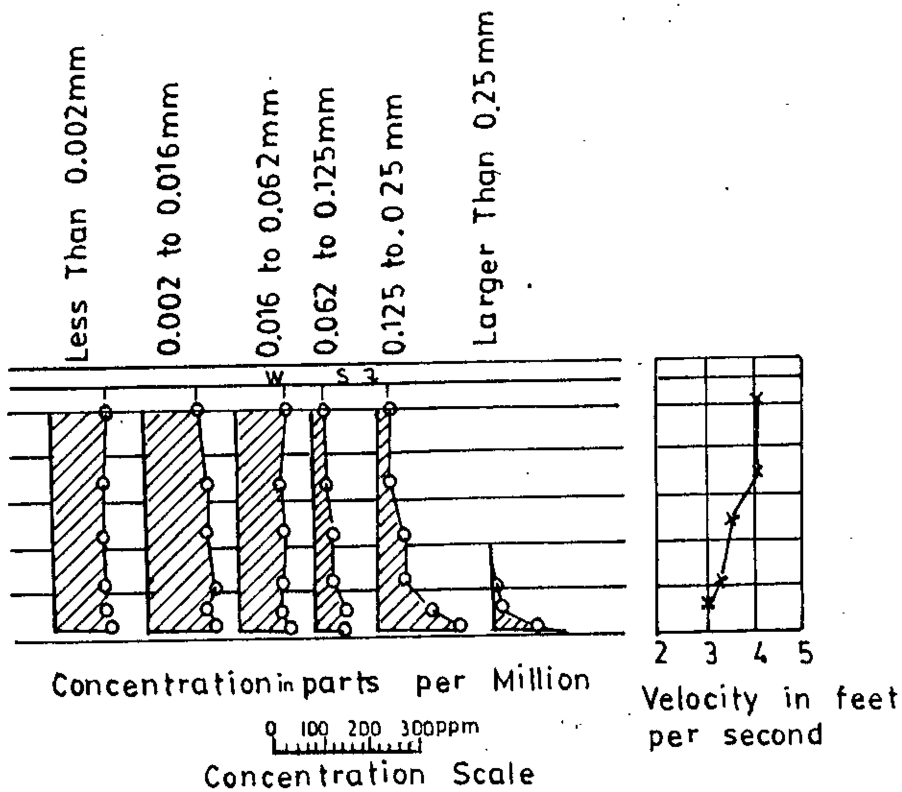


Fig. 2.4 Variation of concentration with particle size and depth (after colby, 1963)

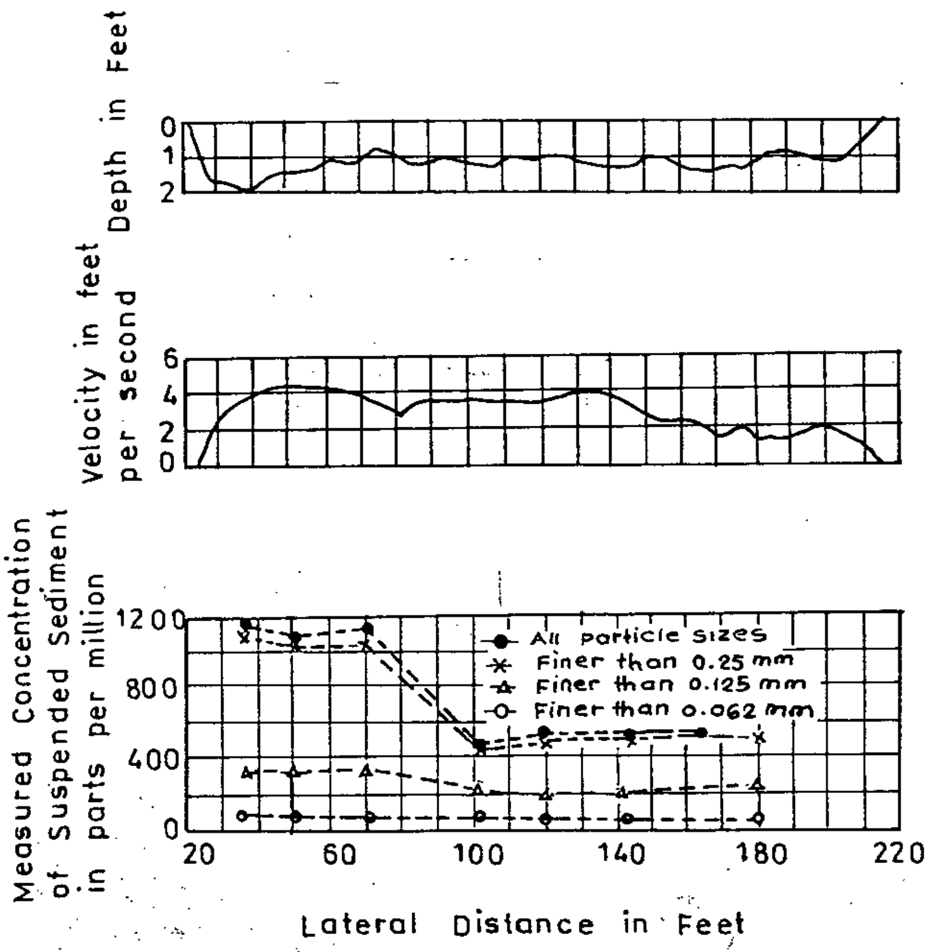


Fig. 2.5 Lateral Variation of Velocity and Concentration (after Colby, 1963)

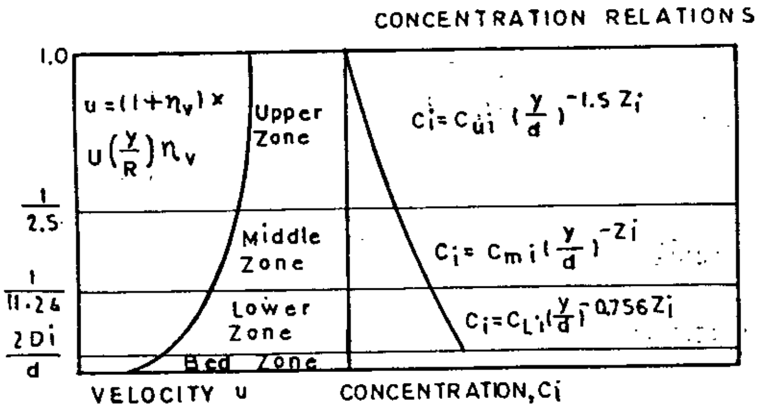


Fig. 2.6 Toffaleti's (1969) velocity, concentration, and sediment discharge relations.

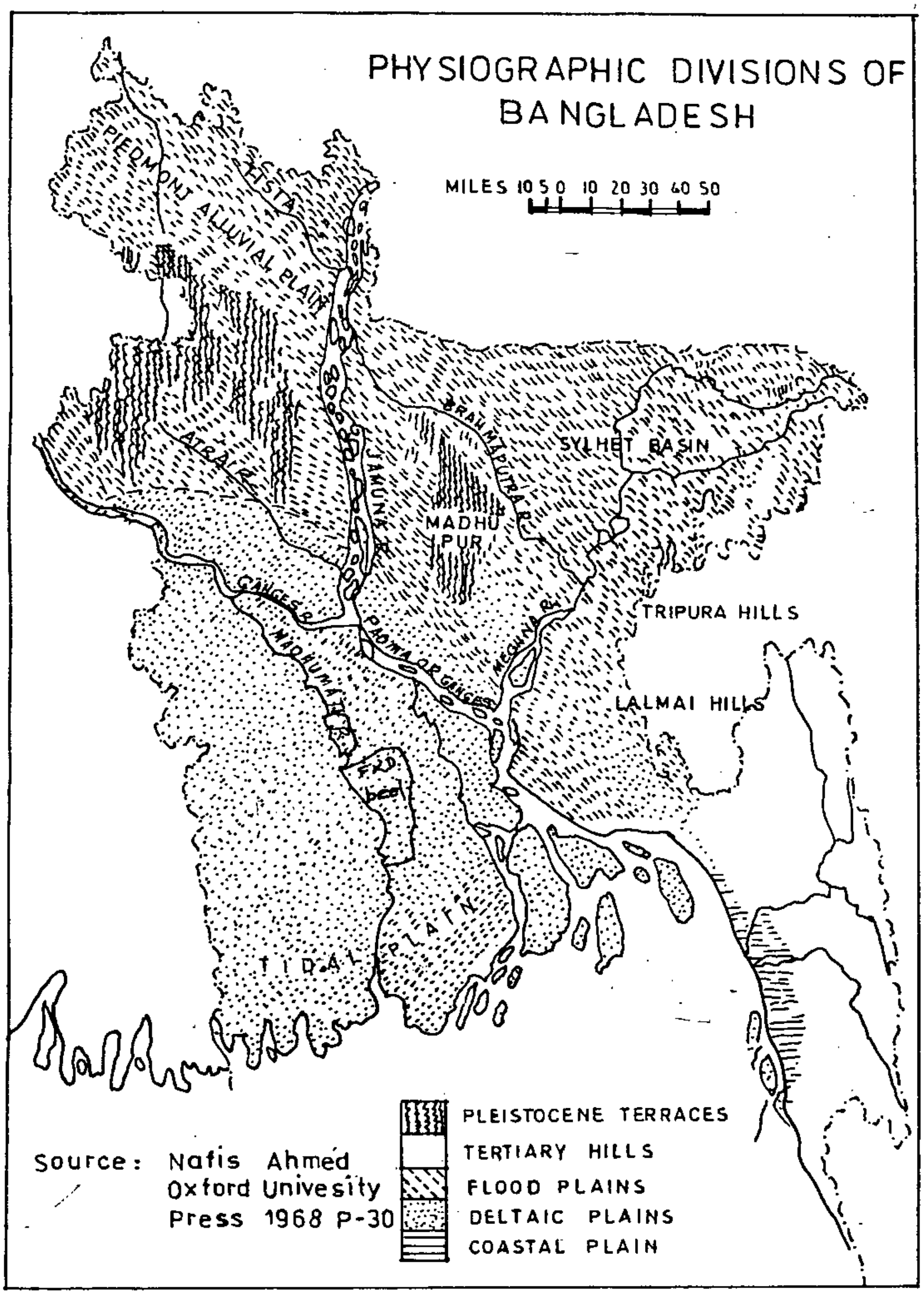


FIG. 4.1 PHYSIOGRAPHIC DIVISIONS OF BANGLADESH

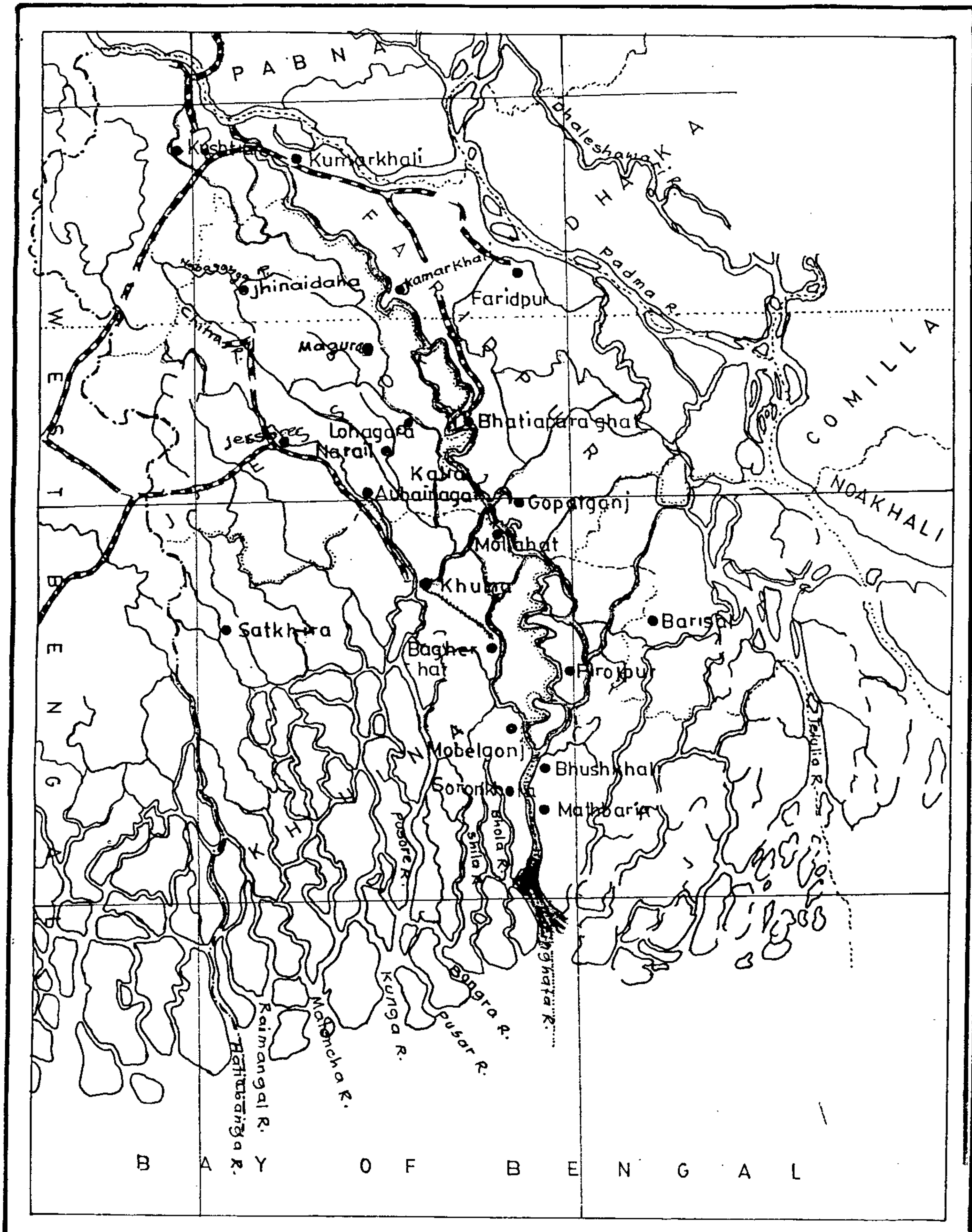


Fig. 4.2 The distributories of the river Gorai.

