APPLICABILITY OF ALLUVIAL ROUGHNESS PREDICTORS FOR GUMTI, KUSHIYARA AND MOHANANDA RIVERS

Submitted by

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

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CERTIFICATE

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

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We hereby recommend that the project work prepared by Md. Jalal Uddin entitled **"Applicability of Alluvial Roughness Predictors for Gumti, Kushiyara and Mohananda Rivers"** be accepted as fulfilling the part of the requirements for the degree of Master in Engineering (Water Resources).

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ABSTRACT

This study has been performed to determine the applicability of alluvial roughness predictors for Gumti, Kushiyara and Mohananda rivers. Eleven roughness predictors are evaluated- six are based on division of roughness and five are based on overall roughness. The selected methods are evaluated on the basis of Manning's n, Chezy's C and Darcy-Weisbach friction factor f and, where possible, on the basis of depth and/or velocity. To predict the roughness characteristics of alluvial rivers, hydrological data, e.g. discharge, cross-sectional area, mean velocity, river width, water surface slope and grain size data of bed materials are needed. The discharge stations Jibanpur, Sherpur and Chapai Nawabganj are selected for the rivers Gumti, Kushiyara and Mohananda, respectively. The required hydrological and grain size data of the selected rivers have been obtained from BWDB and SWMC, respectively.

The Manning, Chezy and Darcy-Weisbach formulae are applied to calculate the roughness coefficients n, C and f, respectively. The roughness coefficients are also predicted by using the selected roughness predictors and then these values were compared with the values obtained by Manning, Chezy and Darcy-Weisbach formulae. Accuracy of different predictors has been determined on the basis of MNE and the standard deviation of the MNE.

Out of the eleven selected roughness predictors, the Einstein and Barbarossa (1952) is the only method, which can satisfactorily predict roughness coefficients for full range of discharges for all the rivers. Some of the roughness predictors seem to predict Manning's n satisfactorily for the full range of discharge of a particular river, e.g. the Shen (1962) method for Gumti river. Shen (1962) method for the Kushiyara river predicts Manning's n about 100% higher than the calculated values for all discharges. Simons` and Richardson (1966) method predicts Manning's n about 48% higher for the Gumti river and 87% higher for the Kushiyara river than the calculated values for all discharges. Some methods predict Manning's n satisfactorily only for moderate to high range of discharge of a particular river, e.g. Haque and Mahmood (1983) and Shen et al. (1990) methods for the Kushiyara river.

Some of the methods, e.g. Engelund and Hansen (1967), Van Rijn (1993), Garde and Ranga Raju (1977), Brownlie (1983), White et al. (1987) and Karim (1995) methods predict roughness coefficients of poor quality for any discharges and all the rivers.

The Einstein and Barbarossa (1952), Shen (1962) and Brownlie (1983) methods for the river Gumti and Brownlie (1983) and Karim (1995) methods for the river Kushiyara predict depth satisfactorily.

During analyses of the hydrological data collected from BWDB for the river Mohananda, the data did not seem reliable, because the calculated Manning's n from observed data vary from 2.316 to 0.081 which are about 30 to 3 times higher than from its normal ranges. So no conclusion could be drawn for the predictors in case of the Mohananda river. But Einstein and Barbarossa (1952) and Shen (1962) methods seem to predict resonable roughness coefficients for the full range of discharges and Haque and Mahmood (1983) method seems to predict resonable results for moderate to high discharges of the Mohananda river.

The Manning's n for the Gumti and the Kushiyara rivers is maximum when the discharge is low and decreases with increasing discharge. For the river Gumti detail analyses indicate the presence of bed forms in all discharges, i.e. throughout the year. For the river Kushiyara; bed forms are only present in the range of low to medium discharges but at high discharges the river bed becomes plane. The response of grain roughness to the change in discharge is relatively less than that for form roughness. The variation of grain roughness is responsive to that of the total roughness but the variation of form roughness is similar to the total roughness.

The Manning's n for the river Gumti is maximum during the month of March when discharge is low; and attains minimum value during the months of May to October when high flow takes place. The Manning's n for the river Kushiyara is maximum during the month of February when the discharge is low and minimum during the month of July to September when the discharge is high. The Mohananda river attains the maximum discharge in the month of September and the minimum discharges in the months of December to May.

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SYMBOLS AND ABBREVIATIONS

Symbols

A	=	Cross-sectional area
B	=	Top width
C	=	Overall Chezy coefficient
C_D	=	Drag coefficient
C_s	=	Concentration of sediment
d	=	Depth of water
d_c	=	Depth of flow above the dune crest
D	=	Diameter of sediment particle
D_*	=	Dimensionless particle diameter
f	=	Darcy - Weisbach friction factor
Fr	=	Froude number
F_s	=	Force due to skin shear
F_p	=	Pressure drag
F_g	=	Grain Froude number
g	=	Acceleration due to gravity (=9.81 m/s^2)
H	=	Bed form height
H	=	Hydraulic depth
L	=	Bed form length
K_s	=	Nikuradse roughness height
n	=	Manning roughness coefficient
Q	=	Discharge
q	=	Discharge per unit width
q_*	=	Dimensionless unit discharge
Re	=	Reynolds number
Re_*	=	Boundary Reynolds number
R	=	Overall hydraulic radius
S	=	Slope of energy line
S_w	=	Water surface slope
s	=	Specific gravity of sediment
T	=	Transport stage parameter
T_*	=	excess shear
U	=	Mean velocity of flow
U_*	=	Shear velocity
U_{*cr}	=	Critical shear velocity
w_s	=	Particle fall velocity
τ_b	=	Bed shear stress
$\tau_{b,cr}$	=	Critical bed shear stress
τ_*	=	Dimensionless shear stress
δ	=	Thickness of laminar sub-layer
ρ_s	=	Density of sediment

ρ	=	Density of water
γ_m	=	Unit weight of sediment-water mixture
γ_s	=	Unit weight of sediment
γ	=	Unit weight of water
σ_g	=	Geometric standard deviation of grain particles
ν	=	Kinematic viscosity of water
μ	=	Dynamic viscosity of water
η	=	Relative depth
θ	=	Shields value
θ_{cr}	=	Critical Shields value
δ_d	=	Dune steepness value
δ_r	=	Ripple steepness value

Abbreviations

AE	=	Assistant Engineer
BWDB	=	Bangladesh Water Development Board
FAP	=	Flood Action Plan
IECO	=	International Engineering Company
LGED	=	Local Government Engineering Department
MNE	=	Mean Normalized Error
RSP	=	River Survey Project
SD	=	Standard Deviation
SDE	=	Sub-Divisional Engineer
SWMC	=	Surface Water Modelling Center

CHAPTER 1

INTRODUCTION

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1.1 General

The flow boundary of an alluvial river is not fixed, but undergoes changes in its characteristic geometry and dimensions through mutual interaction between the flow and the bed. At low discharge the velocity as well as the shear stresses are small and the bed material of the river remains at rest. But if the discharge is gradually increased, the velocity as well as the shear stresses become high and the bed material begins to move and finally set in motion, and the channel bed deforms from its original condition.

In rigid boundary open channels, the resistance to flow is due to grain roughness only, i.e. the roughness of the material forming the channel boundary. It is common for engineers to think of a rigid boundary open channel as having a single roughness value for all discharges. However, the problem of predicting the resistance to flow in alluvial rivers with mobile boundary is complicated by the fact that the configuration of the bed changes with change in flow conditions. This changing bed condition makes it impossible to describe the resistance to flow in an alluvial channel by a single-valued resistance coefficient to a particular stream.

To calculate flow in an alluvial channel an engineer is faced with the problem of determining frictional losses. Unless adequate and complete data are available for calculating roughness coefficients, evaluation of frictional effects is based almost upon the predictor applied. The element of doubt is inevitably present which forces the engineer to be conservative. The degree of conservation exercised is reflected in project costs. If the element of doubt and confusion concerning frictional losses in natural streams could be reduced to lowest practicable limit, substantial savings in the costs of many hydraulic projects undoubtedly would result. So the knowledge of the frictional resistance is required for the design of water resources projects, e.g. irrigation channel design, river improvement works, etc.

1.2 Background of the Study

The resistance to flow of clear water in a channel with fixed boundaries was studied extensively and could be predicted with a satisfactory degree of certainty. However, when a stream has a movable bed and sediment is being transported, the problem of determining the resistance is much more complicated. The problem of resistance to flow over alluvial beds has received considerable attention during the post-war years.

Attempts to determine resistance factors for alluvial channels have been based on modification of existing fixed bed equations. Mayer-Peter and Muller (1948) divided the shear stress acting on the bed into two parts: (i) the shear stress that would be present without sand waves, and (ii) the shear stress due to bed form. This is a division into skin roughness and form roughness. Einstein (1950) and Einstein and Barbarossa (1952) methods, using either the logarithmic velocity equations of Keulegan or the Manning-Strickler equation, separated the total resistance of flow into that resulting from grain friction and that resulting from the form roughness such as bank irregularities and ripples and dunes.

Einstein and Barbarossa (1952) were the pioneer workers in the field of alluvial roughness prediction. Since then a lot of empirical formulae have been developed for predicting roughness of alluvial rivers. The existing resistance relationships of alluvial channels follow two different approaches:

(i) Separate estimate of grain roughness and form drag on basis of grain size distribution of the bed material and the anticipated bed forms. The total roughness is found by adding both roughnesses together.

(ii) Integral methods in which simultaneously the continuity and the Chezy or Manning equations are solved and the total roughness is found. These methods are based on overall flow parameters such as depth of water and depth-averaged flow velocity.

The prediction of flow resistance in alluvial channels is needed for the following major purposes:

i) to estimate stage-discharge relationship,

ii) to estimate sediment transport from the hydraulic characteristics of the channel by means of transport formula,

iii) to design irrigation channels, and

iv) for river improvement works.

Moreover, knowledge of the roughness characteristics of alluvial streams is of great value when dealing with the location of bridges, training works, flood control works, navigation and channel improvement, backwater computation due to confluences and barrages, mathematical and physical modelling of flow, prediction of aggradation and degradation due to presence of hydraulic structures and so on. In all cases the prediction is concerned either with a channel for which, under a given hydraulic conditions, no observations are available or with a channel that does not yet exist. For river studies it is desired to select a predictor on the basis of available observations and to use the predictor to extrapolate to circumstances for which no data are or will be available.

Almost all of the rivers in Bangladesh are of alluvial in nature. The three mighty rivers and their tributaries and distributaries are characterized by variability of flow, the sediment transport as well as channel configuration. Stage, discharge and sediment transport change daily, seasonally and annually. Changes in stage continuously influence scour and fill patterns along the channel bed as well as the magnitude of hydraulic roughness.

Khan (1975) determined the roughness coefficient of some rivers of Bangladesh. Later on Khan (1979) developed the relationship between discharge, mean velocity of flow and depth of scour for 1966-67 flood period of the Brahmaputra river. Chowdhury (1996) studied stage-discharge relationship for the river Jamuna at Bahadurabad. The River Survey Project (FAP-24) reported that bed form roughness is the main component of the overall resistance in the river Jamuna. The validity of Soil Conservation Service (SCS) method (French, 1986) for estimating roughness coefficients for the river Jumuna was also investigated by the River Survey Project (FAP-24). Alam (1998) investigated the applicability of some alluvial roughness predictors for the Ganges river at the stations Hardinge Bridge and Baruria. He found that most of the roughness predictors over-predicts roughness at high discharges and

under-predicts at low discharges. Only the Shen (1962) method predicts the roughness coefficients of the river Ganges satisfactory. It was also observed that the grain roughness increases with increasing discharges whereas form roughness decreases with discharges and the bed forms remained throughout the year in the river Ganges.

Since Einstein and Barbarossa (1952), many studies have been done in this field and a large number of roughness predictors have been published. Some of them predict overall roughness and others are based on method of division of roughness. Every method claims a quite good accuracy of its own. Since all the rivers in Bangladesh are alluvial rivers, so the applicability of these predictors for the rivers of Bangladesh is of great importance and also interesting from engineering point of view.

The aim of the present study is to investigate the applicability of eleven roughness predictors for the rivers Gumti, Kushiyara and Mohananda. Out of these eleven roughness predictors five methods are based on overall roughness and the rest six methods are based on division of roughness. The selected overall roughness predictors are Simon and Richardson (1966), Garde and Ranga Raju (1977), Brownlie (1983), White et al. (1987) and Karim (1995). The methods based on division of roughness are Einstein and Barbarossa (1952), Shen (1962), Engelund and Hansen (1967), Haque and Mahmood (1983), Shen et al. (1990) and Van Rijn (1993).

1.3 Objective of the Study

The specific objectives of the proposed study are

(i) To estimate the Manning's roughness coefficient n, Chezy's resistance factor C and Darcy-Weisbach friction factor f of the Gumti, Kushiyara and Mohananda rivers using the streamflow data collected by Bangladesh Water Development Board.

(ii) To investigate the validity of the methods proposed by Simon and Richardson (1966), Garde and Ranga Raju (1977), Brownlie (1983), White et al. (1987) and Karim (1995) for predicting total roughness and Einstein and Barbarossa (1952), Shen (1962), Engelund and Hansen (1967), Haque and Mahmood (1983), Shen et al. (1990) and Van Rijn (1993) for predicting the grain, form and total roughnessess of the above mentioned rivers. The value of n, C and f for total roughness predicted by these methods will be compared with those obtained in (i) above.

CHAPTER 2

REVIEW OF LITERATURE

2.1 Introduction

Rivers of Bangladesh are classified into large (major) and minor rivers. There are also large numbers of streams that originate from haors and beels and act as drainage channels or khals. Past studies have been mainly related to large rivers and very few of minor rivers. Related past studies are discussed here.

2.2 Review of Previous Studies in Bangladesh

The large rivers in Bangladesh are the Brahmaputra (Jamuna), the Ganges (Padma) and the Meghna . These rivers are considered to belong to the main river system of Bangladesh. The Teesta, Dharla, Dudkumar, Karotoa, Mohananda, Surma, Kushiyara, Gumti, Gorai, Arial khan, Dhaleswari etc. are among the minor rivers.

Many studies have been performed for the large rivers. Due to their great tendency to change course, large rivers have been subject to investigation and studies. Minor rivers, however, have been studied while formulating water resources projects involving these rivers.

The paper that first gave a comprehensive overview of the geology of the basin of the main rivers in Bangladesh was published by Morgan and Mclntyre (1959). The papers by Coleman (1968, 1969) on "Brahmaputra river: channel processes and sedimentation" are the bench- mark papers for the sedimentology of the main rivers of Bangladesh. Khan (1975) determined the roughness coefficients of some rivers of Bangladesh. Later on Khan (1979) developed the relationship between discharge, mean velocity of flow, depth of scour and maximum deposition for 1966-1967 flood period of the Brahmaputra river. Hossain (1989) studied the geomorphological characteristics of the Ganges up to the Ganges-Jamuna confluence. The study was mainly on the shifting of river, dominant discharge and the variation of the width to depth ratio. Hossain (1992) also computed dominant discharge for the Ganges utilizing data at the Hardinge Bridge station.

Within the framework of project formulations, quite a large number of studies have been carried out regarding the major rivers. For examples, NEDECO (1968) in its report on "Survey of inland water-ways and ports", studied the variations in the elevation of river bed and sediment transport of the rivers Ganges and Padma. IECO in 1980 studied the rivers of South-Western area under South-Western regional plan. NEDECO also studied the river Ganges in connection with "Rehabilitation of Ganges - Kobodak Project" during 1983. Under the Flood Action Plan - 9B, studies were made on the Meghna and the Padma rivers. The analyses of the morphological data yielded an insight into the morphological characteristics of the Padma river. The River Survey Project (FAP-24) conducted jointly by the Delft Hydraulics and the Danish Hydraulic Institute was initiated in 1992 and completed in 1997. Along with other relevant activities carried out, it surveyed the bed material composition, mineralogical properties of bed material and sediment transport rate for the Ganges-Padma and the lower Meghna river systems. The River Survey Project (FAP-24) produced the sediment rating curve for both fine and suspended sediment transports for Hardinge Bridge and Baruria for the Ganges and Goalunda for the Padma.

Alam (1998) investigated the applicability of some alluvial roughness predictors for the Ganges river at the stations Hardinge bridge and Baruria. It was an intensive work to verify the applicability of alluvial roughness predictors in the alluvial river of Bangladesh. He used discharge data of 18 years for both the stations. Number of observations used were 923 for Hardinge bridge and 577 for Baruria. The roughness coefficients n, C and f were predicted using thirteen selected alluvial roughness predictors and then compared with n, C and f calculated by the Manning, Chezy and Darcy-Wiesbach formulae. The accuracies of the predictors were determined on the basis variation of error in the range of ±10%, ±20% and ±30% and on the basis of MNE. According to Alam (1998) most of the predictors over-predicts roughness at high discharges and under-predicts at low discharges. A few of them, however, predicts Manning's n reasonably in medium as well as high discharges. Only Shen (1962) method predicts roughness coefficients for the full range of discharge of the Ganges river satisfactorily. Alam (1998) also observed that grain roughness n′ increases with increasing discharge but form roughness decreases with increasing discharge. The bed of the Ganges river never attains to plane bed, neither at low discharges nor at high discharges, i.e. the bed forms remain throughout the year. According to analyses of Alam (1998) even anti-dunes do not form because the maximum Froude number was 0.331 at Hardinge bridge and 0.315 at Baruria, to form

anti-dunes minimum Froude number should be 0.6 (Yalin, 1992). He also found that the influence of form roughness on total roughness at high discharges is negligible but increases gradually with decreasing discharge. At low discharges form roughness is the major component of total roughness where as at high discharges grain roughness is dominant in the total roughness.

Study on minor rivers has specially been advanced during preparation of regional plans under FAP. North-East Regional Plan (FAP-6) covers the Kushiyara, North-West Regional Plans (FAP-2) covers the Mohananda river and South-East-Regional Plan (FAP-5) covers the Gumti river. However, all these service being important minor rivers, have also been studied during different project preparation, viz. Gumti Phase-1, Kalni-Kushiyara pilot dredging project etc.

2.3 Roughness Characteristics

The roughness characteristics of a river can be defined by resistance coefficients, shear stress, shear velocity, roughness height, sediment properties, sediment transport, etc.

2.3.1 Manning's n

In 1889 the Irish engineer Robert Manning presented a formula, which was later modified to its present form (Chow, 1959)

$$U = \frac{1}{n} R^{2/3} S^{1/2} \qquad \text{(In SI units)} \qquad (2.1)$$

And

$$U = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(In FPS units)} \qquad (2.2)$$

where, U = mean velocity of flow

R = hydraulic radius

S = slope of the energy line

n = co-efficient of roughness, specifically known as Manning's n.

The Manning's n is obtained from Eq. (2.2) as

$$n = \frac{1}{U} R^{2/3} S^{1/2} \tag{2.3}$$

2.3.2 Chezy's C

As early as 1769 the French engineer Antoine Chezy developed the first uniform-flow formula known as the Chezy formula, which is usually expressed as

$$U = C\sqrt{RS} \tag{2.4}$$

where, U = mean velocity of flow

R = hydraulic radius

S = slope of the energy line

C = factor of flow resistance, called Chezy's C

The Chezy's C is obtain from Eq. (2.4) as

$$C = \frac{U}{\sqrt{RS}} \tag{2.5}$$

2.3.3 Darcy - Wiesbach friction factor f

The Darcy-Wiesbach formula, primarily used to assess flow resistance in pipe flow, is given by

$$h_f = f \frac{L}{d} \frac{U^2}{2g} \tag{2.6}$$

where, h_f = frictional loss

f = friction factor

d = diameter of pipe

g = acceleration due to gravity

The above equation can also be applied to flow in an open channel using d = 4R and the energy gradient, $S = h_f/L$. Then Eq. (2.6) can be rewritten for the friction factor as

$$f = \frac{8gRS}{U^2} \tag{2.7}$$

2.3.4 Relation between n, C and f

The relationships between Manning's n , Chezy's C and Darcy-Weisbach friction factor f are given by

$$C = \sqrt{\frac{8g}{f}} \tag{2.8}$$

and

$$C = \frac{1}{n} R^{1/6} \tag{2.9}$$

2.3.5 Strickler formula

The strickler formula (Strickler, 1923) has been developed from observations on boulder rivers in Switzerland and this formula is valid for hydraulically rough surfaces only. The Chezy's roughness coefficient C, according to Strickler, is given by (French, 1986)

$$C = 25\left[\frac{R}{K_s}\right]^{1/6} \tag{2.10}$$

where, K_s is the Nikuradse equivalent sand grain roughness.

The Strickler formula for grain roughness n′ of Manning's n is (Chang, 1988)

$$n = \frac{(D_{50})^{1/6}}{21.1} \tag{2.11}$$

where, D_{50} is the median particle diameter.

2.3.6 White-Colebrook formula

White and Colebrook derived an expression for Chezy's C by integrating the velocity distribution in the transition regime. According to White and Colebrook, the Chezy's coefficient for a hydraulically rough boundary is given by

$$C = 18\log\left(\frac{12R}{K_s}\right) \tag{2.12}$$

2.3.7 Roughness height K_s

Nikuradse introduced the concept of an equivalent sand grain roughness, e.g. the size of uniform sand grains pasted on a surface for classifying the roughness of different surfaces. This concept of equivalent sand roughness was named after Nikuradse and is referred to as the Nikuradse equivalent sand grain roughness (K_s). This is a parameter which characterises not only the vertical size of the roughness elements but also their orientation, geometric arrangement and spacing.

Various researchers carried out experiments or analyzed data in order to determine the roughness height (K_s). An overview of grain roughness values found in literature is given below:

Einstein and Barbarossa	(1952) :	$K_s = D_{65}$
Engelund and Hansen	(1967) :	$K_s = 2D_{65}$
Ackers - White	(1973) :	$K_s = 1.25D_{35}$
Hey	(1973) :	$K_s = 3.5D_{84}$
Mahmood	(1971) :	$K_s = 5.1D_{84}$
Van Rijn	(1984) :	$K_s = 3D_{90}$

According to French (1986), the value of K_s for alluvial rivers are given below:

1.	Flat uniform sand bed	:	$K_s = D_{90}$
2.	Flat graded sand bed	:	$K_s = D_{65}$ to D_{90}
3.	Sand ripples / dunes	:	$K_s =$ 1/2 ripple / dune height

2.3.8 Shear stress τ_0

When water flows in an open channel, a force is developed that acts in the direction of flow on the channel bed. This force, which is simply the pull of water on the wetted area, is known as the tractive force or shear force or drag force. The average value of the tractive force per unit wetted area is called the shear stress and is represented by (Chow, 1959)

$$\tau_0 = \rho g R S \qquad (2.13)$$

In hydraulics the shear in the direction of motion is important. The shear velocity is another parameter having dimensions of velocity and defined as

$$U_* = \sqrt{\frac{\tau_0}{\rho}} \tag{2.14}$$

Since $\tau_o = \rho gRS$, the shear velocity can also be expressed as

$$U_* = \sqrt{gRS} \tag{2.15}$$

2.4 State of Flow

The state or behavior of open channel flow is governed basically by the effects of viscosity and gravity relative to the inertial forces of the flow.

2.4.1 Effect of viscosity

Depending on the value of Reynolds number the flow may be laminar, turbulent, or transitional depending on the effect of viscosity relative to inertia. The flow is laminar if the viscous forces are so strong relative to the inertia forces that viscosity plays a significant role in determining the flow behavior. The flow is turbulent if the viscous forces are weak relative to the inertia forces. In turbulent flow the fluid particles moves in irregular paths, which are neither smooth nor fixed but still represent the forward motion of the fluid. Between laminar and turbulent flows there is a mixed or transitional state. The Reynolds number expresses the ratio between inertial and viscous forces and can be written as

$$Re = \frac{UL}{\nu} \tag{2.16}$$

where, U = a characteristic velocity, taken to be the mean velocity of flow

L = a characteristic length, taken to be equal to the hydraulic radius R

ν = kinematic viscosity of water

When

Re <500, the flow is laminar,

Re>12500, the flow is turbulent, and

500<Re<12500, there is a mixed or transitional state in which the flows can be classified neither as laminar nor turbulent.

2.4.2 Effect of gravity

The prime motivating force for open channel flow is that due to gravity and is equal to the component of the weight of water along the channel slope. The effect of gravity relative to inertia on open channel flow is determined by a dimensionless number called the Froude number which is expressed as

$$F_r = \frac{U}{\sqrt{gh}} \tag{2.17}$$

where, h = hydraulic depth of flow

g = acceleration due to gravity

when

Fr = 1, the flow is in a critical state with the inertial and gravitational forces in equilibrium,

Fr<1, the flow is in a subcritical state and the gravitational forces are dominant, and

Fr>1, the flow is in a supercritical state and the inertial forces are dominant.