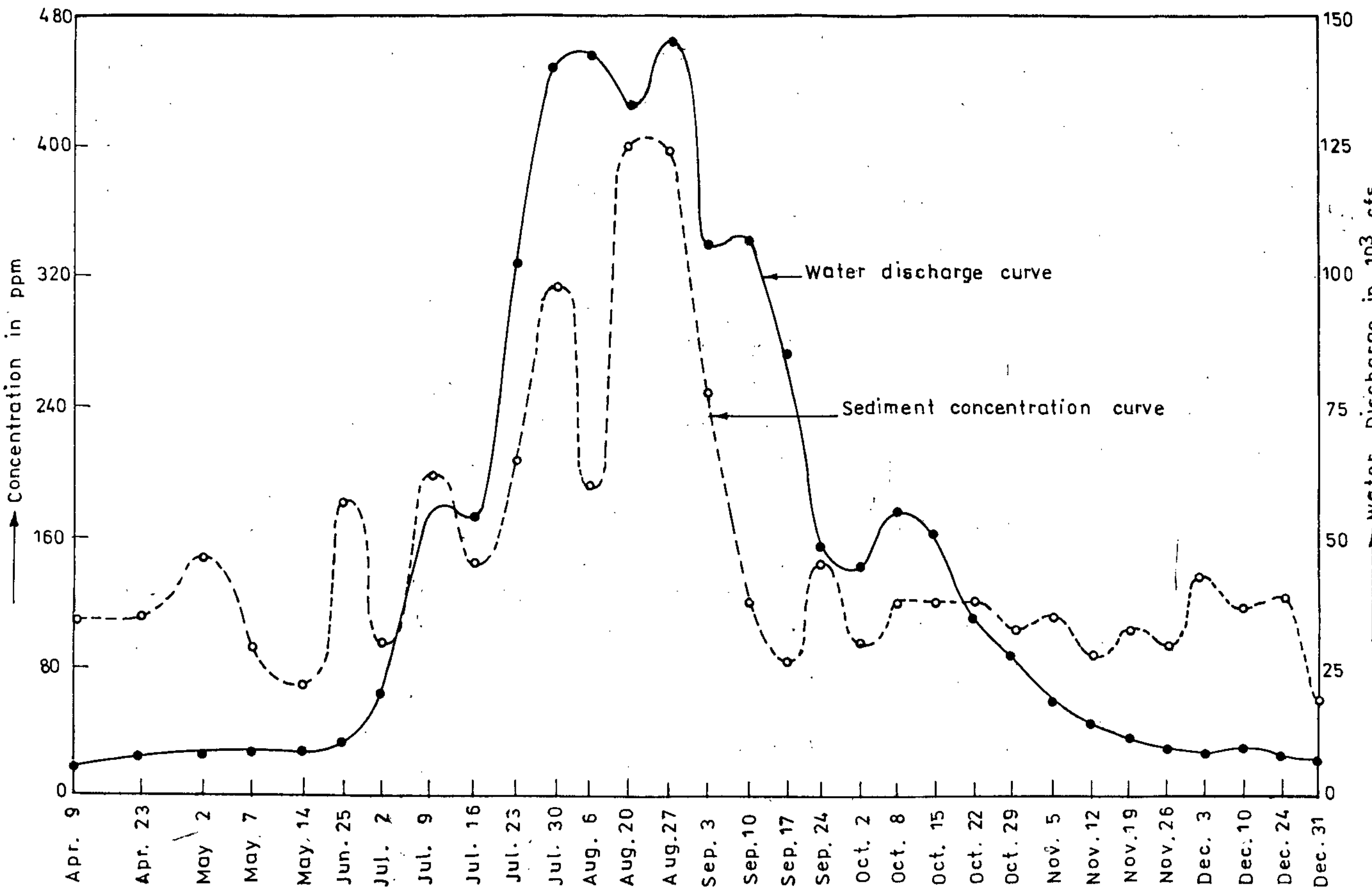


Fig. 4.13 Water discharge and sediment concentration of Gorai at Gorai rly. bridge station, 1979

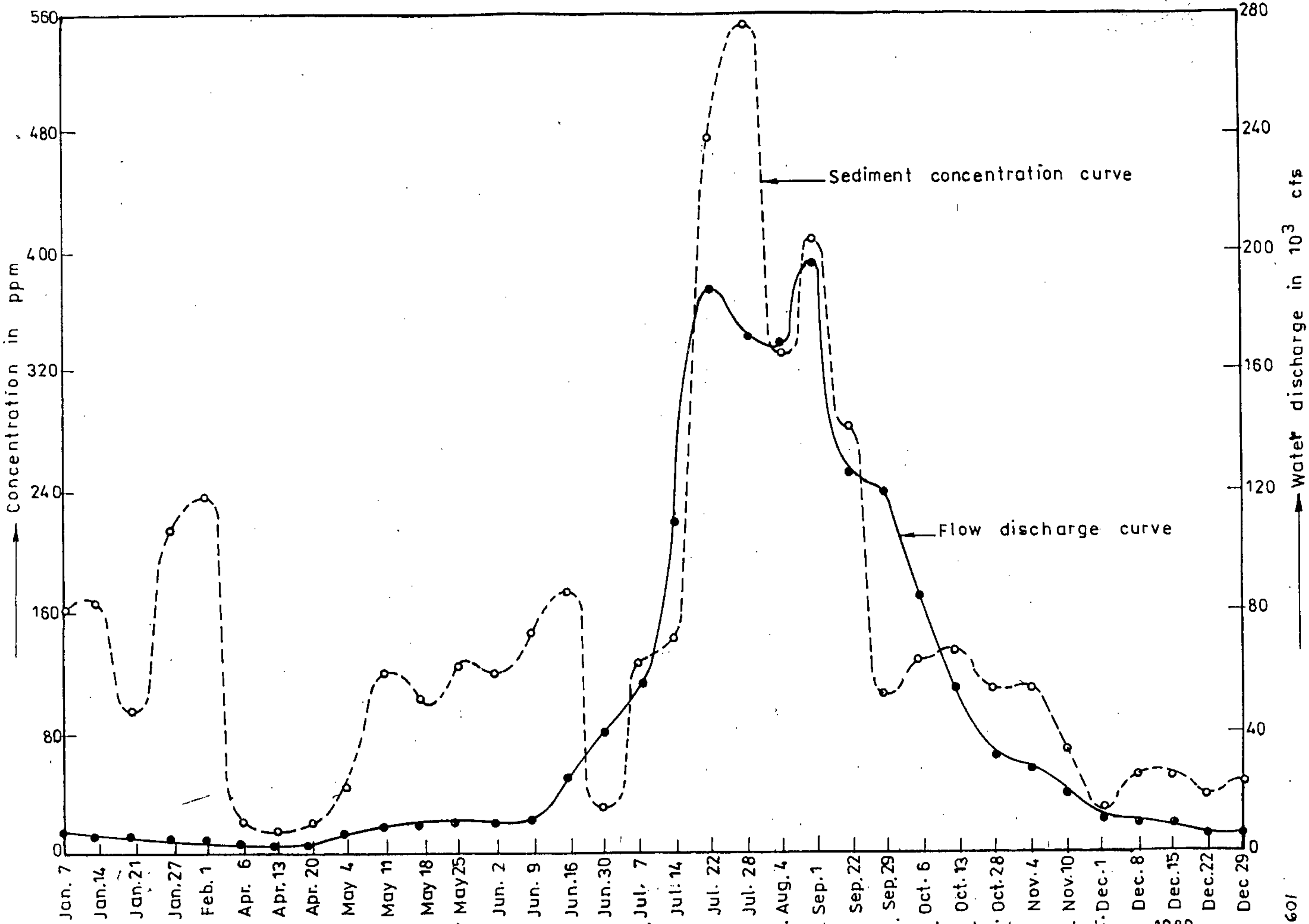


Fig. 4.4 Water discharge and sediment concentration of Gorai at Gorai rly. bridge station, 1980

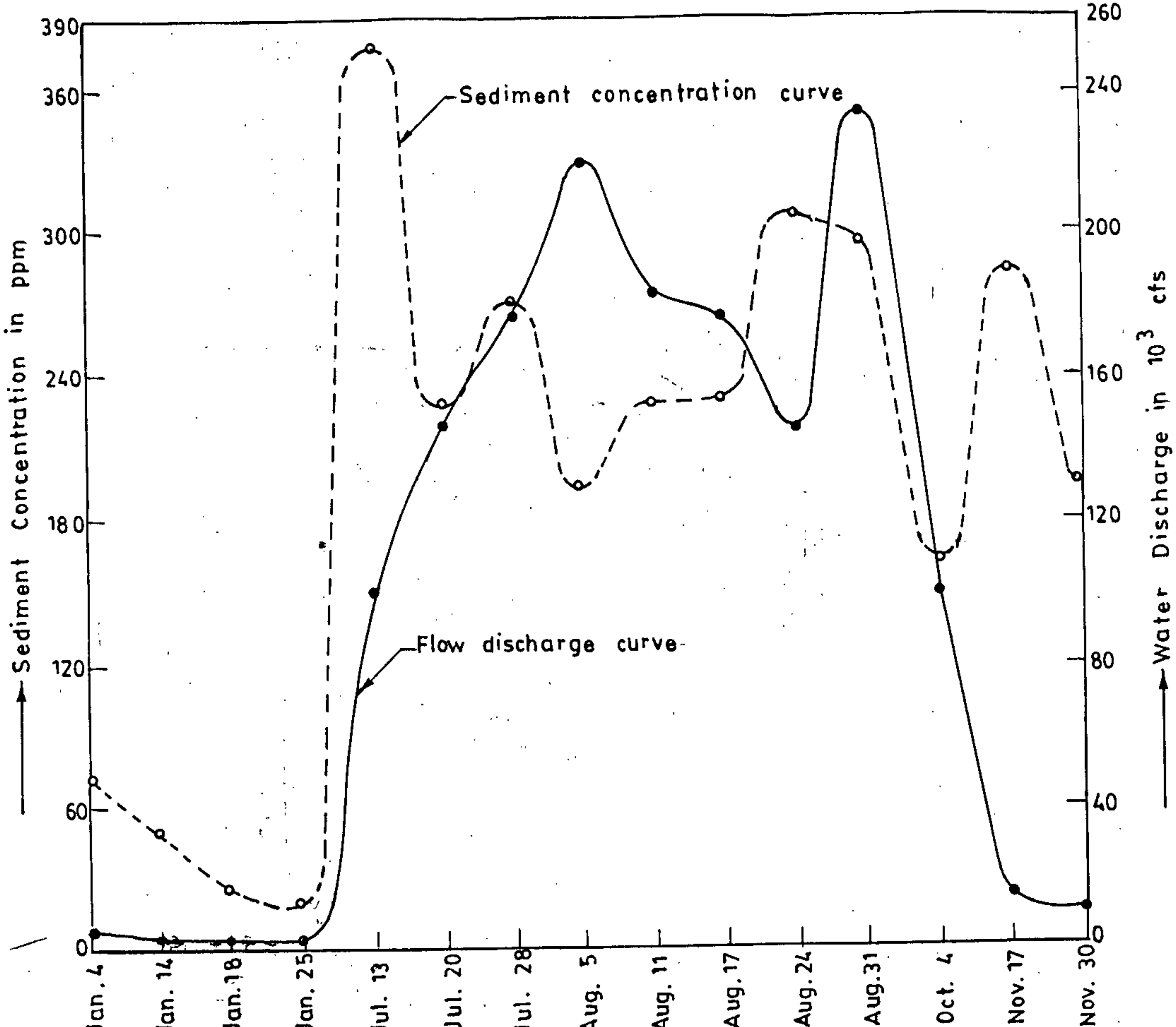


Fig. 4.5 Water discharge and sediment concentration of Gorai at Gorai rly. bridge station, 1981

110

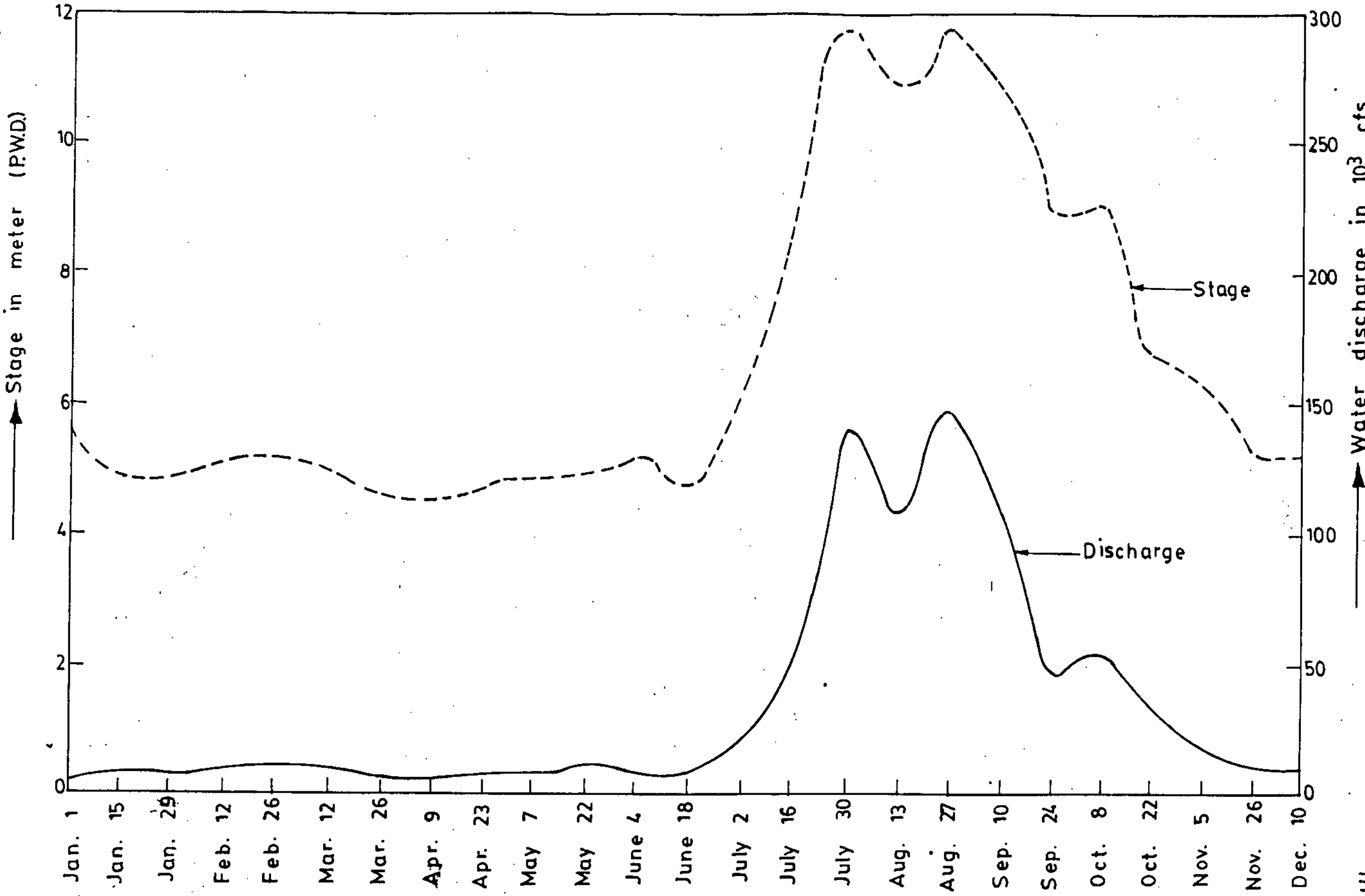


Fig. 4.6 Stage and discharge of Gorai river at Gorai railway bridge station, 1979