2.5 Properties of Sediments

Properties of sediments include the properties of the particles and those of the sediment mixture. The former consists of the particle size, shape, fall velocity and so on, and the later includes the size distribution, specific weight, angle of repose and so forth.

2.5.1 Cohesive and non-cohesive sediments

For the non-cohesive sediments there is no physical-chemical interaction between individual particles. For cohesive sediments the physical-chemical interaction between particles plays a significant role. This is important for initiation of motion and also for transportation of sediment. Alluvial rivers are non-cohesive in character. For this reason aspects of non-cohesive sediments will be discussed.

2.5.2 Grain size

The size distribution is obtained by the separation of a sample into a number of size classes, known as mechanical analysis. For fine sediments (D<70 μm) the size is usually determined by measuring the fall velocity. For coarser sediments (D>70 μm) the size is frequently determined by sieving, meaning that size is expressed using the definition of the sieve diameter and the results are presented as cumulative. The fraction or percentage by weight of a sediment that is smaller than a given size is known as sediment size.

2.5.3 Particle size curve

Sediment comprises of particles of a range of sizes. A particle size curve depicts the cummulative size distribution of a sediment sample. A particle size curve is characterised by the following parameters.

2.5.2.1 Median particle size

Median particle size (D_{50}) is defined as the size at which 50% of the particles by weight is smaller.

2.5.2.2 Mean particle

Mean particle size is defined as $D_m = \Sigma(\frac{p_i d_i}{100})$ (2.18)

where, P_i = percentage by weight of each particle size fraction d_i.

2.5.4 Geometric standard deviation

The geometric standard deviation σ_g is defined as

$$\sigma_g = \sqrt{\frac{D_{84}}{D_{16}}} = \frac{1}{2}\left[\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}}\right] \quad (2.19)$$

in which, D_{16} = particle size at which 16% by weight is smaller

D_{84} = particle size at which 84% by weight is smaller.

2.5.5 Particle fall velocity

The fall velocity w_s of a sphere is the fall velocity when the fluid drag force on the particle is in equilibrium with the gravity force. It is given by (Van Rijn, 1993)

$$w_s = \left[\frac{4(s-1)gD}{3C_D}\right]^{0.5} \tag{2.20}$$

where, D = sphere diameter

s = specific gravity of falling particle

C_D = drag coefficient

g = acceleration due to gravity

The drag coefficient C_D is a function of the Reynolds number $Re = \frac{w_s D}{\nu}$ and shape factor. The terminal fall velocity of non-spherical sediment particles can be determined from the following formulae:

$$w_s = \frac{(s-1)gD^2}{18\nu} \quad for\ 1 < D \leq 100\mu m \tag{2.21}$$

$$w_s = \frac{10\nu}{D}\left[\left(1 + \frac{0.01(s-1)gD^3}{\nu^2}\right)^{0.5} - 1\right] \quad for\ 100 < D < 1000\mu m \tag{2.22}$$

$$w_s = 1.1[(s-1)gD]^{0.5} \quad for\ D \geq 1000\mu m \tag{2.23}$$

2.5.6 Angle of repose

The angle of repose is a direct function of the median size and angularity (Lane, 1955; Simon, 1957). This parameter is also a function of size, shape and porosity of the material. This parameter is important in critical tractive force study, design of stable channels and other hydraulic problems. The angle of repose increases with decreasing roundness and size of particles. For sand particles the angle of repose lies in the range of 30^0 to 40^0 (Van Rijn, 1993).

2.6 Initiation of Motion

The motion of a particle on the river bed depends on the interaction of two opposing forces, e.g. the applied force and the resisting force. The former is caused by the hydrodynamics of flow; the

later is associated with the submersed weight. The particle will be set in motion if the disturbing forces overcome the resistance. The magnitude of forces on the particle depends on particle dimensions and shape, roughness of the bed, local mean velocities and velocity fluctuations. In turbulent flow conditions, the velocities fluctuate in space and time and that the initiation of motion is not merely a deterministic phenomenon but a stochastic process as well.

2.6.1 Threshold value for initiation of motion

When the hydrodynamic force acting on a grain of sediment or an aggregate of cohesive particles has reached a value that, if increased slightly, will put the grain or aggregate into motion, critical or threshold conditions are said to have been reached. When critical condition is obtained, values of such quantities as the bed shear stress, the stage of a stream or its mean velocity are said to have their threshold values. Under threshold conditions, a stream is also said to be competent to move its sediment, i.e. it has reached its competent shear stress, stage and velocity. Three different threshold values are often used in literature. They are the critical shear stress, the critical Shields value and the critical shear velocity.

2.6.1.1 Critical shear stress

The critical shear stress is defined as the shear stress developed on a particle where the applied forces are just balanced by the resisting force. Dubuat (1786) first introduced the critical shear stress for the initiation of motion. The fluid forces acting on a sediment particle resting on a horizontal bed consist of skin friction force and pressure force. The skin friction force acts on the surface of the particles by viscous shear. The pressure force consisting of a drag force and a lift force is generated by pressure difference along the surface of this particle. The threshold value according to the critical shear stress ($\tau_{b,cr}$) is defined as follows:

i. In case the bed shear stress, τ_b, is smaller than the critical bed shear stress ($\tau_{b,cr}$), the sediment particle on the bed will be in equilibrium.

ii. In case the bed shear stress, τ_b, is greater than or equal to the critical bed shear stress ($\tau_{b,cr}$), the sediment particle will be set in motion.

The critical shear stress can be obtained from Fig. 2.1. For the particle diameter of non-uniform bed material, the median particle size D_{50} should be used.

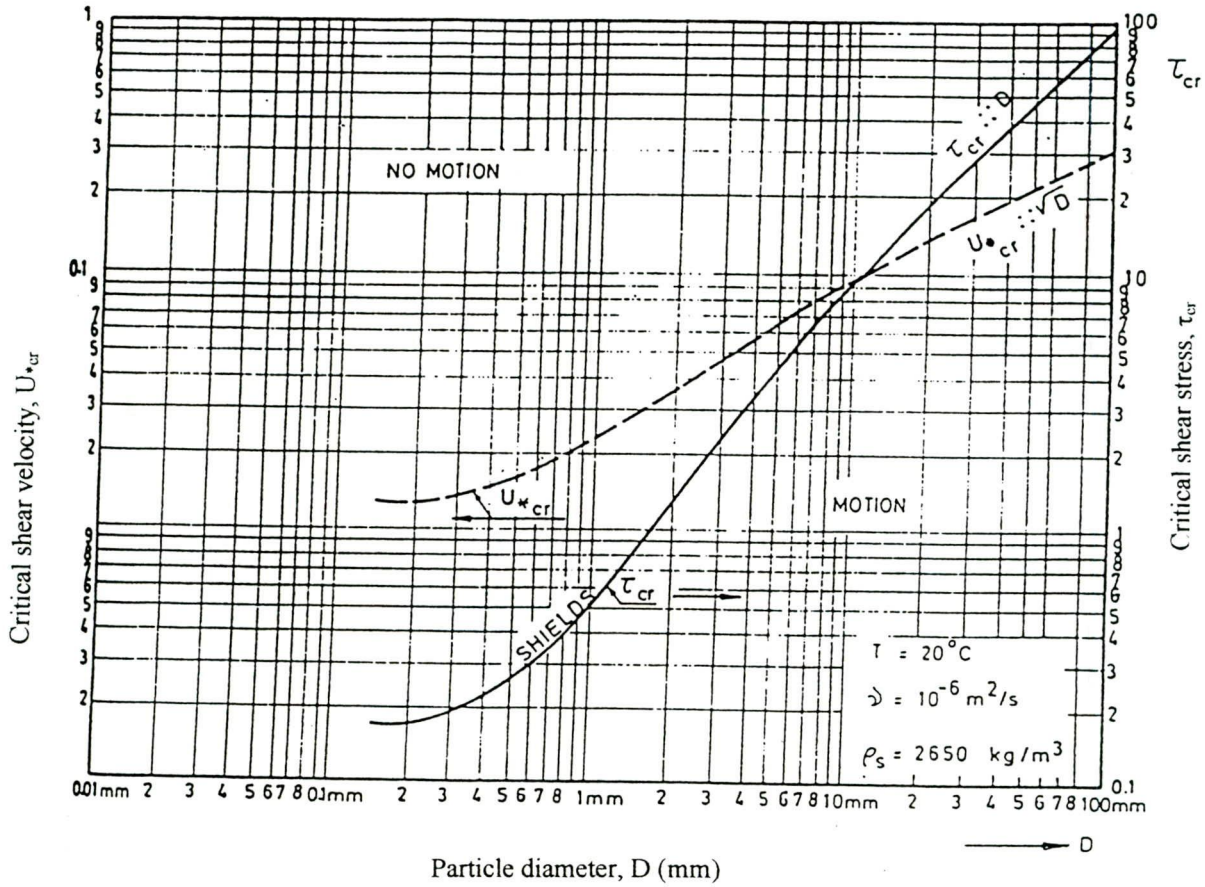

Fig. 2.1 Critical shear stress and critical shear velocity as a function of particle size, for $\rho_s = 2650$ k_g/m^3 (after Shields, 1936)

Figure 2.2 shows the critical bed shear stress according to Shields as a function of the particle diameter D for a temperature of T = 10^0, 20^0 and 30^0C. The influence of temperature is significant for a particle diameter smaller than 600 μm.

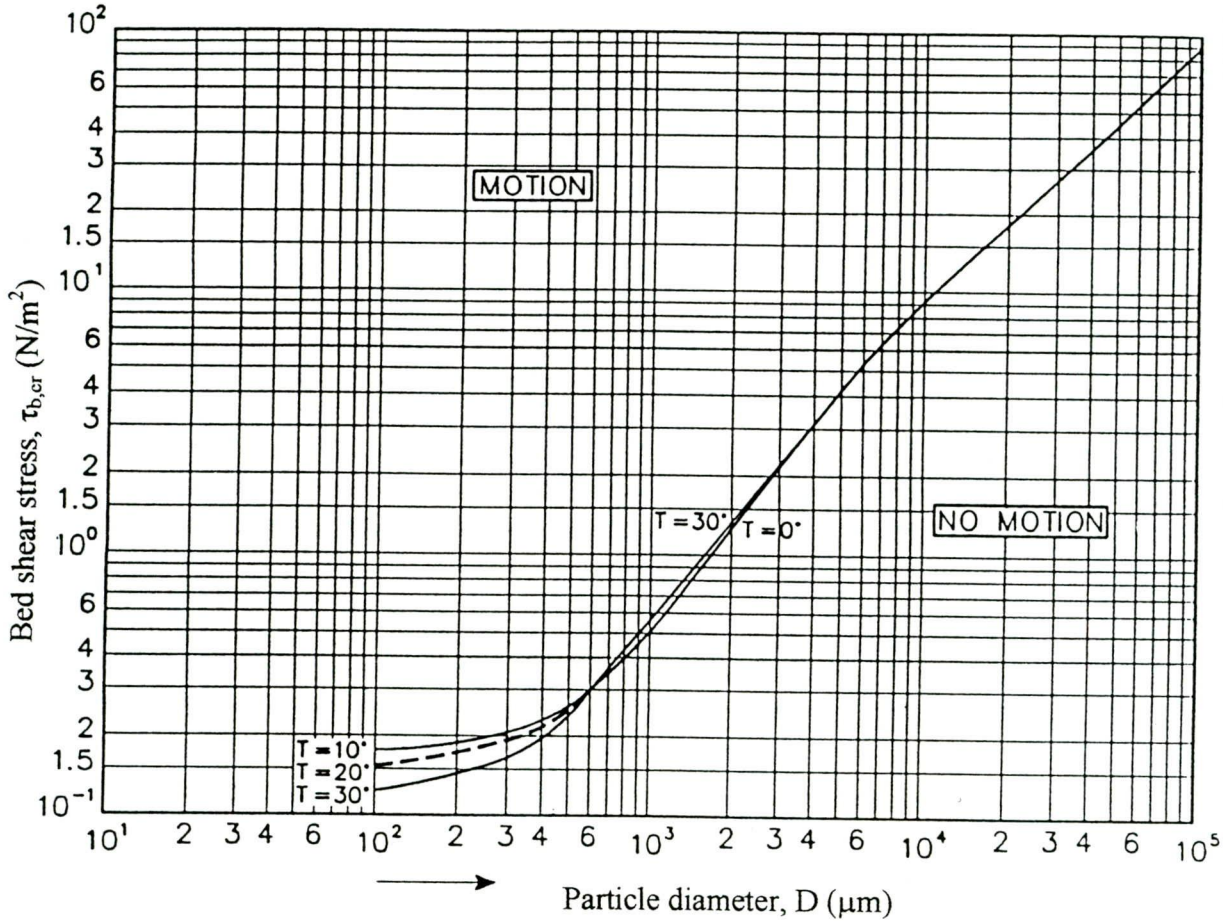

Fig.2.2 Initiation of motion for a current over a plane bed (after Shields, 1936)

2.6.1.2 Critical Shields value

Fig. 2.3 shows the functional relationship of the equation $\frac{\tau_c}{(\gamma_s - \gamma)d} = F(\frac{U_{*c}d)}{\nu})$ established on experimental data, obtained by Shields (1936) and other investigators. Where, the left hand side of this equation is the dimensionless critical shear stress and is often referred to as the **critical shield stress** τ_{*c}. The right hand side is called **boundary Reynolds number** and is noted by R_{e*}.

Mathematically, the Shields value (θ) is defined as

$$\theta = \frac{U_*^2}{g(s-1)D} \tag{2.24}$$

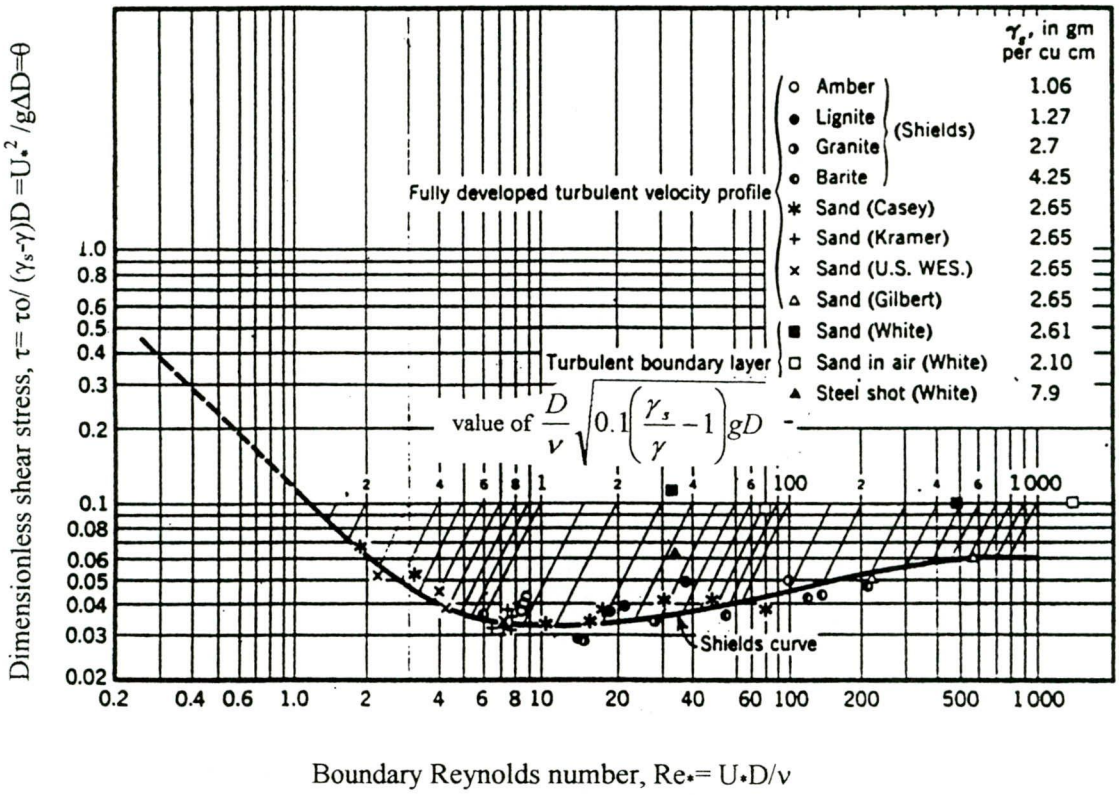

Fig.2.3 Shields diagram (after Prins, 1978)

The condition for the initiation of motion of a sediment particle on a horizontal bed is that the Shields value θ should be greater or at least equal to the critical Shields value θ_{cr}. The critical Shields value is a function of the boundary Reynolds number $Re_* = \frac{U_* D}{\nu}$, the median particle size D_{50} and the porosity of the deposited material. For a given sediment particle the critical Shields value can be written as

$$\theta_{cr} \leq \theta \tag{2.25}$$

in which

$$\theta_{cr} = f(Re_*) = f(\frac{U_* D}{\nu}) \tag{2.26}$$

Shields (1936) carried out experiments for determining the threshold values of sediment particles lying on a flat bed.

2.6.1.3 Critical shear velocity

The critical shear velocity, U_{*c}, is defined as $U_{*c} = \sqrt{\frac{\tau_{b,cr}}{\rho}}$ (2.27)

The threshold value according to the shear velocity is defined as follows:

i. In case the shear velocity, U_*, is smaller than the critical shear velocity, U_{*cr}, the sediment particle on the bed will be in equilibrium.

i. In case the shear velocity, U_*, is greater than or equal to the critical shear velocity, U_{*cr}, the sediment particle will be set in motion.

The critical shear velocity can be obtained from Fig. 2.1. For the particle diameter of non-uniform bed material, the critical shear velocity U_{*cr} can be obtained using median particle size D_{50} from Fig. 2.3.

2.7 Transport of Sediment

When the average shear stress on the bed of an alluvial channel exceeds the critical tractive stress for the bed material, the particles on the bed begin to move in the direction of the flow. The particles move in different ways depending on flow conditions, ratio of the densities of the fluid and the sediment and the size of the sediment. Above a certain flow velocity, individual particles start moving along the bed by rolling, sliding and making small jumps. The sediment transport can be classified according to origin and mechanism as in the Fig. 2.4.

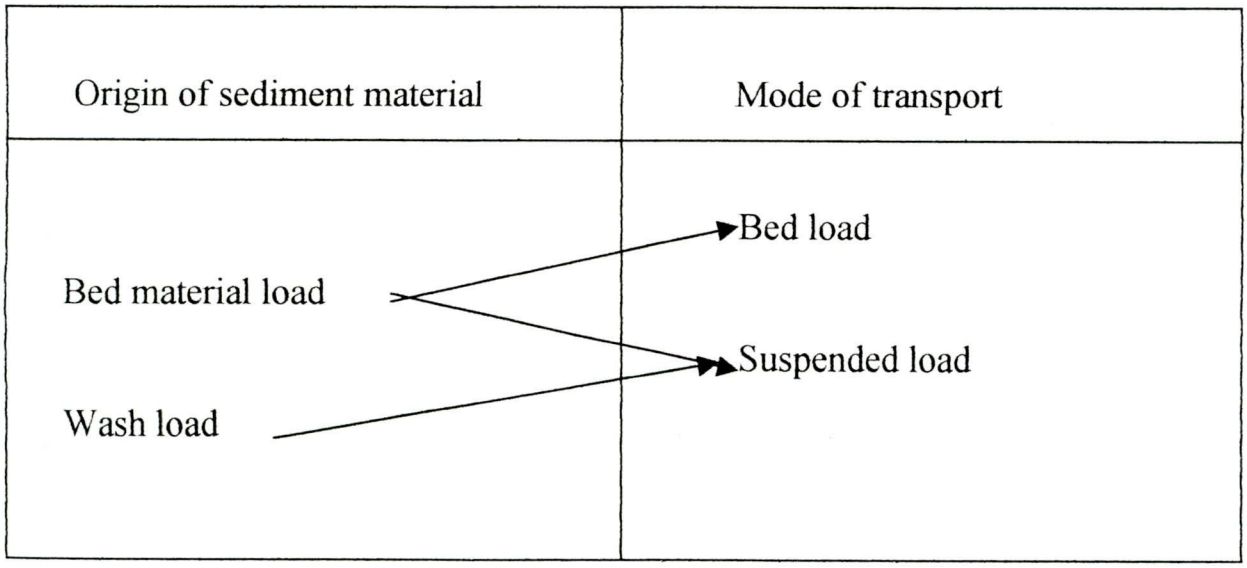

Fig.2.4 Origin and mode of sediment transport.

2.8 Factors Affecting Bed Forms and Resistance to Flow

The type of bed configuration and the dimensions of the bed forms are dependent on the properties of the flow, fluid and bed materials. But the large number of variables which affect bed forms are interdependent. Some of the variables change with flow conditions and alter their roles from dependent to independent variables. It is difficult, specially in field studies, to differentiate between the independent and dependent variables. According to Simons and Senturk (1977) the principle variables involved in the analysis of flow in alluvial channels are

$$f(U, d, S, \rho, \mu, g, D, \sigma, \rho_s, S_p, S_r, S_c, f_s) = 0 \tag{2.28}$$

where,

U	=	average velocity of flow
d	=	average depth
S	=	slope of energy grade line
ρ	=	density of water
μ	=	dynamic viscosity of water
g	=	acceleration due to gravity
D	=	diameter of the bed material
σ	=	size distribution of bed material
ρ_s	=	density of sediment
S_p	=	shape factor of the particles
S_r	=	shape factor for the reach of the stream
S_c	=	shape factor for the cross-section of the stream
f_s	=	seepage force in the bed of the stream.

In Eq. (2.28) fluid viscosity μ and particle shape factor S_p can be eliminated if the fall velocity of the bed material w_s is included in the list of the variables (Simons and Senturk, 1977)

$$U = F(d, S, \rho, \rho_s, g, D, \sigma, w_s, S_r, S_c, f_s) \tag{2.29}$$

Depth (d)

It is observed that, in flumes and natural canals in which slope and bed materials are essentially constant, the bed forms change with change in flow depth. An increase in depth,

without varying slope and bed material, may cause a dune bed to change to a plane bed or anti-une, and that a decrease in depth may reverse the process.

Slope (S)

At steep slopes there is an increase in the gravity component of the submerged weight of the sediment in the direction of flow. With increase in slope keeping depth of flow constant, the bed configuration changes. Resistance to flow is a function of slope even when the bed configuration does not change.

Density

The density of sediment-water mixture can be computed from the relationship (Simons et al., 1963).

$$\gamma_m = \frac{\gamma \gamma_s}{\gamma_s - C_s(\gamma_s - \gamma)} \tag{2.30}$$

where C_s is the concentration of the suspended sediment by weight expressed as a percentage. The increased density of the fluid is important in determining the submerged weight of sediment and hence the magnitude of the lift and drag forces.

Size of bed material (D)

The effects of physical size of the bed material on bed configurations and resistance to flow include: (i) its influence on the fall velocity, (ii) its effect on grain roughness, and (iii) its effect on the turbulent structure and the velocity field of flow.

Gradation of bed material (σ)

The gradation of the bed material has a significant effect on bed forms. Bed forms of uniform sand are more angular and have a larger resistance to flow than graded sand with identical mean fall diameter.

Fall velocity (w_s)

Fall velocity is the primary variable that determines the interactions between the bed material and

the fluid. For a given depth and slope, it significantly affects the bed form that will occur, the actual dimensions of the bed form, and expect for the contribution of the grain roughness, the resistance to flow. If w_s is a variable an increase in its value causes

i) an increase in the resistance for dune bed,
ii) a decrease in the resistance for ripple bed, and
iii) a change from ripples to dunes.

2.9 Alluvial Roughness

The roughness of an alluvial channel with movable bed and banks depends on

1. the roughness of the bed material or grain roughness, and
2. the hydraulic resistance induced by the shape of the bed form or form drag.

For the roughness of an alluvial channel, therefore, we can write

$$\tau_b = \tau'_b + \tau''_b \tag{2.31}$$

where, $\tau_b =$ overall bed shear stress

$\tau'_b =$ bed shear stress related to grain roughness, and

$\tau''_b =$ bed shear stress related to form drag.

where, τ_b = overall bed shear stress

τ'_b = bed shear stress related to grain roughness, and

τ''_b = bed shear stress related to form drag.