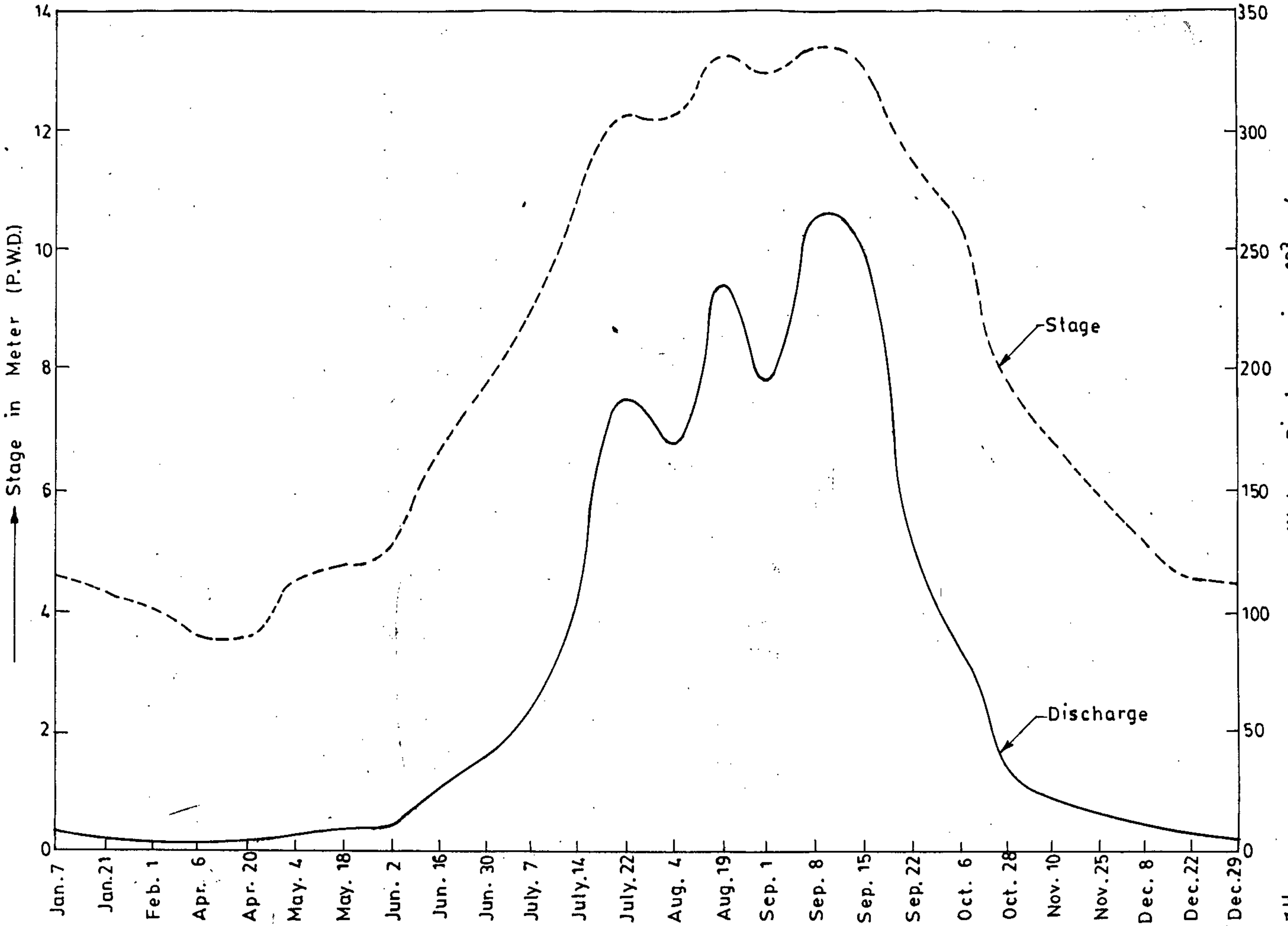


Fig. 4.7 Stage and Discharge of Gorai river at Gorai railway bridge station, 1980

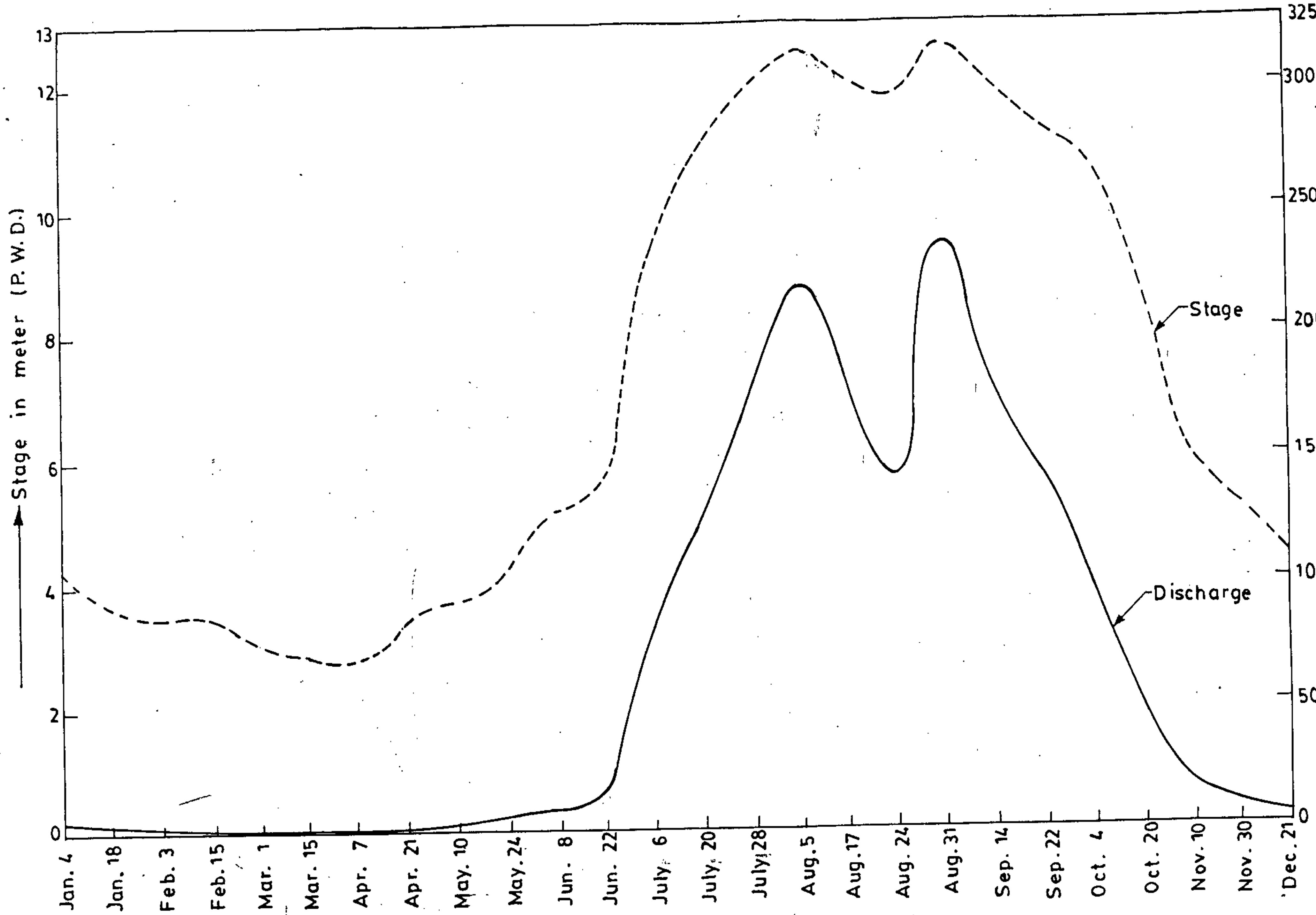


Fig. 4.8 Stage and discharge of Gorai river at Gorai railway station, 1981

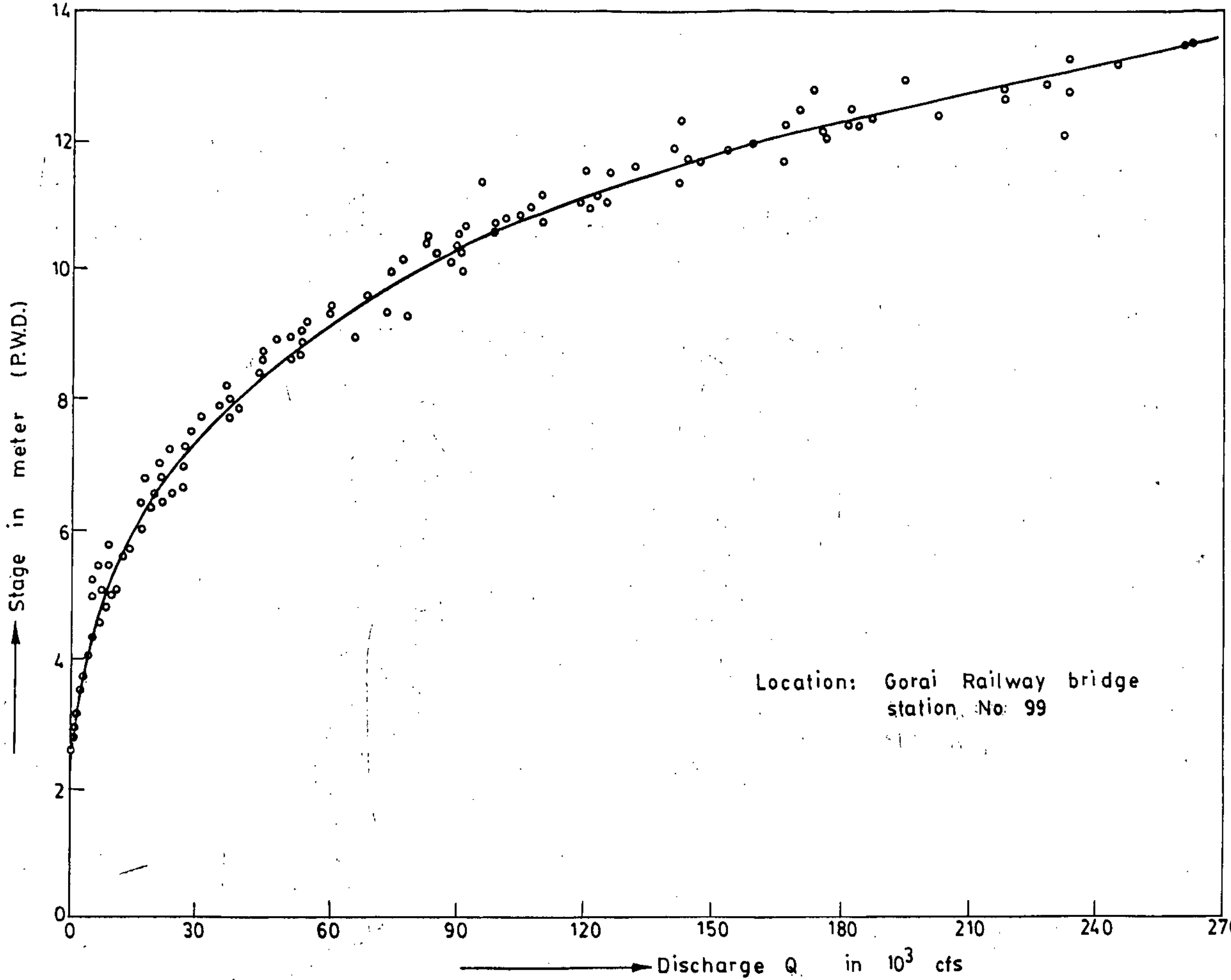
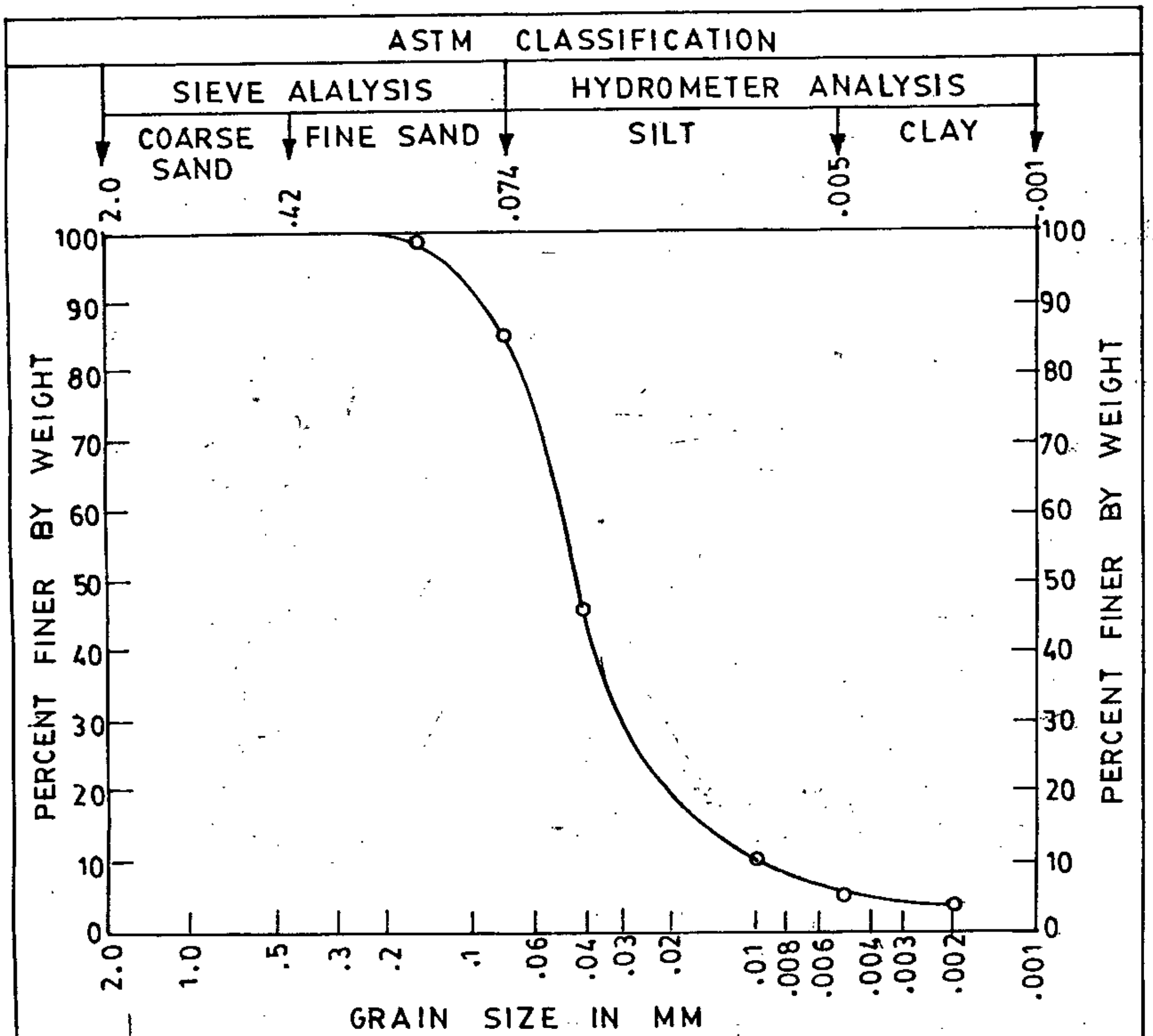


Fig. 4·9 Stage discharge relation for the Gorai river at Gorai railway bridge 1976-84

GRADATION CURVE OF BED MATERIALS

RIVER: GORAI STATION: RLY. BRIDGE



DATE	LEGEND	POSITION	DEPTH, FT.	D_{65}, mm.	D_{35}, mm	D_m, mm	SLOPE
19.11.64		—o—o—		0.057	0.033	0.048	

Fig. 5.1 Gradation Curve of Bed Material of Gorai River at Gorai Rly. Bridge Station

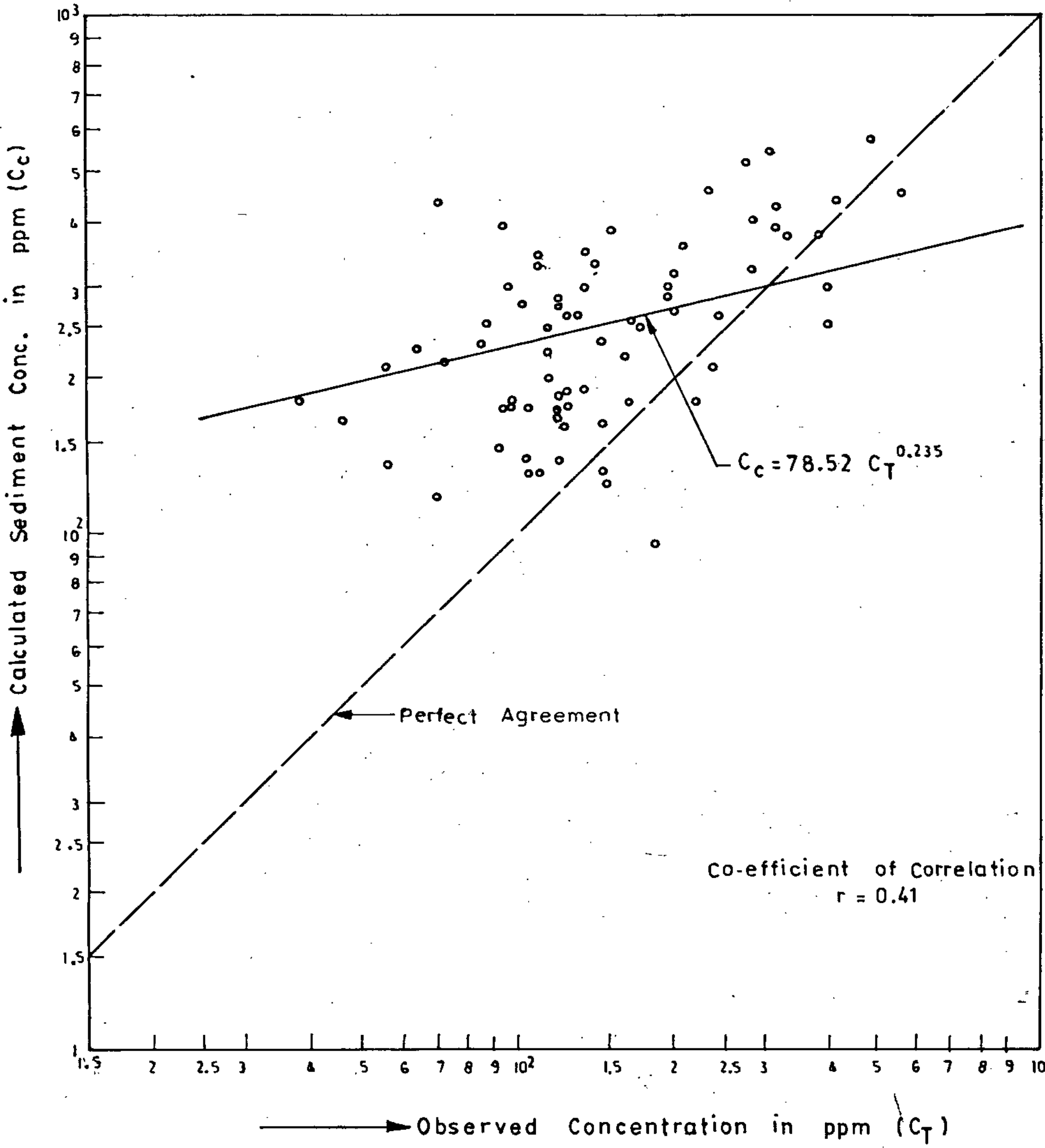


Fig. 5.2 Relation between observed sediment conc. and sediment conc. calculated by STRATHCLYDE (HOSSAIN 1984) formula.

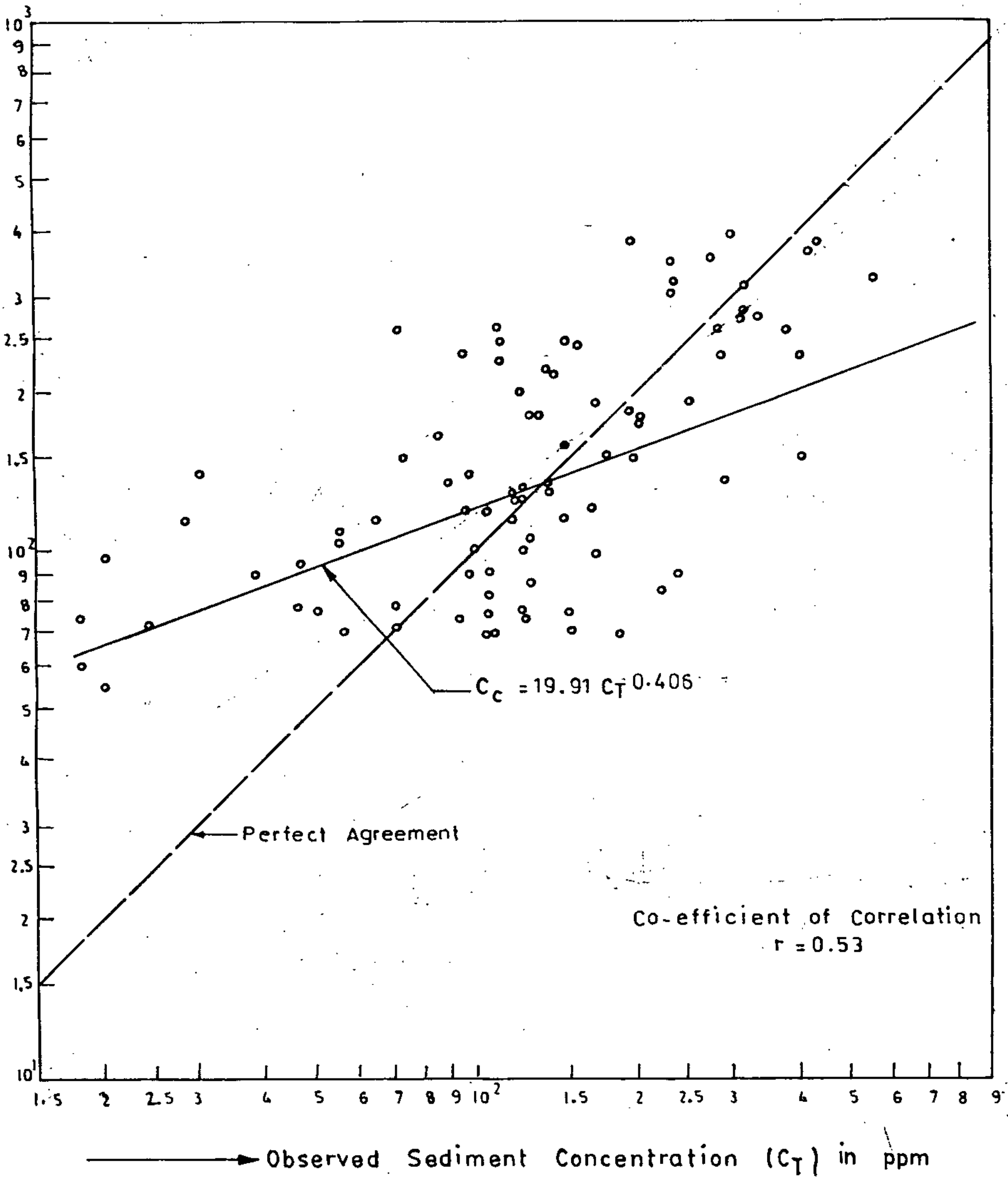


Fig. 5.3 Relation between observed sediment conc. and sediment conc. calculated by MANTZ formula

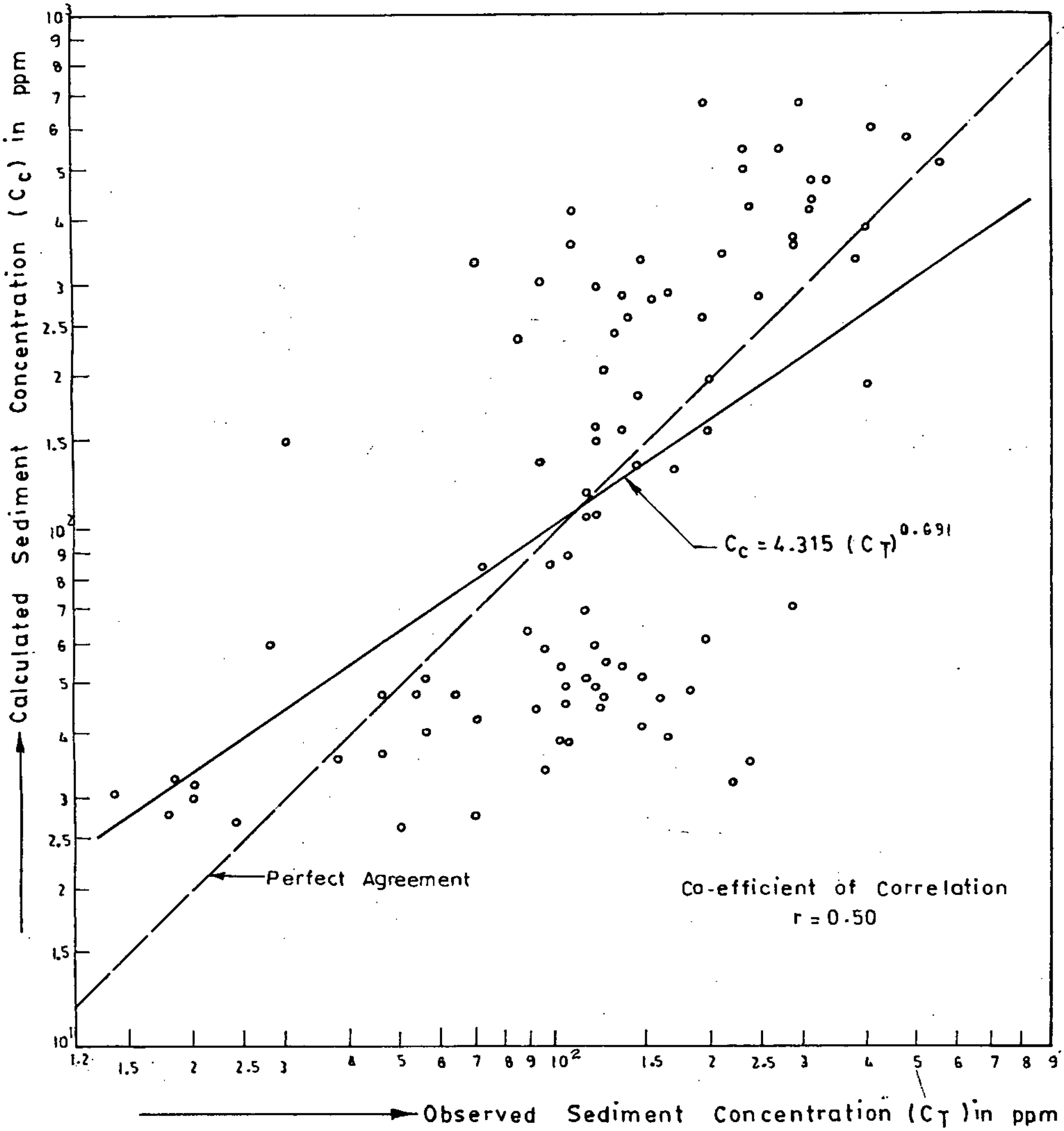


Fig. 5.4 Relation between observed sediment conc. and sediment conc. calculated by ENGELUND – HANSEN formula