2.9.1 Grain roughness related bed shear stress (τ'_b)

The grain shear stress varies with position on the bed form surface, increasing from zero value at the flow reattachment point to a maximum value at the crest. The grain roughness related bed shear stress results from the skin friction exerted between the flow and each individual sediment particle. Division of the horizontal component of the total skin friction force ($F_{s,h}$) by the flow width and bed form length yields the grain force related bed shear stress acting against the flow (Fig. 2.5). It is to be mentioned that in case of a plane bed the skin friction force acts on the entire length of the bed, while in case of a dune the skin force acts on the luff side of a dune only.

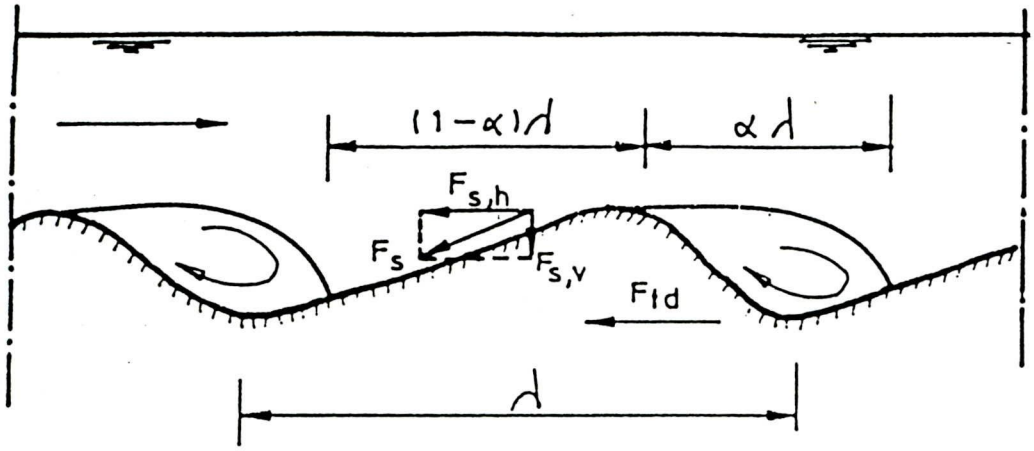

Fig.2.5 Forces acting on a fluid around a bed form (after Raudkivi, 1990)

2.9.2 Form drag related bed-shear stress (τ''_b)

The form drag related bed-shear stress results from pressure differences over the bed form. The flow is accelerated on the luff side, while it is decelerated on the lee side of the bed form. As a result the pressure on the luff side will be smaller than on the lee side. Integration of pressure over the length of the bed form results in horizontal force (F_{fd}) acting against the flow (Fig. 2.5). Dividing this force F_{fd} by the flow width and the bed form length yields the form drag related bed shear stress. The form drag related bed shear stress and hence the total bed shear stress changes with changes of bed form from plane bed to ripples, dunes, washed out dunes, a flat bed, anti-dunes and chute and pools. In Fig. 2.6 the variation of the bed shear stress τ_b and the Darcy-Weisbach friction factor f with the depth average velocity is shown. In this figure it can be seen that the influence of the form drag related bed shear stress is only important for ripples, dunes and anti-dunes. In case of immobile bed and a transition flat bed, the influence of the form drag related bed shear stress can be neglected.

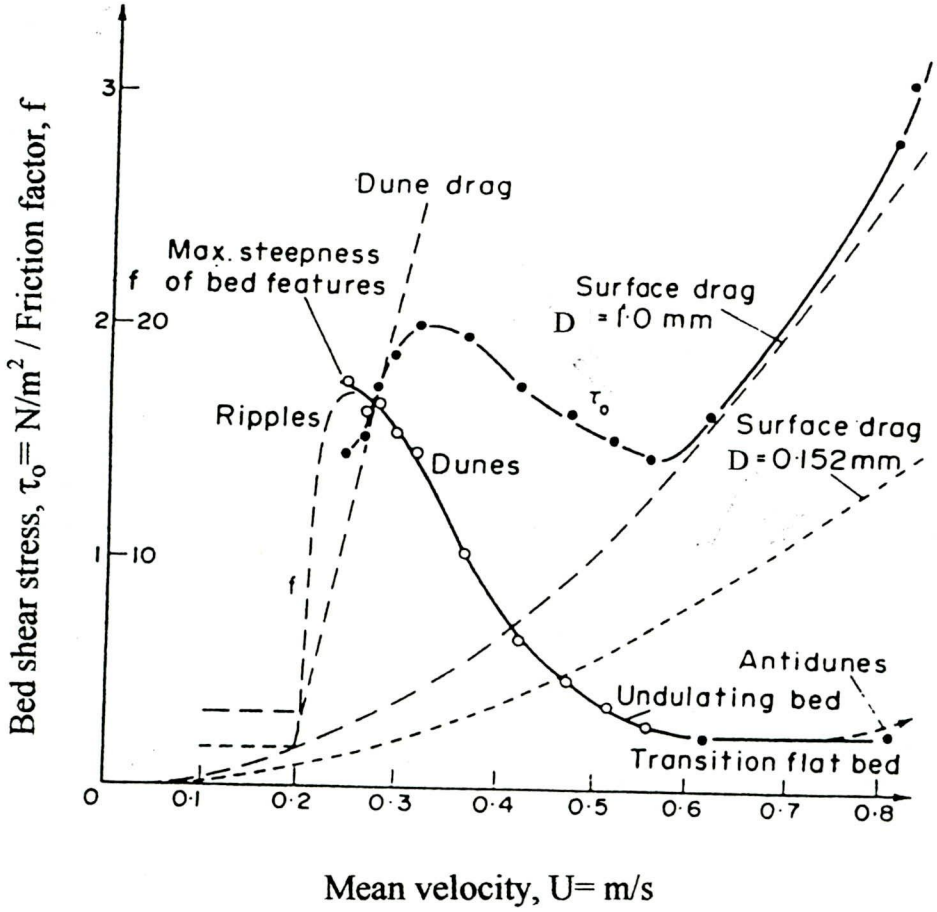

Fig.2.6 Variation of bed shear stress τ_0 and Darcy-Weisbach friction Factor f with mean velocity U in flow over a fine sand bed (after Raudkivi, 1990)

2.10 Parameters Describing Surface Drag and Form Drag

Many studies and analyses have considered various ways of expressing form and surface drags. Two general approaches are

1. If S′ corresponds to the fraction of the energy slope required to overcome surface drag and S″ the fraction required to offset form drag it is possible to write (Mayer-Peter and Muller, 1948)

   $$S = S' + S'' \tag{2.32}$$

2. Einstein and Barbarossa (1952) divided the total resistance of the bed into resistance due to sand grains and resistance due to the bed formations, e.g.

   $$\tau_b = \tau_b' + \tau_b''$$

or

$$\gamma RS = \gamma R'S + \gamma R''S$$

or

$$R = R' + R'' \tag{2.33}$$

2.11 Parameters for Describing Resistance to Flow

Friction velocity U_*

We know that total bed shear stress is the sum of bed shear stress related to grain roughness and bed shear stress related to grain shear. It is assumed that τ_b, τ'_b and τ''_b are caused by the same flow velocity U. Thus also

$$\tau_b = \tau'_b + \tau''_b$$

or

$$U_*^2 \rho = U_*'^2 \rho + U_*''^2 \rho \qquad \left[\because U_* = \sqrt{\frac{\tau_b}{\rho}}\right] \tag{2.34}$$

$$U_*^2 = U_*'^2 + U_*''^2$$

or

$$RS = (RS)' + (RS)'' \qquad \left[\because U_* = C\sqrt{RS}\right] \tag{2.35}$$

Chezy's resistance factor C

The Chezy's formula expressed in Eq. 2.4 can be rearranged as

$$\frac{U^2}{C^2} = RS = (RS)' + (RS)'' \tag{2.36}$$

or

$$\frac{1}{C^2} = \frac{(RS)'}{U^2} + \frac{(RS)''}{U^2} = (\frac{1}{C'})^2 + (\frac{1}{C''})^2 \tag{2.37}$$

Darcy-Wiesbach friction factor f

Taylor and Brooks (1962) suggested that the Darcy-Weisbach friction factor f be divided into two parts, that due to grain roughness

$$f' = \frac{8gR'S}{U^2} \tag{2.38}$$

and that due to form roughness

$$f'' = \frac{8gR''S}{U^2} \tag{2.39}$$

In this case, the total friction factor

$$f = f' + f'' \tag{2.40}$$

Manning's roughness coefficient n

According to the Bajorunas (1952), the Manning's roughness coefficient n is composed of two parts and can be determined by the relationship

$$n = n' + n'' \tag{2.41}$$

where, n' is due to the grain roughness and n'' is due to form roughness. The value of n' is obtained as

$$n' = \frac{n}{(R/R')^{2/3}} \tag{2.42}$$

2.12 Bed Forms

The phenomenon of bed forms in alluvial rivers was perhaps first described in the classical research of Gilbert (1914). The bed feature has fascinated engineers and scientists because of its association

with so many aspects of river sedimentation and river morphology. The bed forms are flow-induced and directly affect the roughness or flow resistance. Different types of bed forms are developed in the alluvial rivers with increasing discharge and sediment load. So the nature of bed forms is of critical importance for the analysis of resistance factors. When the bed forms crest are perpendicular to the main flow direction, the bed forms are called transverse bed forms, such as ripples, dunes and anti-dunes. Bed forms with their crest parallel to the flow are called longitudinal bed forms such as ribbons and ridges. The morphological regimes for unidirectional currents over a sand bed ($D_{50} \leq$ 2000 μm) can be classified into

- lower transport regime with flat bed, ribbons and ridges, ripples, dunes and bars,
- transition regime with washed out dunes and sand waves, and
- upper transition regime with flat mobile bed and sand waves.

In literature, considering hydraulic point of view, roughly four groups of classification methods are available. Engelund (1967) used Froude number as a classification parameter, while Liu (1957) described the type of bed forms in terms of a suspension parameter and a particle-related Reynolds number. Simons and Richardson (1966) used the stream power and the median fall diameter as basic parameters. They expressed bed forms types as a function of τ_b, U and D. Van Rijn (1993) used a dimensionless bed shear stress parameter and a dimensionless particle parameter to classify the bed forms types. Van Rijn bed forms types are function of [T, D_*] in which

T = transport stage parameter

D_* = dimensionless particle diameter

The classification of bed forms according to Simons (1966) is given in the Table 2.1. and the corresponding bed forms types by Simons and Richardson (1966) are depicted in Fig. 2.7.

Table 2.1 Classification of bed forms (after Simons, 1966 and addition by Prins, 1978)

Flow regime	Bed form	Bed material concentration	Mode of sediment transport	Type of roughness	Roughness $C/\sqrt{g}$
Lower regime	1. ripples 2. ripple on dunes 3. dunes	10-200 ppm 100-1200 ppm 200-2000 ppm	discrete steps	Form roughness predominates	7.8-12.4 7.0-13.2
Transition	1. washed out dunes	1000-3000 ppm	-	variable	7.0-20.0
Upper regime	1. plane beds 2. antidunes 3. chutes and pools	2000-6000 ppm >2000 ppm >2000 ppm	continuous	Grain roughness predominates	16.3-20.0 10.8-20.0 9.4-10.7

Van Rijn (1984, 1989) used flume and field data to develop his classification. Van Rijn defined the following two parameters:

$$D_* = D_{50}\left[\frac{(s-1)g}{\nu^2}\right]^{1/3} \tag{2.43}$$

$$T = \frac{(U_*)^2}{(U_{*cr})^2} - 1 \tag{2.44}$$

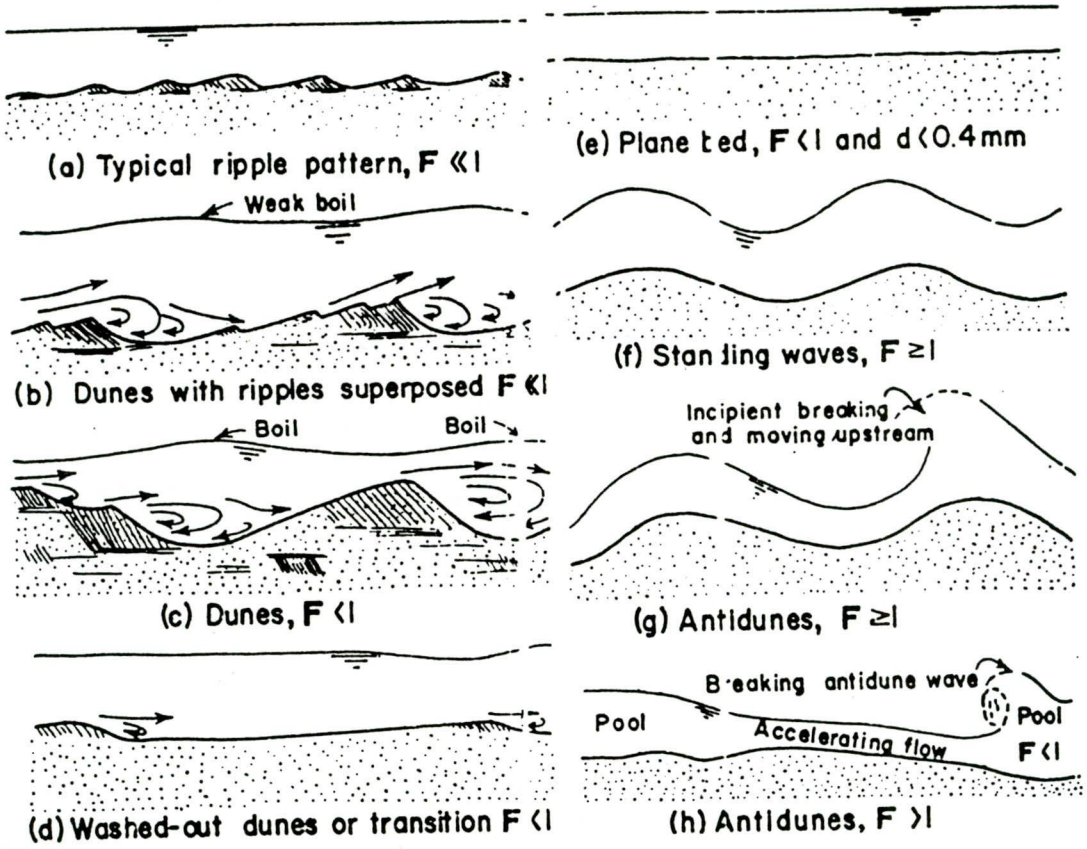

Fig.2.7 Bed form types in rivers according to Simons and Richardson (1966)

According to Van Rijn the classification of bed forms are summarised in Table 2.2. and Fig.2.8.

Table 2.2 Classification of bed forms (after Van Rijn, 1993)

Transport regime		Particle size	
		$1 \le D_* < 10$	$D_* > 10$
Lower	$0 \le T \le 3$	mini-ripples	Dunes
	$3 < T \le 10$	mega-ripples and dunes	Dunes
	$10 < T \le 15$	dunes	Dunes
Transition	$15 < T < 25$	wash out dunes, sand waves	
Upper	$T \ge 25, Fr < 0.8$	(symmetrical) sand waves	
	$T \ge 25, Fr \ge 0.8$	plane bed and/or anti-dunes	

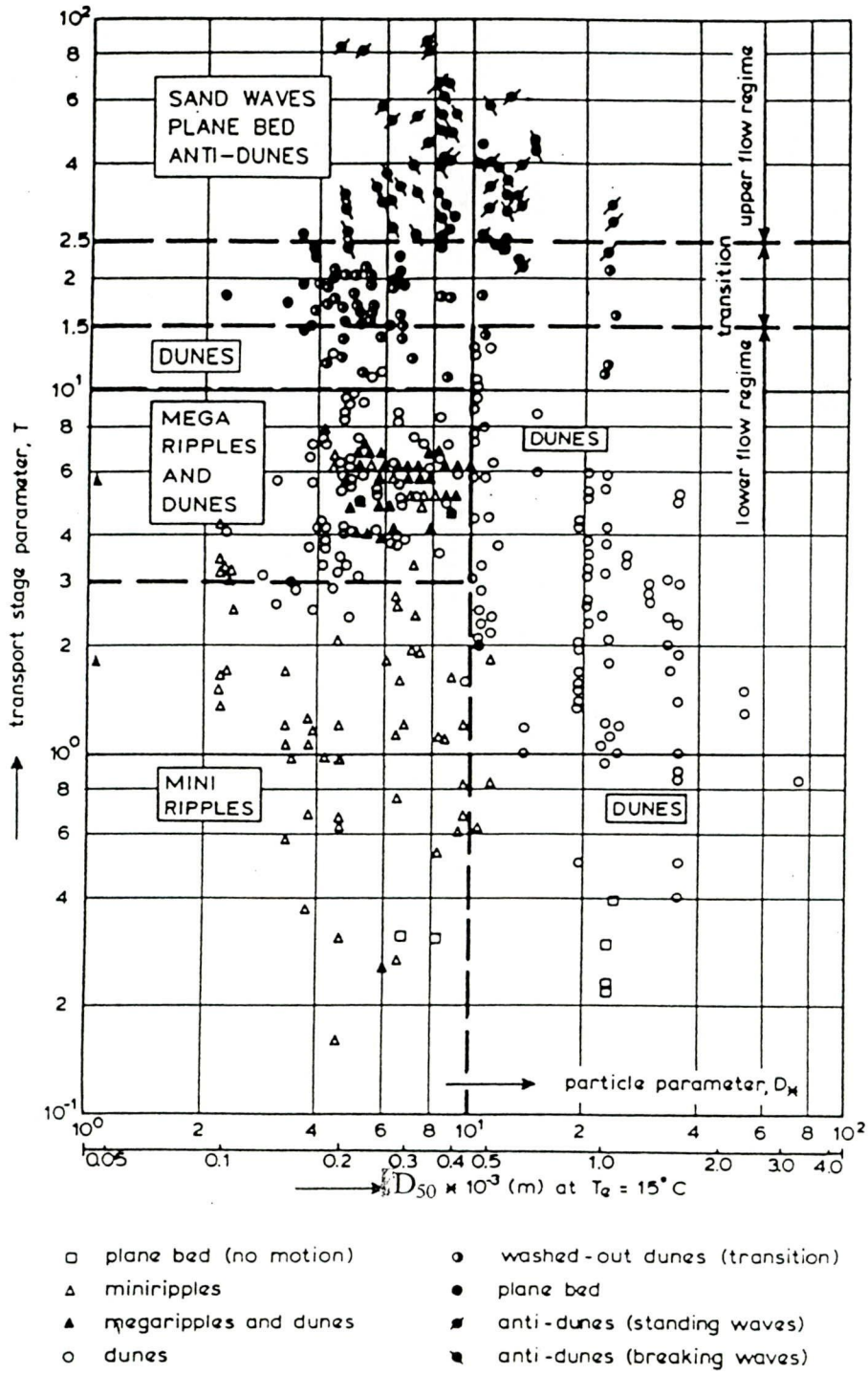

Fig.2.8 Bed form classification diagram for currents, Van Rijn (1984, 1989)

Yalin (1992) classified bed forms as follows:

1. **Ripples:** Ripples are defined as the bed forms "imprinted" by the viscous flow structures (undulations) at the bed of turbulent flow. When ripples are forming, the bed roughness changes and the flow at the bed become saturated with transported grains. The length of the ripple is independent of flow depth.

2. **Dunes and bars:** Dunes and bars are due to the quasi-uniformities caused by the structure of turbulence. The length of the turbulent structure is proportional to one of the characteristic lengths d, B or $\frac{\nu}{U_*}$. Dunes and bars are defined as those bed forms which are produced by vertical bursts and horizontal bursts, respectively.

Turbulence is due to multitude of eddies having various sizes. These eddies are superimposed on each other and, ultimately, on the mean flow. Eddies are not permanent - they are born, they live and they die. In any turbulent flow, the largest eddies play an important part. Larger eddies are generated near the flow boundaries, that is, near the bed in case of an open channel flow. Immediately after its birth, the prospective eddy is ejected upwards and, as it moves away from the bed, it is conveyed downstream by the flow. During this motion, the upward velocity of the eddy progressively decreases but its size increases. Eventually the size of the eddy becomes a micro-turbulent eddy. However, soon after that the eddy hits the bed and is destroyed. The destruction of the eddy prompts the generation of a new (second) eddy. Having passed through the same stages, this second eddy causes, in turn, the generation of the third eddy, and so on. Bursting process, or simply burst, is the term which stands for the above outlined sequence of events, which occurs between the births of two consecutive eddies. The vertical turbulence is formed by the eddies whose axes of rotation are perpendicular to the vertical planes. Yet an open channel flow has turbulent eddies also in plan, whose axes are perpendicular to the horizontal plane, form horizontal turbulence.

3. **Anti-dunes:** If Froude number is greater than 0.60, then standing waves or anti-dunes occur. The relative height of standing waves increases with Froude number. When flow is supercritical they are usually able to deform the initially flat surface of a mobile bed as to produce bed forms: anti-dunes.

Since the bed forms are finite, a bed form exists if it has a non-zero height. This is the case when $\eta = \left(\frac{U_*}{U_{*cr}}\right)^2 > 1$. However, the existence of a bed form depends on the values of the following parameters:

$$\frac{B}{d}, Z = \frac{d}{D_{50}}, X = \frac{D_{50}U_*}{\nu} \text{ and } Fr = \frac{U}{\sqrt{gh}}$$

The value of the Froude number Fr mainly indicates whether the bed forms whose lengths are proportional to depth d are dunes or anti-dunes. Therefore, attention was focused on the remaining other three variables of the above. Based on X, Yalin classified regimes as smooth, transitional and rough.

The development of larger bed forms (dunes and bars) takes place past the bed covered by ripple. When X≤2.5, then it is mainly ripple which is present on the bed surface. Earlier developed ripples protect the bed against any further deformation. In the transitional regime when 2.5≤X≤35 ripples are superimposed on larger bed forms. With the increment of X the prominence of ripples decreases and that of larger bed form increases. When X≥35 in rough turbulent flow, ripples are no longer present and the larger bed forms have their maximum prominence.

2.13 Dimensions of Bed Forms

A study of the geometric properties of sand waves is important for three reasons: (i) to understand how the bed forms are related to the characteristics of fluid flow and bed material in an alluvial channel, (ii) to understand how these geometric elements affect the resistance to flow and sediment transport in an alluvial channel, and (iii) to get an insight into the dynamic forces of a stream responsible for transport and deposition of ancient deposits from a knowledge of relationship between the geometric properties of sand waves and flow variables. Some of the methods for predicting bed form dimensions are as follows:

2.13.1 Allen (1963) method

Allen (1963) has shown by physical reasoning and analysis of observed data the following relationship:

For dune and ripple bed

$$\text{Log } d = 0.8271 \text{ Log } H + 0.8901 \tag{2.45}$$

in which

d = depth of water

H = bed form height

For ripple bed (H < 0.15m)

$$\text{Log } H = 0.9508 \text{ Log } L - 1.0867 \tag{2.46}$$

For dune bed

$$\text{Log } H = 0.7384 \text{ Log } L - 1.0746 \tag{2.47}$$

in which

L = bed form length

2.13.2 Senturk (1967) method

From dimensional analysis and eliminating parameters of secondary importance, the following is the functional equation for bed form type:

$$\text{Bed form type} = f\left(S, \frac{R}{D}, Fr, Fr_g, \frac{U_*}{w_s}, R_w\right) \tag{2.48}$$

where,

Fr_g = Froude number related to grain size $= \dfrac{\rho U_*^2}{(\rho_s - \rho) g D_{50}}$

R_w = Reynolds number related to the fall velocity $= \dfrac{D_{65} w_{65}}{\nu}$

Different phases of bed form predicted by Senturk are summarized as follows:

(i) Ripples form for $U > U_{BM} > U_B$

where, U_{BM} = velocity corresponding to the beginning of motion

$$U_B = 9.57\,(gSR_B'')^{1/2} \tag{2.49}$$

$$R''_B = \text{Exp}_{10}(3.4936 - 0.091 \text{ Log } C + \text{Log } D_{65}) \tag{2.50}$$

$$C = \frac{D_{35}}{D_{65}}(s - 1)\frac{R_w^2}{S} \tag{2.51}$$

(ii) Transition from plane bed to ripple occurs for $U<U_{BM}<U_X$

where,

$$U_x = (21.6 - 0.373 \text{ Log } C)\,(gSR_x)^{1/2} \tag{2.52}$$

$$R_x = \text{Exp}_{10}(2.4 - 0.057 \text{ Log } C + \text{Log } D_{65}) \tag{2.53}$$