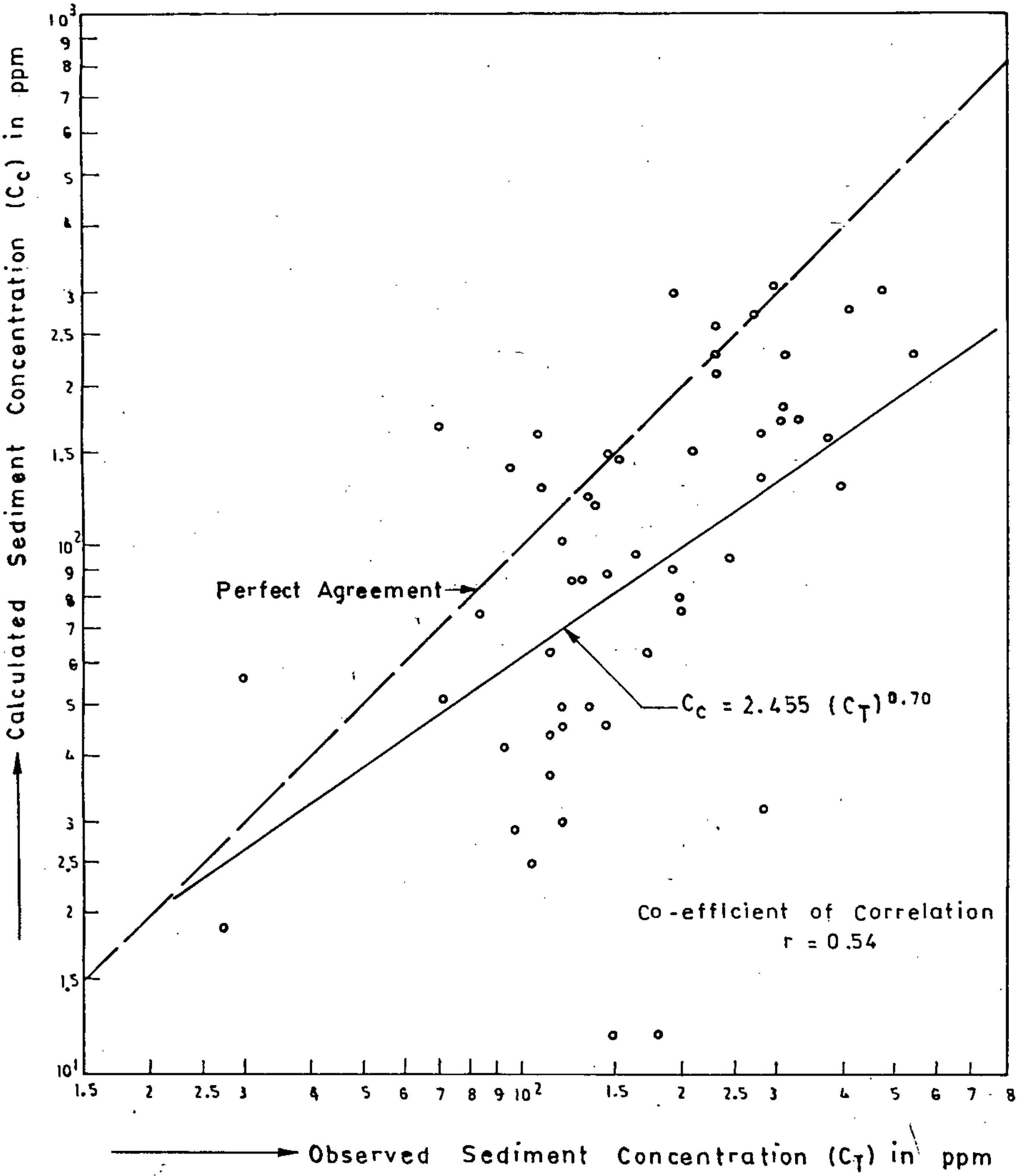


Fig. 5.5 Relation between observed sediment conc. and sediment conc. calculated by YANG formula

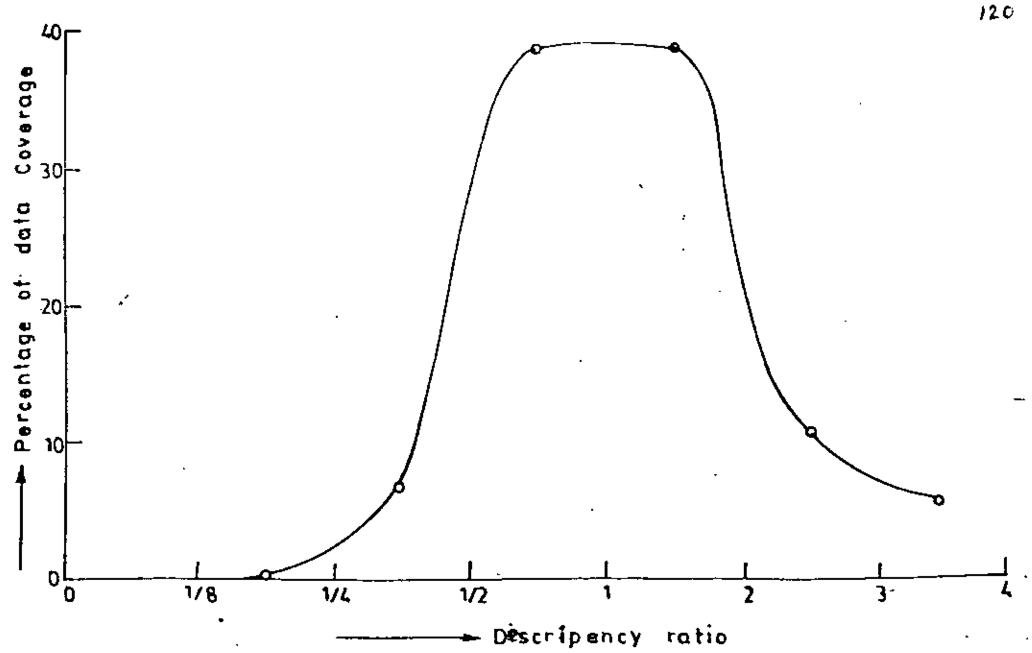


Fig. 5.6 Relationship between percentage of data coverage and descripency ratio after MANTZ equation

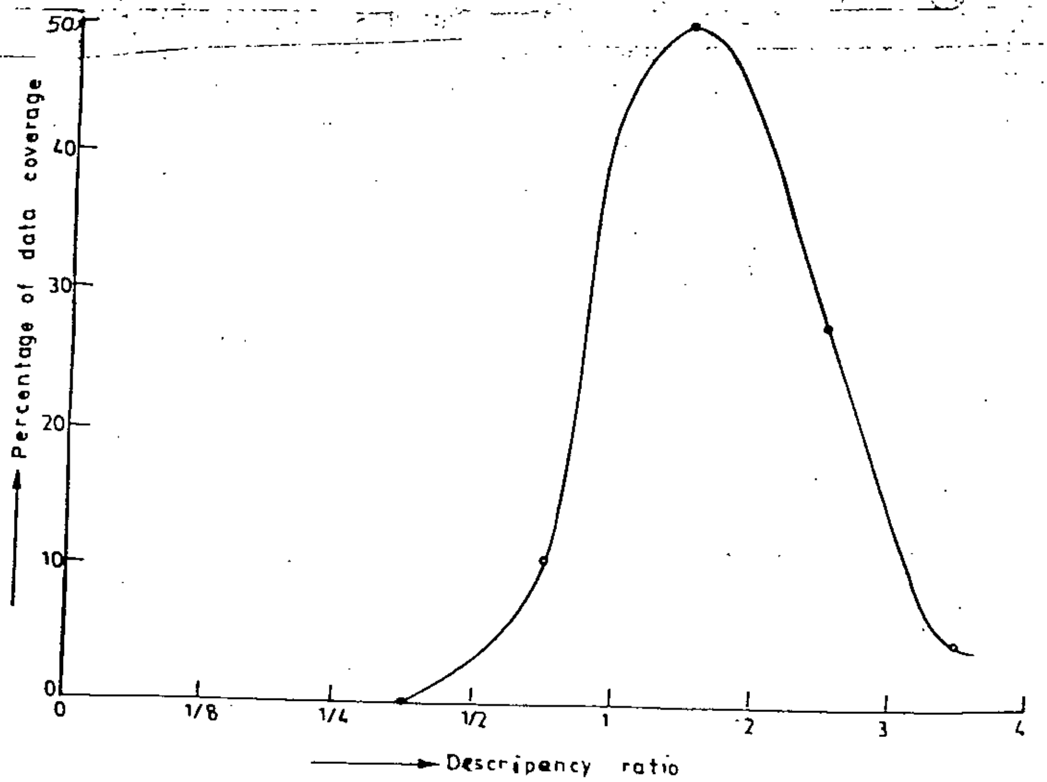


Fig. 5.7 Relation between percentage of data coverage and descripency ratio after STRATHCLYDE (Hossain 1984) formula.

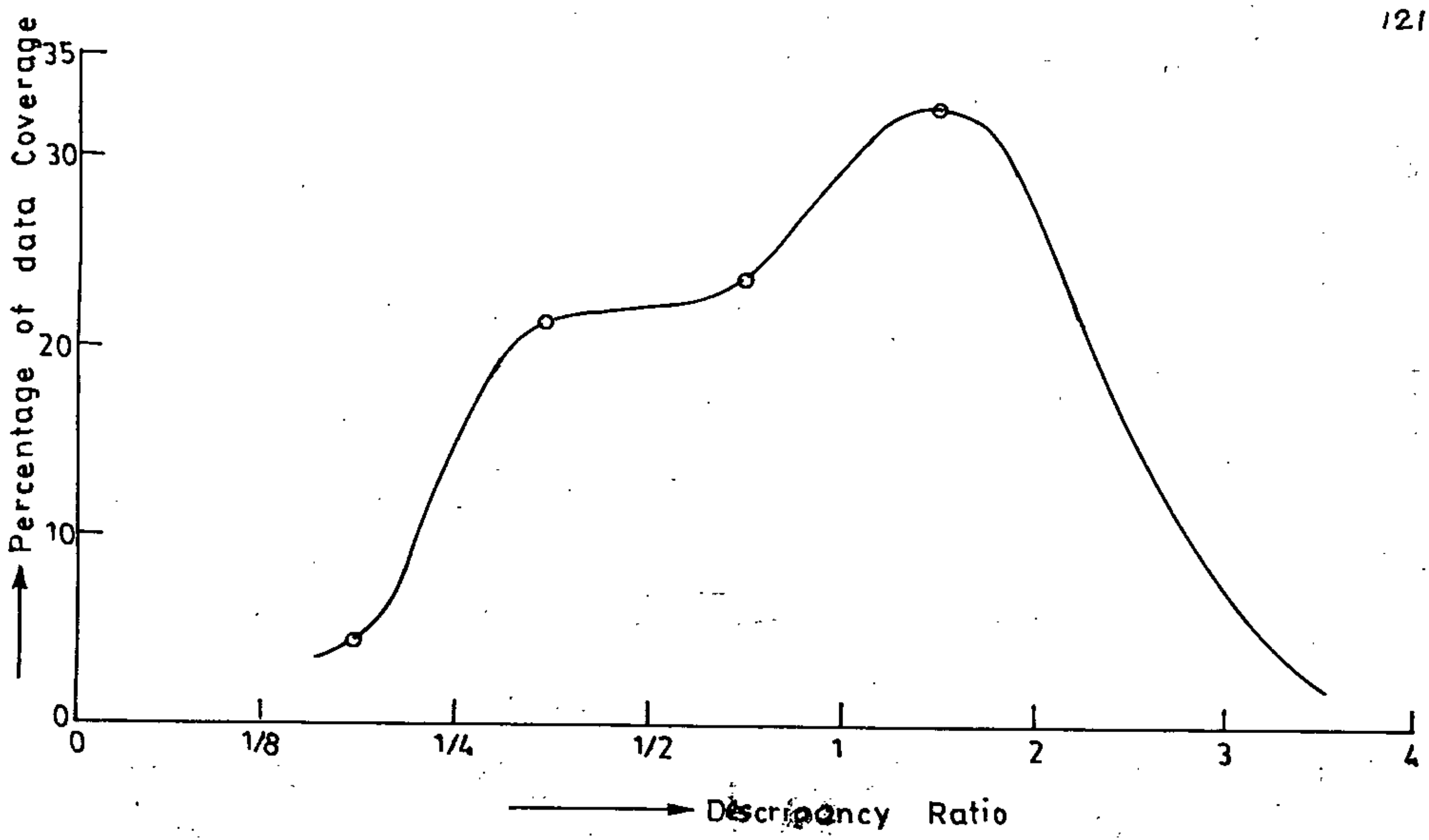


Fig. 5.8 Relationship between percentage of data coverage and descripancy ratio after ENGELUND – HANSEN formula.