(iii) The transition from dunes to plane bed with sediment transport occurs for $U>U_B>U_{BM}$ but $U<U_{PL}$

where,

$$2a(d + 6.5 \text{ Log } R')\sqrt{R'} = U_B - \frac{\sqrt{R'}(R' - R_B'')}{a(e + 6.5 \text{ Log } R')} \tag{2.54}$$

$$2a = \sqrt{gS} \tag{2.55}$$

$$d = 6 + 6.5 \text{ Log}(\frac{1}{D_{65}}) \tag{2.56}$$

$$e = 11.65 + 6.5 \text{ Log}(\frac{1}{D_{65}}) \tag{2.57}$$

$$U_{PL} = (6 + 6.5 \text{ Log } \frac{R'}{D_{65}})(gR'S)^{1/2} \tag{2.58}$$

(iv) Plane bed with sediment movement occurs for $U>U_{PL}>U_B>U_{BM}$

(v) The transition from ripples to dune bed occurs for $U_N<U<U_M$

where,

$$2a(c' + 11 \text{ Log } R'')\sqrt{R''} = U_x - \frac{\sqrt{R''}(R'' - R_R'')}{a(b - 11 \text{ Log } R'')} \tag{2.59}$$

$$b = 48 - \text{Log } C - 11 \text{Log}(\frac{1}{D_{65}}) - 9.57 \tag{2.60}$$

$$c' = b + 9.57 \tag{2.61}$$

The R_R'' value is obtained from the equation

$$U_x = (-52 + 2.5 \text{ Log } C + 18 \text{ Log } \frac{R_R''}{D_{65}})(gSR_R'')^{1/2} \tag{2.62}$$

To compute U_N the following equation may be solved for R'':

$$U_x + \frac{\sqrt{R_x}(R_x - R'')}{a(b - 11 \text{ Log } R_x)} = (-52 + 2.5 \text{ Log } C + 18 \text{ Log } \frac{R''}{D_{65}})(gSR'')^{1/2} \tag{2.63}$$

$$U_N = (-52 + 2.5 \text{ Log } C + 18 \text{ Log } \frac{R''}{D_{65}})(gSR'')^{1/2} \tag{2.64}$$

(vi) Well-formed ripples develop for $U_{BM} < U < U_N$

(vii) Well-formed dunes develop for $U_M < U < U_B$.

2.13.3 Vajda (1990) method

Vajda (1990) made the predictive evaluation of bed forms characteristics in alluvial rivers of cohesionless bed. The prediction model is limited to subcritical flow. It can be applied locally in order to predict bed forms in an irregular cross-section. The most important aspect of this prediction model is the use of analytical equation for determining dimensionless critical bed shear, also called critical mobility number, instead of using Shields diagram or any modified Shields diagram.

The complexity of the alluvial roughness pattern makes a purely analytical approach to the corresponding resistance problem impractical. It is assumed, in general, that alluvial bed forms attain equilibrium state after a long period of superimposed quasi-steady flow conditions. The bed forms types and dimensions can in the subsequent equilibrium process be related empirically to the total equivalent bed shear $\tau_b = \rho U_*^2$. The dimensionless parameters to which observed equilibrium bed forms geometry usually related are

- the dimensionless grain parameter

$$D_* = \left[\frac{(\rho_s - \rho)}{\rho} g / \nu^2\right]^{1/3} D_{50} \tag{2.65}$$

- the dimensionless bed shear

$$\tau_* = \frac{U_*^2}{\frac{(\rho_s - \rho)}{\rho} g D_{50}} \tag{2.66}$$

or, rather, its excess shear version

$$T_* = \frac{\tau_*}{\tau_{*c}} - 1 \tag{2.67}$$

where, τ_{*c} is the critical value of τ_*

- the relative depth $\eta = \dfrac{d}{D_{50}}$

The critical mobility number is originally related to grain Reynolds number $Re_* = \dfrac{U_* D_{50}}{\nu}$.

However, such a representation makes the excess shear T_* pertaining to a particular D_{50} difficult.

It can be made direct only in the modified Shields diagram (Fig. 2.9), in which the abscissa depends on D_* only. The analytical form of critical mobility curve can be written in the form

$$\tau_{*c} = \frac{0.2}{D_*} + 0.056 \tanh\left(\frac{D_*^{0.77}}{25}\right) \tag{2.68}$$

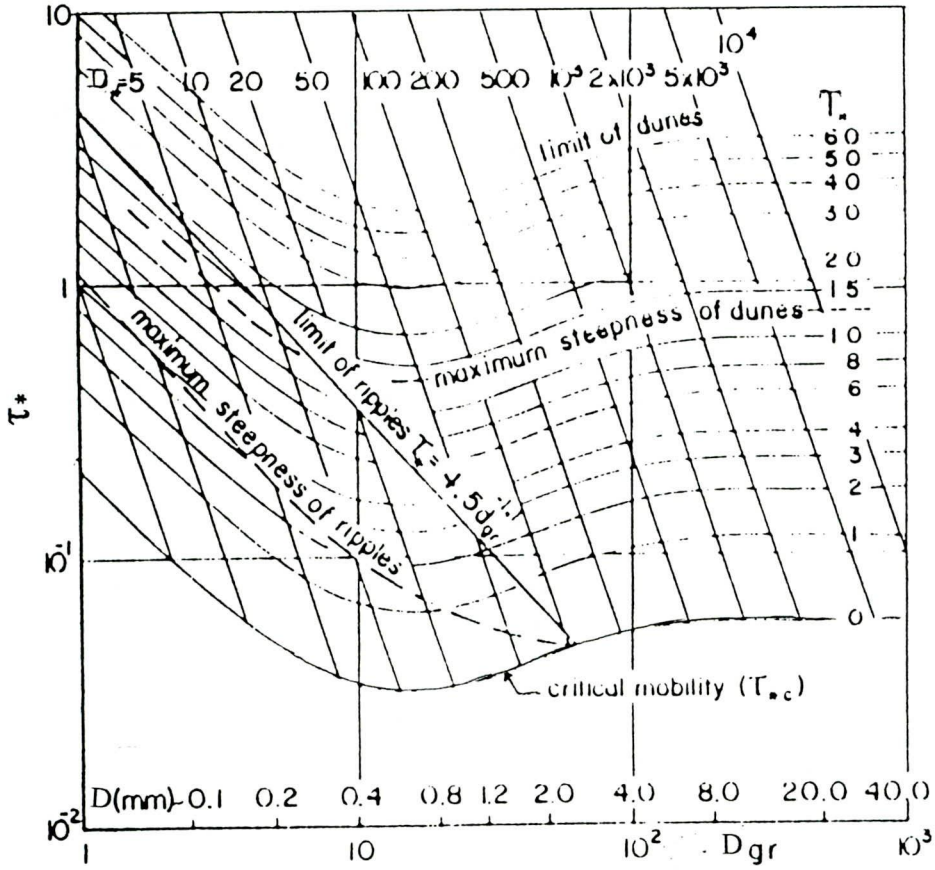

Fig.2.9 **Modified Shields diagram (after Vajda, 1990)**

The sand ripples, on the other hand, are found to be independent from its relative depth, because their height is usually much smaller than the flow depth, but they strongly depends on D_*. According to Vajda(1990), sand ripples could actually be regarded as roughness the dimensions of which are variable with flow conditions. In view of this variability, the location of the nominal bed cannot be identified in alluvial conditions, rather with the mean bed elevation. It is similarly assumed that all the parameters can be related to a vertically averaged mean velocity pertaining to that bed level.

With the above assumptions, the tentative correlation functions of the equilibrium bed-form geometry can be written as follows:

Dune geometry

$$L = 7R \tag{2.69}$$

$$\frac{H}{L} = \frac{T_*}{D_{80}} \exp\left[\frac{-T_*/13}{1+(\eta+169)[1-\coth(\eta/169)]}\right] \tag{2.70}$$

Ripple geometry

$$L = \frac{2300\, D_{50}[1+0.01\exp(2.3\, D_*^{0.75}\tau_*^{0.5})]}{D_*^{1.5}\tau_*^{0.5}} \tag{2.71}$$

$$\frac{H}{L} = \frac{0.15\, T_*(D_*\tau_*)^{0.5}\exp(-T_*/3)}{[1+0.01\exp(2.3\, D_*^{0.75}\tau_*^{0.5})]} \tag{2.72}$$

2.13.4 Yalin (1992) method:

Yalin(1992) classified dimensions of bed forms as follows:

Dune length and height

The dune length L_d is given by

$$L_d = \varphi_d\,(X, Z)\ 6d \tag{2.73}$$

where,

$$Z = \frac{d}{D_{50}} \text{ and } X = \frac{D_{50}U_*}{\nu}$$

If the flow past the flat initial bed is rough turbulent, then $F_d\,(X,Z) \approx 1$. Yalin used the following component-equation. for the evaluation of $F_d\,(X,Z)$:

$$\Phi_d(X,Z) = 1 + 0.01\frac{(Z-40)(Z-400)}{Z}e^{-m} \tag{2.74}$$

in which $m = 0.055\sqrt{Z} + 0.04X$.

The dune steepness $(\frac{H_d}{L_d})$ varies as a function of the dimensionless flow intensity h, and its value decreases with the increment of relative skin roughness (Z). Furthermore, the value of dune steepness must be affected by the grain size Reynolds number X. Hence, in general, the dune steepness must be a function of h, Z and X. Hence the dune steepness can be given by the form $\delta_d = y_d\,(X, h, Z)$.

Yalin expressed dune steepness as

$$\delta_d = \psi_{xd}(X)\ \psi_d(\eta,Z) = (\delta_d)_{max}\ f_d(z),\ \psi_{xd}(X) \tag{2.75}$$

where,

$$(\delta_d)_{max} = 0.06\ (1 - e^{-0.008Z})$$

$$f_d(\zeta) = \zeta e^{1-z}$$

$$\zeta = \frac{\eta - 1}{\bar{\eta}_d - 1}$$

$$\bar{n}_d = 14(1 - e^{-0.003Z}) + 2$$

and identify $\psi_{xd}(X)$ with a non-decreasing continuous function which is zero when X = 0 and which is nearly unity when X≥35. The component equation

$$\psi_{xd}(X) = 1 - e^{-(x/10)2} \tag{2.76}$$

is such a function. The analogous decrement of the ripple steepness d_r with the increasing X can be reflected by

$$\psi_{Xr}(X) = 1 - e^{((X-2.5)/14)^2}\ \ if\ X > 2.5 \tag{2.77}$$

Ripples

Ripple length can be expressed functionally as

$$L_r = \phi_r(X,\xi)\, D_{50} \tag{2.78}$$

where,

$$\xi = \left(\frac{\gamma_s D_{50}^3}{\rho v^2}\right)^{1/3}$$

Yalin expressed ripple length L_r as

$$\frac{L_r}{D_{50}} = \phi_r(X,\xi) = \frac{(L_r)_{min}}{D_{50}}[4P(1-P)^{-1}] \tag{2.79}$$

where,

$$\frac{(L_r)_{min}}{D} = \frac{2650}{\xi^{0.88}}\ \ and\ P = \sqrt{\frac{\eta}{21}}$$

The geometry of ripples does not depend on the flow depth. Hence Z can not be a variable in the expression of ripple steepness. Ripple steepness can be expressed functionally as

$\delta_r = \psi_{Xr}(x)\psi_r(\eta)$ Yalin experimentally found that

$$(\delta_r)_{max} \approx 0.14, \overline{\eta_r} - 1 \approx 10 \text{ and } (\eta_r)_{max} - 1 \approx 20$$

where $\overline{\eta_r}$ and $(h_r)_{max}$ are h-values which corresponds to $d_r = (d_r)_{max}$ and $d_r = 0$ (at advanced stages) respectively. Introducing normalized coordinates $\eta_r = \frac{\delta_r}{(\delta_r)_{max}}$ and $\zeta' = \frac{\eta - 1}{\overline{\eta_r} - 1}$, the normalized ripple function $\eta_r = f_r(\zeta)$

Yalin adopted

$$f_r(\zeta') = r_{fd}(\zeta) = r\zeta' e^{1-\zeta'} \tag{2.78}$$

where correction factor r can be given by

$$r \equiv 1 \text{ if } \zeta' \leq 1; r \equiv \zeta(z - \zeta') \text{ if } 1 < \zeta' < Z$$