127

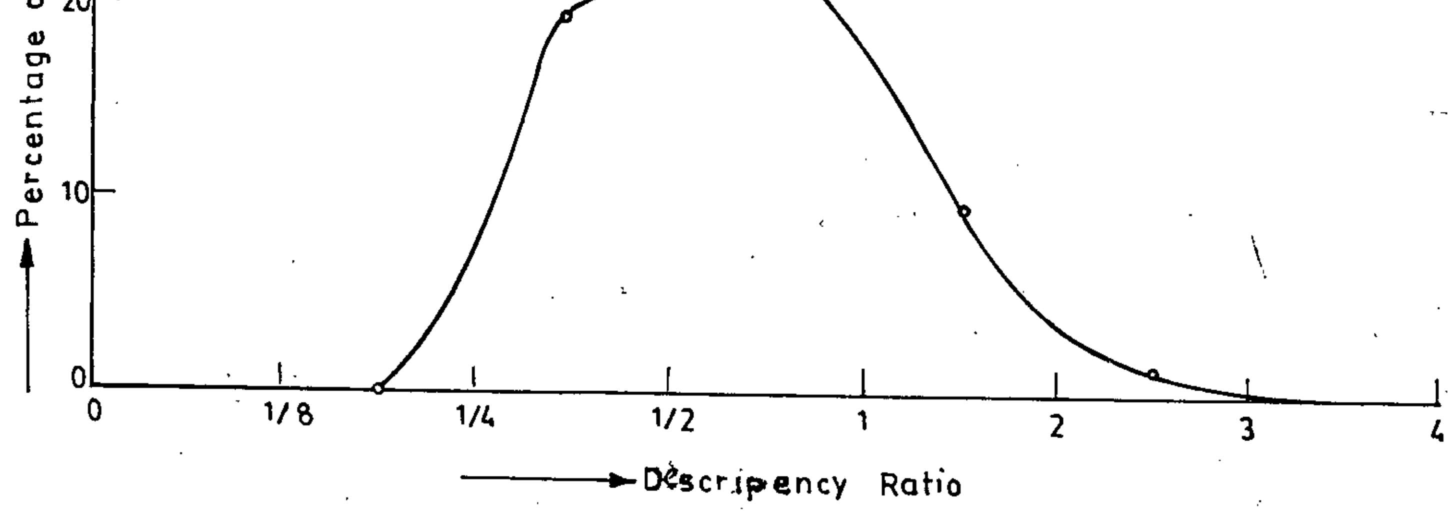


Fig. 5.9 Relation between percentage of data coverage and descripency ratio after YANG formula

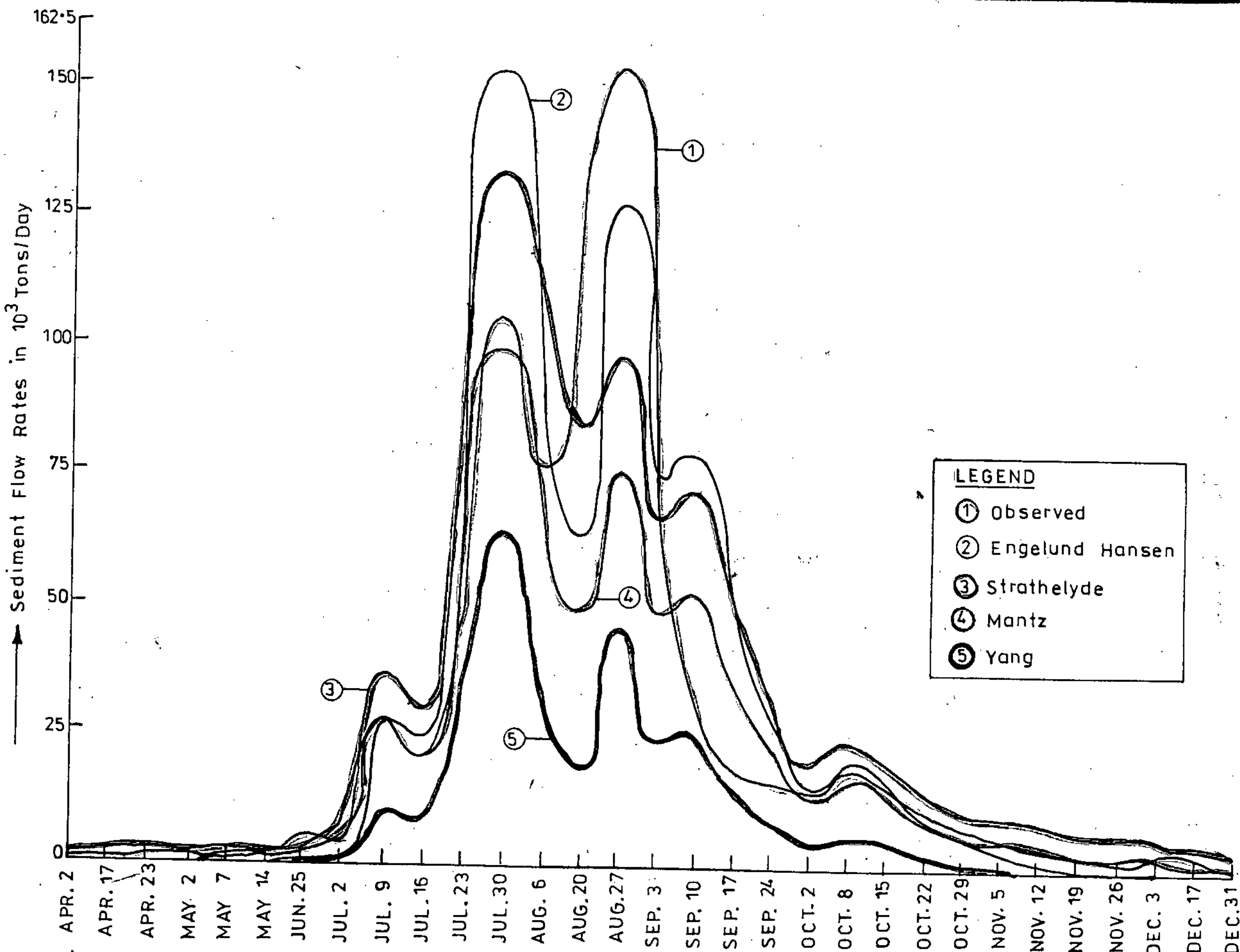


Fig. 5.10 Sediment discharge by different formula for the year 1979

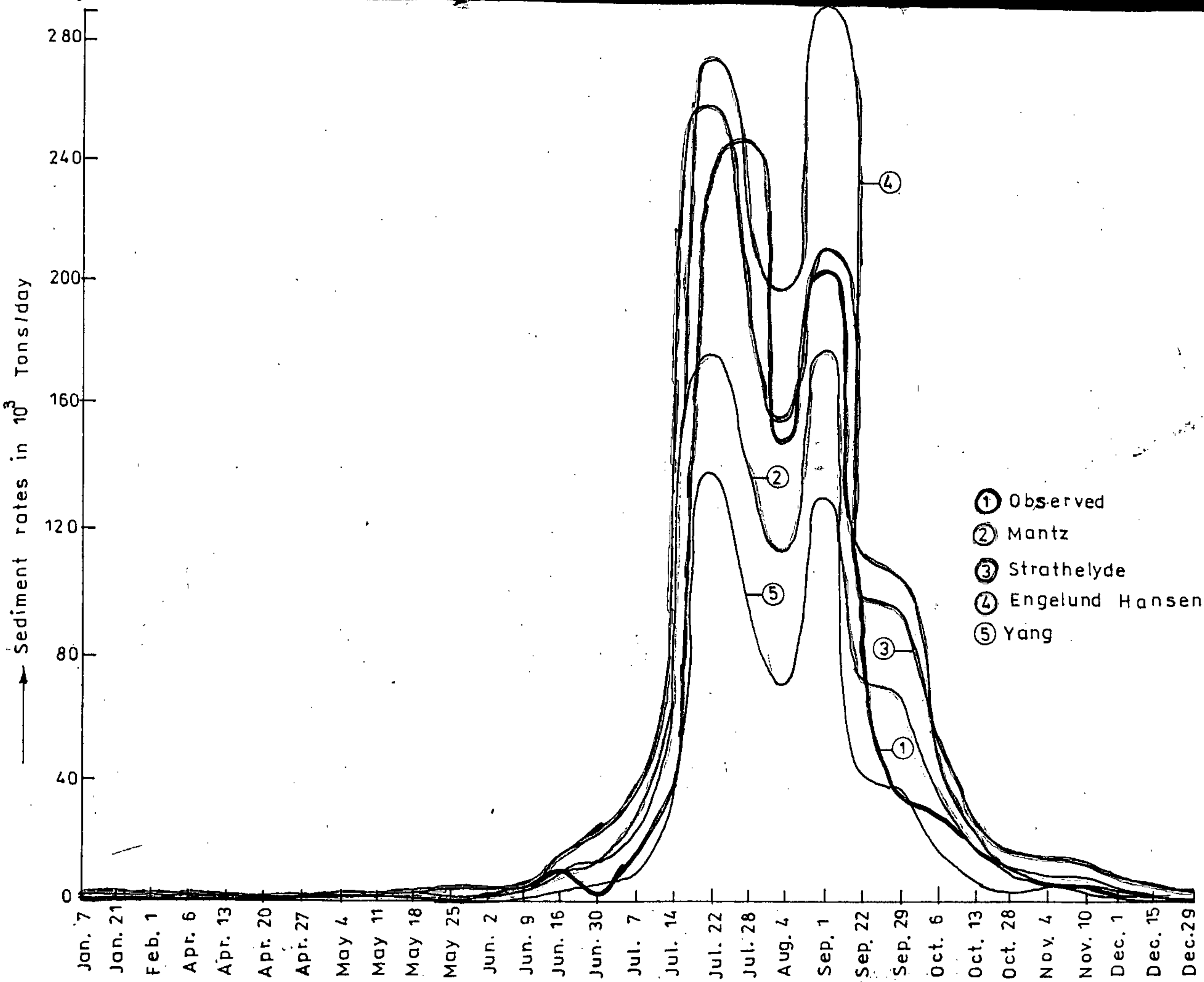


Fig. 5.11 Sediment discharge by different formula for the year, 1980

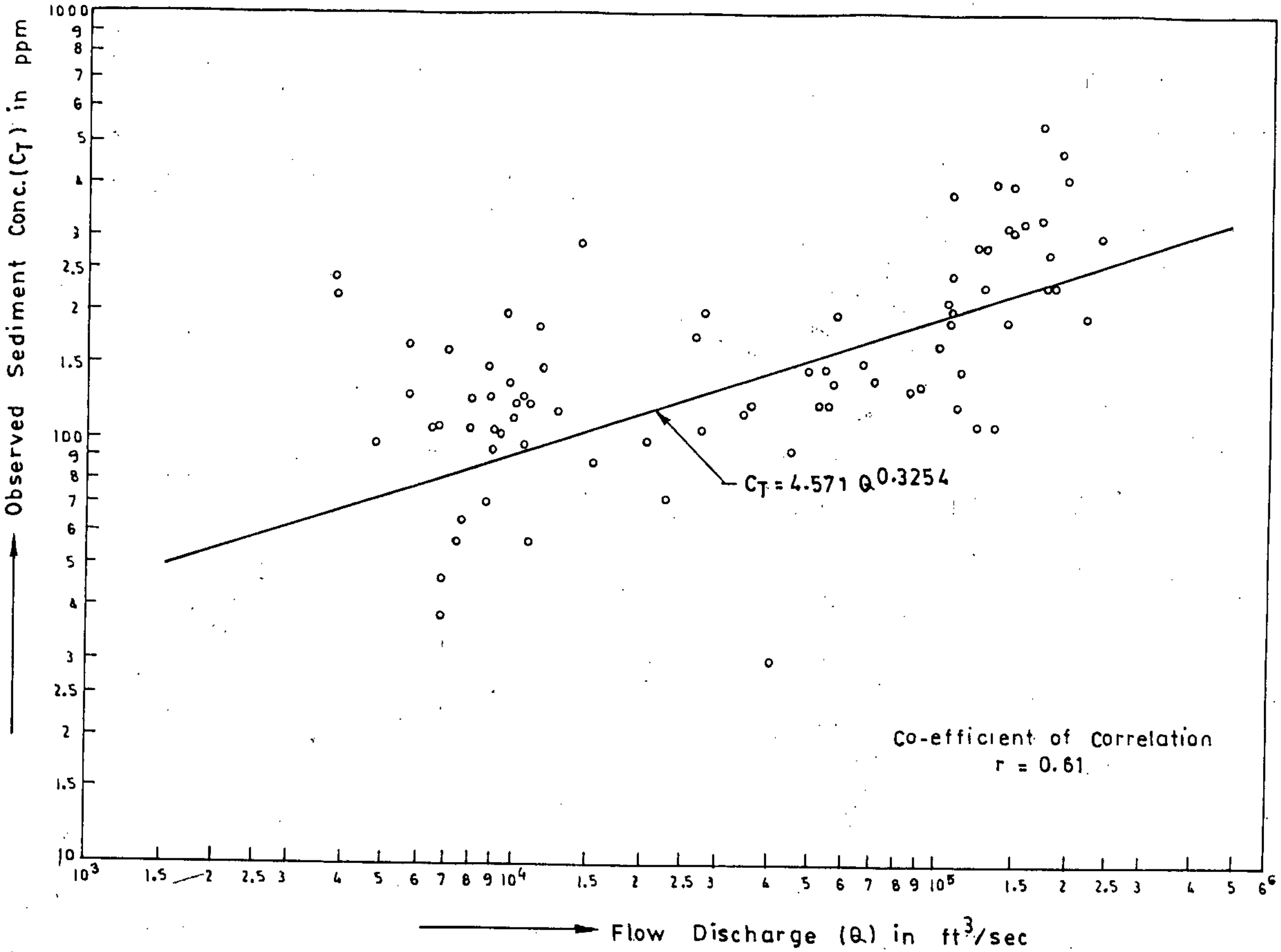


Fig. 5.12 Relation between observed sediment conc. and flow discharge of the river at Gorai rly. bridge.

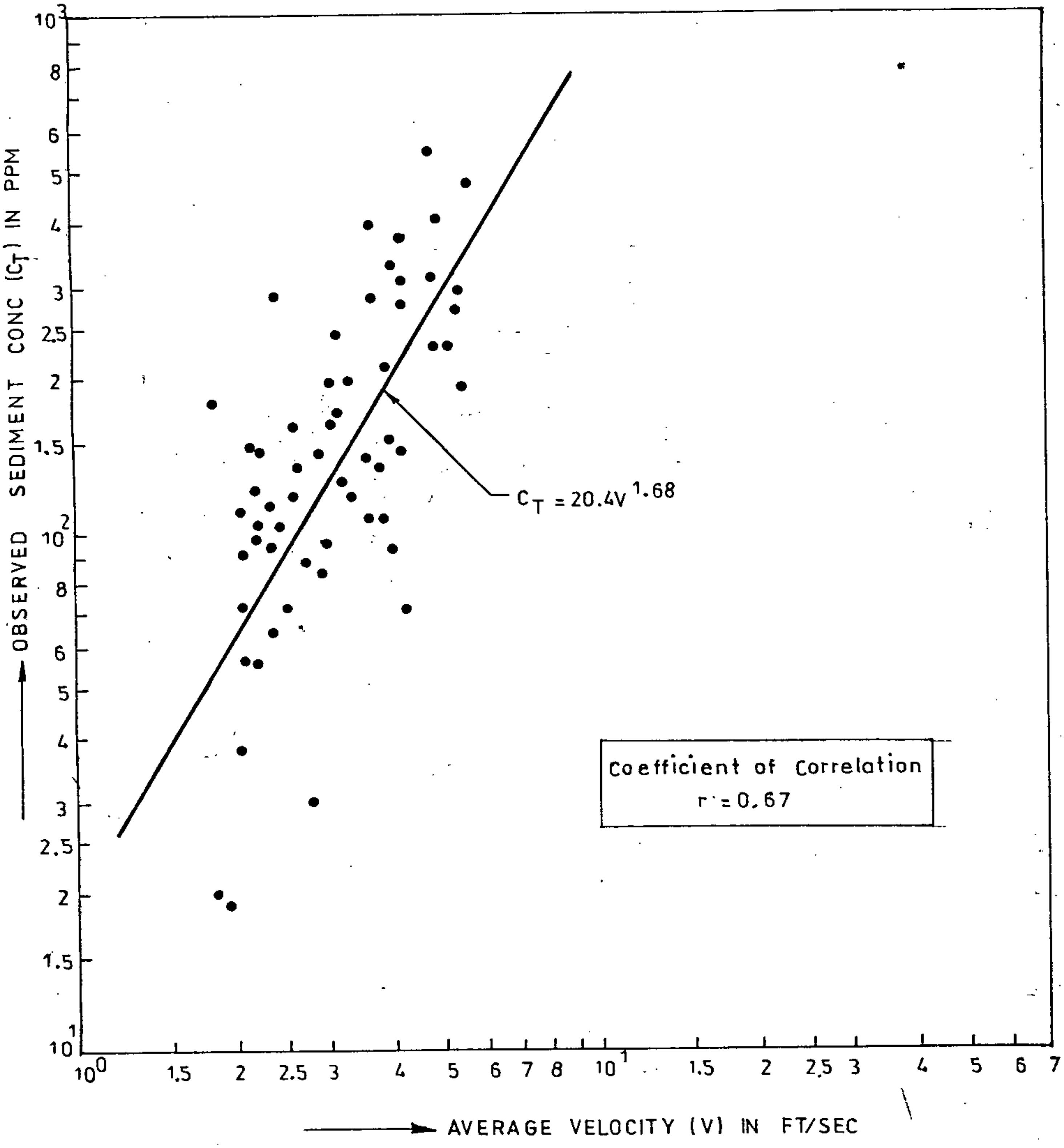


Fig. 5.13 Relation between observed sediment conc. and velocity of flow in the river Gorai at Gorai Rly. bridge station.

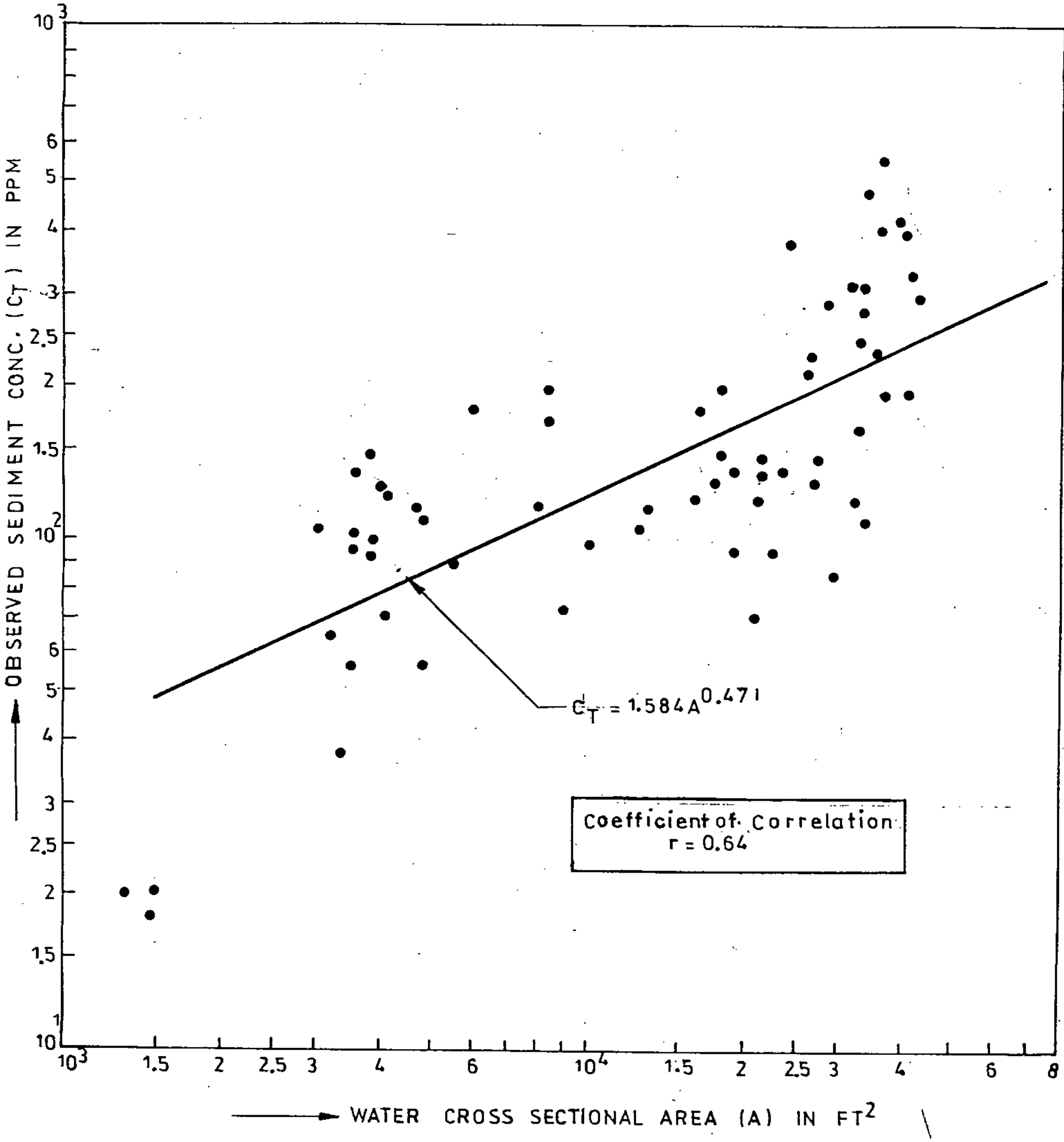


Fig. 5.14 Relation between observed sediment conc. and water cross sectional area of the river Gorai at Gorai Rly bridge station.

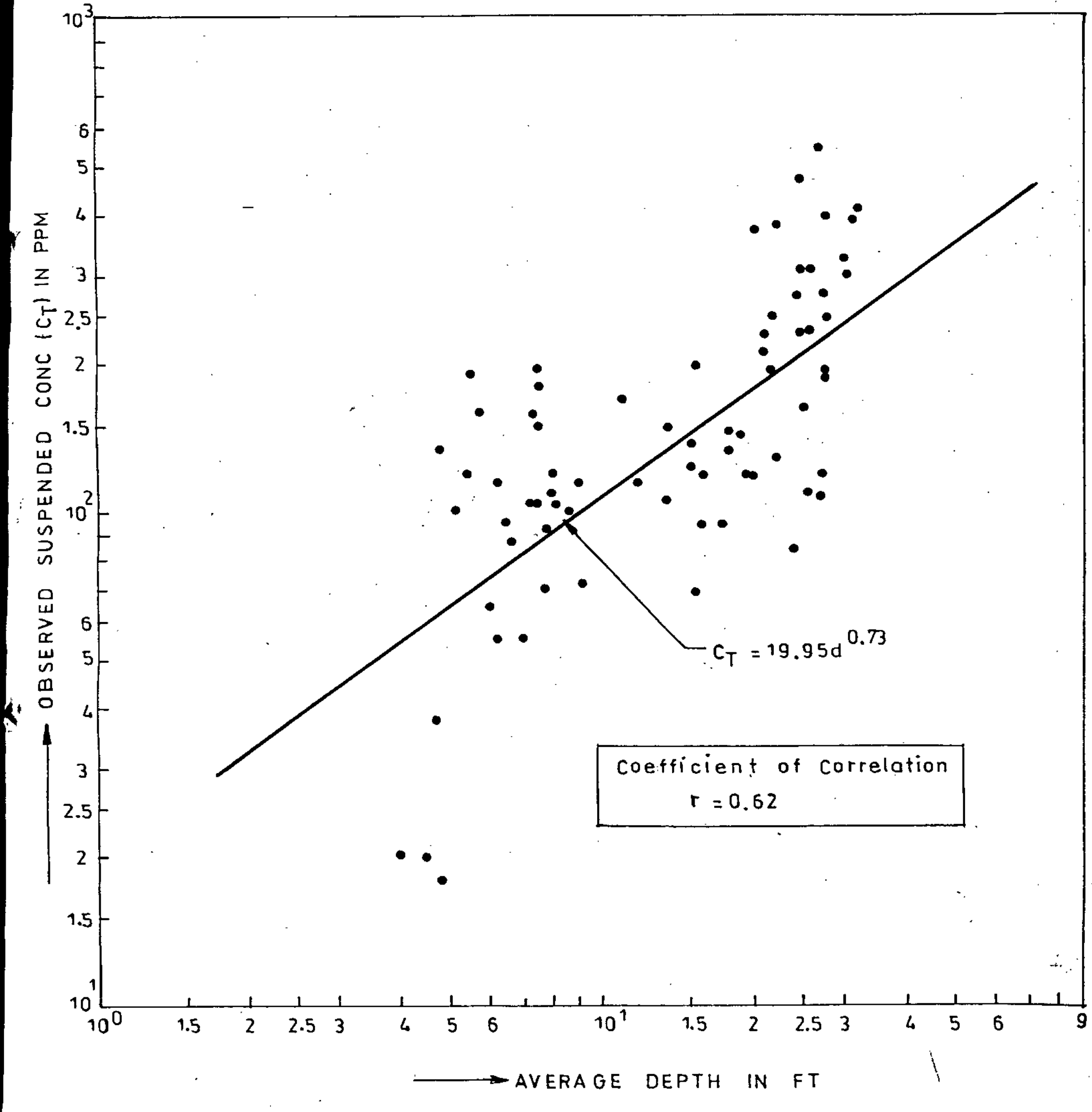


Fig. 5.15 Relation between observd sediment concentration and average depth of the river Gorai at Gorai Rly. bridge station.

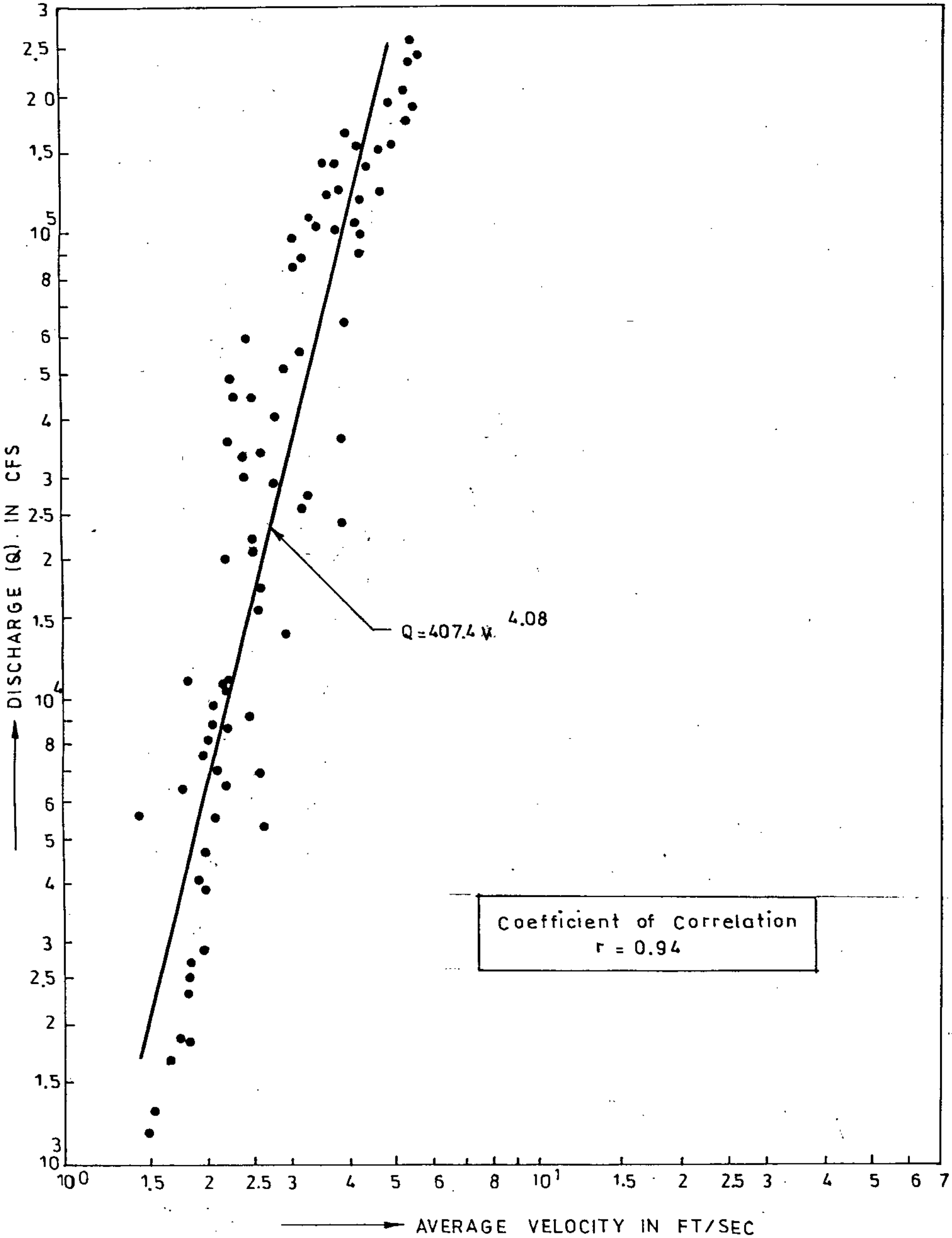


Fig. 5.16. Relation between water discharge and average velocity of the river Gorai at Gorai. Rly. bridge 1976 - 84

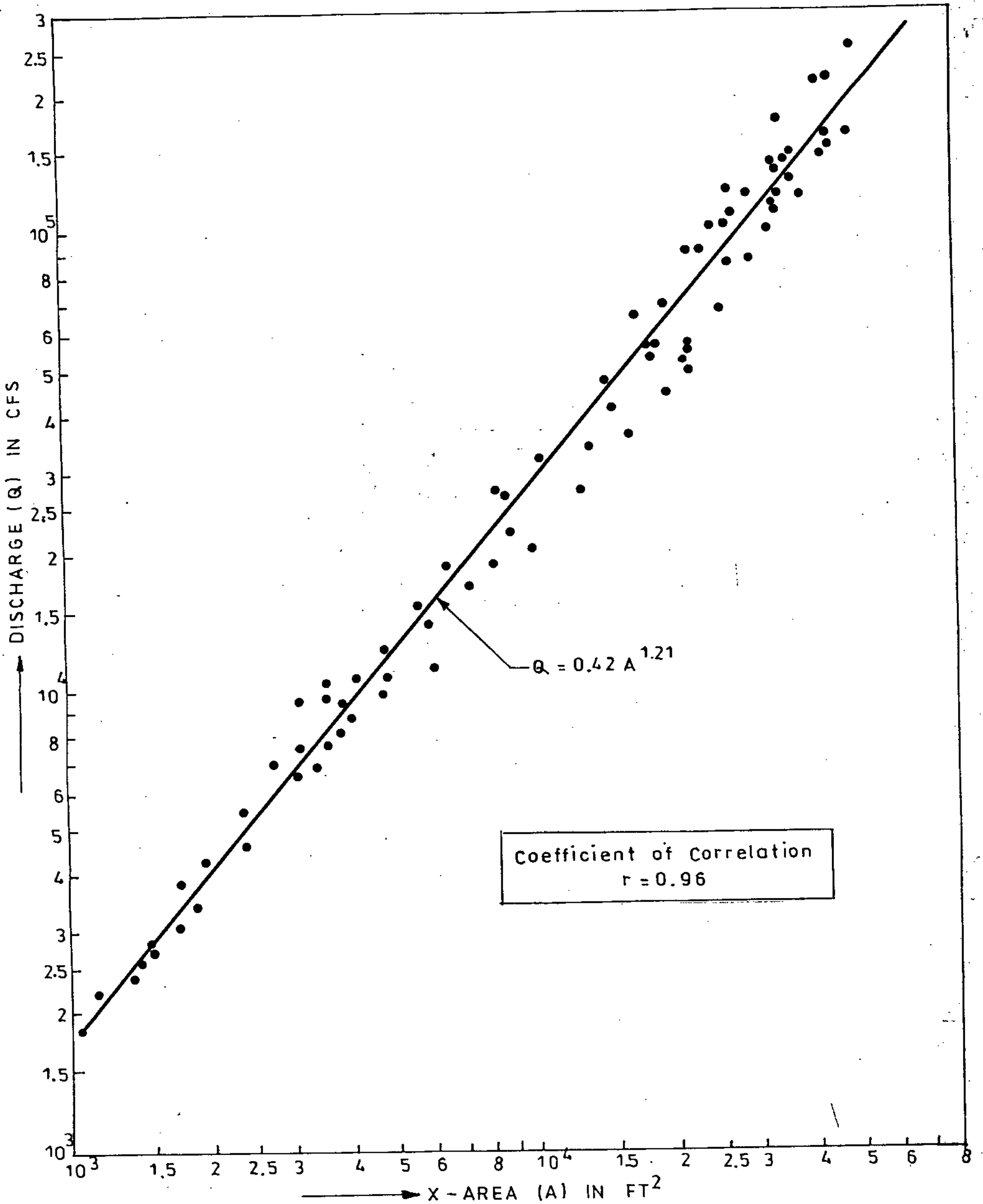


Fig. 5.17 Relation between flow discharge and water X-area of the river at Gorai Rly. bridge station 1976 - 84

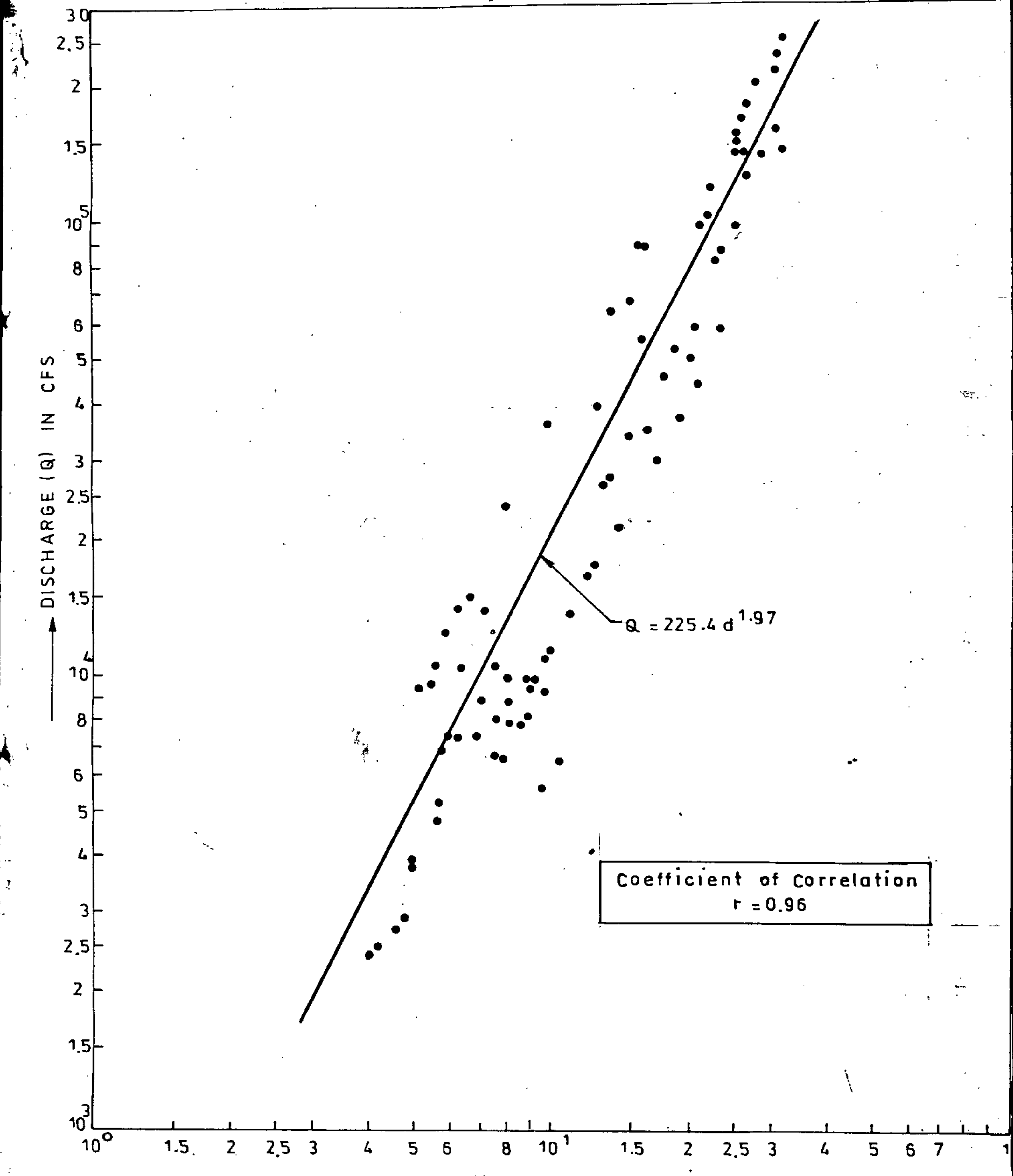


Fig.5.18 Relation between water discharge and average depth of the river Gorai at Gorai Rly. bridge 1976-84

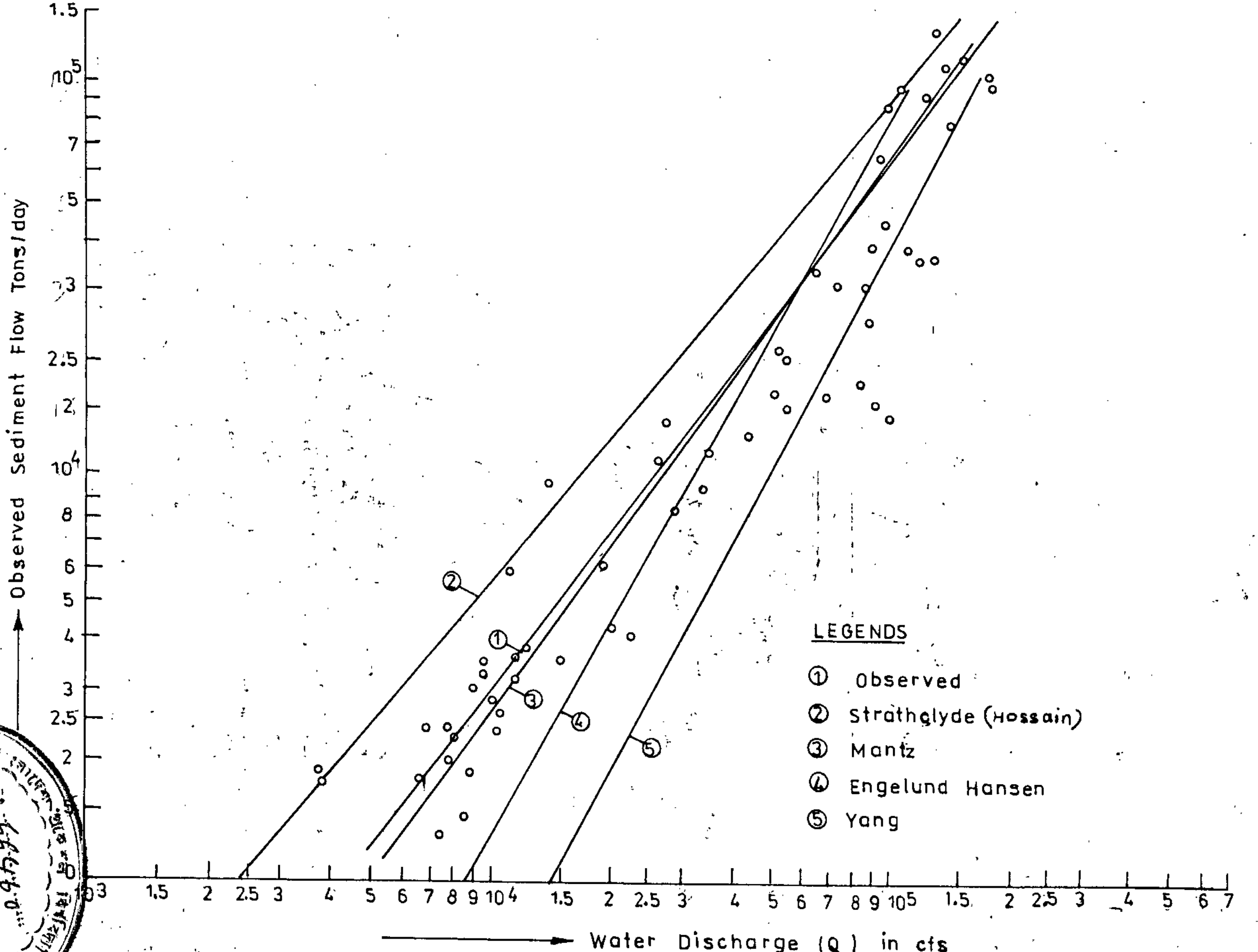


Fig. 5.19 Sediment discharge as a function of water discharge for the Gorai river obtained from observations and calculations by several formula.