The ripple steepness ultimately becomes

$$\frac{H_r}{L_r} = \delta_r = \psi_{Xr}(X) 0.14 f_r(\zeta') \tag{2.79}$$

2.13.5 Van Rijn (1993) method

Van Rijn (1993) proposed a method to classify bed forms. Bed forms dimensions according to Van Rijn (1993) are as follows:

1. Ripples, lower regime

When the velocities are somewhat higher (10-20%) than the critical velocity for initiation of motion and the median particle size is smaller than 500 μm, mini-ripples are generated at the bed surface. Ripples that are developed during this stage remain small with ripple length much smaller than the water depth. Yalin (1985) reports values in the range of

$$H_r = 50 \text{ to } 200\ D_{50} \tag{2.82}$$

$$L_r = 500 \text{ to } 1000\ D_{50} \tag{2.83}$$

As the velocities near the bed become larger, the ripples become more irregular in shape, height and spacing. In that case the variance of the ripple length and height becomes rather larger. The largest ripples may have a length upto the water depth and are commonly called as mega-ripples. It is assumed that the ripples will completely disappear for T≈10. The expressions for the relative height and length of mega-ripples are given by

$$\frac{H_r}{d} = 0.02(1 - e^{-0.1T})(10 - T) \text{ for } 1 \le D_* \le 10 \text{ and } 3 \le T \le 10 \tag{2.84}$$

$$L_r = 0.5d = 0.5R \tag{2.85}$$

2. Dunes, lower regime

Dune-type is another typical type of bed forms of the lower flow regime. Dunes have an isometrical profile with a rather steep lee-side and a gentle luff side. The length of the dunes is strongly related to the water depth with values in the range of 3d to 15d. Based on the analysis of flume and field data, Van Rijn (1982, 1984) proposed the following relationship for the dune height and length:

$$\frac{H_d}{d_d} = 0.11\left(\frac{D_{50}}{d}\right)^{0.3}(1 - e^{-0.5T})(25 - T) \tag{2.86}$$

$$L_d = 7.3d \tag{2.87}$$

Where, d = water depth from water surface to mean-bed level.

The dune height increases weakly with particle diameter for a given water depth. The dunes are assumed to be washed out for $T \ge 25$.

3. Washed-out dunes and sand waves, transition region

The bed forms generated at low velocities are washed out at high velocities. In the transition regime the sediment particles are transported mainly in suspension. This has a strong effect on the bed form shape. The bed forms in the transition regime which is most likely to occur, are washed out dunes and sand waves. Both types occur simultaneously. The dimensions of washed-out dunes are same as the dimensions of dunes. These bed forms fully disappear for T=25.

The dimensions of the symmetrical sand waves are described by

$$\frac{H_{sw}}{d} = 0.15(1 - e^{0.05(T-15)})(1 - Fr^2) \text{ for } T \ge 15 \tag{2.88}$$

$$L_{sw} = 10d \qquad \text{for } T \ge 15 \tag{2.89}$$

4. Plane bed and sand waves, upper regime

Two sub-regimes can be distinguished in the upper transport regime:

Subcritical upper transition regime : $T \ge 25$ and $Fr \le 0.8$

Supercritical upper transition regime : $T \ge 25$ and $Fr \ge 0.8$

In the subcritical upper regime the bed form types will be symmetrical sand waves with dimensions mentioned above. In the supercritical upper regime the bed form types will be plane bed and/or anti-dunes. Anti-dune bed forms are sand waves with a nearly symmetrical shape in phase with the water surface waves. Anti-dunes do not exist as a continuous train of bed waves. Anti-dunes are bed forms with a length scale of about 10 times the water depth.

2.14 Discussion on Alluvial Roughness Predictors

Resistance to flow in alluvial channels depends on the geometry of the bed patterns that are formed by the flow, the properties of the sediment, and the transport rate of the sediment. But the bed patterns are the main contributor of the flow resistance. The problem of resistance to flow over alluvial beds has received considerable attention during the 4th and 5th decades of this century. Attempts to determine resistance for alluvial channels have been based on modifications of fixed bed equations.

It has already been previously discussed that the roughness predictors of an alluvial channel in principle are of two kinds, viz.:

(i) Integral methods in which simultaneously the continuity and the Chezy or Manning equations are solved and the total roughness is found. These methods are based on overall flow parameters such as depth of water and depth-averaged flow velocity. Another integral method of predicting alluvial roughness is utilizing the principles of dimensional analysis.

(ii) Separate estimate of grain roughness and form drag on basis of grain size distribution of the bed material and the anticipated bed forms. The total roughness is found by adding both roughnesses together.

Einstein and Barbarosaa (1952) were the first to suggest a resistance relationship considering total resistance to be the sum of grain resistance and form resistance. According to them the entire expression for the form resistance is a function of shape, size, and frequency of the irregularities along the length of the channel. In an absolutely uniform channel these losses are obviously equal to zero. The effect of these irregularities is to distort the flow pattern. When the

discharge is least, the distortion of the flow pattern is greatest. As the discharge increases the distortion of the flow pattern becomes less and less because of the alignment of the flow becomes progressively straighter. Consequently, the additional friction loss diminishes as the discharge increases.

The thinking of form resistance as a part of total resistance is further modified by the researchers, with the increased knowledge of bed configuration of alluvial channels. The geometric characteristics of dunes are neither similar to ripples nor to that of anti-dunes. Moreover, superimposed ripples on dune bed made the problem more complicated. Bed form resistance changes with the change in size and length of a bed form as well as with the change in bed configurations - such as from ripples to dunes and from dunes to anti-dunes. On the other side, researchers are trying hard from analytical model study, laboratory flume study and using field data to formulate the form friction as a function of bed form height and length.

The Einstein and Barbarossa (1952) method was the first attempt to predict the stage-discharge relations for natural streams with an alluvial bed. The approach was based on the hypothesis that the resistance of an alluvial bed can be separated into two distinct entities - the resistance due to the presence of the grains and the resistance offered by the bed undulations. Einstein and Barbarossa have suggested the use of logarithmic relation, with a correlation term for viscous effects, to describe the resistance of the grains. The resistance corresponding to the bed undulations, U/U_*'', was shown to be a function of the average height, length and drag coefficient of the bed undulations; these in tern were assumed to be a function of intensity of shear on representative particles, ψ'. The concept of a form resistance diagram developed by Einstein and Barbarossa is now 45 years old. It is still probably the most widely quoted of any existing technique.

Bajorunas (1952) assumed that the Manning's roughness coefficient n is the sum of roughness coefficient n' due to grain roughness and roughness coefficient n'' due to form roughness. Bajorunas concurs that, for practical purposes, the Manning-Strickler equation is more convenient to use than logarithmic velocity equation for predicting grain roughnesses. Einstein and Chien (1958) suggested that the scatter of points of ψ' versus U/U_*'' could be due to

appreciable variations in the value of standard deviation of the bed materials. To check this, data from flumes and rivers, having almost the same sediment size, the same standard deviation and approximately the same value of ψ' were chosen, and the computed values of U/U_*'' for these runs were compared. Garde and Albertson (1959) have shown that Froude number is one of the parameter defining the regimes of flow and the approximate size of bed undulations, when the undulations were pronounced. Thus it was hoped that the lines of constant Froude number could be drawn on the plots of ψ' versus U/U_*''; but attempts in this direction did not lead to any systematization of the points on the plots of ψ' versus U/U_*''. Shen (1962) improved the Einstein and Barbarossa method and suggested that the resistance related to form is a function of both sediment transport and the Reynolds number related to fall velocity. Veiga de Cunha (1967) analyzed data from a Portuguese river with a bed composed of relatively coarse material. He concluded that for ψ' greater than about 5, U/U_*'' is a doubled-valued function of ψ' and presented a graphical relation between these two quantities.

Shinohara and Tsubaki (1959) on the basis of Nikuradse roughness value gave a prediction formula for the alluvial roughness. Vanoni and Nomicos (1960) found the reduction in Manning's n to be substantial if the bed is plane; however, their experiments also revealed that, with dunes on the bottom, the effect of suspended load on the resistance coefficient is a secondary importance. Liu and Hwang (1961) performed a dimensional analysis of the resistance problem and proposed an equation for mean velocity in a straight alluvial channel. Garde and Ranga Raju (1963) noted that the resistance of an alluvial channel may be expressed in dimensionless form. Viscous effects partly govern the resistance for plane bed with fine sediment. But the role of viscosity in the determination of resistance in the ripple and dune bed regime was not clearly understood. A check on the accuracy of Liu and Hwang (1961) method was made by Garde and Ranga Raju (1966) by comparing the observed velocities with the computed velocities for data from flumes and natural streams. The error was in many cases greater than ± 50 percent. In view of the large changes in resistance in different regime, analyzing data from flumes, canals and natural streams, Garde and Ranga Raju (1966) proposed different resistance relationships for different regimes. By analyzing additional data sets, Ranga Raju (1970) recognized needs of some modifications of Garde and Ranga Raju (1966) method. He found a noticeable effect of the sediment size on the resistance relationships.

Simons and Richardson (1961) concluded that the forms of bed roughness that exist in an alluvial channel because of similarities in form, resistance to flow and sediment transport are divided into categories of lower flow regime, transition regime and upper flow regime. They also concluded that the form of bed roughness depends primarily on the slope of the energy grade line, depth of flow, fall velocity or effective fall diameter of the bed material and slope of the channel. Simons and Richardson (1962) from flume study observed that the form of bed roughness in alluvial channels is a function of the sediment characteristics and the characteristics of the flow. The bed configuration can be changed by altering either discharge, slope, temperature or the median diameter or size distribution of the bed material. Simons and Richardson (1966) suggested a set of resistance equations for the computation of Chezy's C depending on each type of bed configuration.

Engelund (1966) proposed a hypothesis similar to Einstein and Barbarossa and divided energy gradient into components due to grain roughness and form drag. He proposed a new line of analysis based on similarity considerations. The essence of Engelund's similarity hypothesis is that for two dynamically similar streams, the dimensionless shear due to grain takes on equal values. In the closing discussion to his paper (Engelund, 1967) he introduced some modifications. Using data by Guy et al. (1966), Engelund and Hansen (1967) obtained relations for both lower and upper flow regimes. These relations are obtained from Engelund's universal relationship between dimensionless grain shear stress and the total shear stress. Brownlie (1983) extended the curve of Engelund (1966) into regions of the upper flow regime. Engelund (1977) used another approach for dunes to relate the form friction coefficient to the bed form parameters.

Vanoni and Hwang (1967) undertook to determine experimentally what geometric characteristics of alluvial channel bed forms determine their hydraulic roughness. They concluded that ripple-covered beds are hydrodynamically rough in the sense that the friction factor is independent of Reynolds number. Also the pressure on a ripple attains its minimum value at the crest and its maximum value on the face near the point of reattachment. Raudkivi (1967) conducted a set of laboratory flume experiments using 0.40 mm diameter sand and developed a graphical relationship which gives mean velocity as a function of shear velocity. Senturk (1965, 1967)

made a set of assumptions which lead to the determination of a group of formulae defining the resistance of flow in transition regions that also indicate the bed form expected.

Lovera and Kennedy (1969) proposed friction factor relations for flows with flat bed. Lovera and Kennedy constructed a chart for grain friction factor as a function of Reynolds number for constant values of relative depth. Alam and Kennedy (1969) developed a chart for predicting form drag friction as a function of relative depth for constant values of grain Froude number and constant values of Froude number.

A unique concept of the mechanics of alluvial channel flows was put forth by Maddock (1969). According to his view, the only determinate relationship among the variables for flow in alluvial streams is that between mean velocity, water discharge and sediment discharge. Moreover, this relationship is different for each of the three velocity ranges (low velocity, medium velocity and high velocity) he distinguished.

Mostafa and Mcdarmid (1971) avoided the division method and expressed Manning's n in a dimensionally homogeneous form. They expressed dimensionless Manning's coefficient as a function of Froude number and represented them graphically. Chu and Mostafa (1979) presented a technique which is essentially a mathematical expression of the graphical technique presented by Mostafa and Mcdarmid (1971). The new analysis allows a straight-forward adaptation of the technique to numerical modeling applications.

The abrupt change in the gradient of the bed form at the crest causes the flow to separate and form an eddy. A kinematic theory taking into account the presence of the leeward eddy and the effect of the bed form shape on the flow field was first developed by Mercer (1971). In Mercer's analysis, the length of eddy was fixed arbitrarily at one-half of the bed form wave length. His development was followed and generalized by Mercer and Haque (1973). Haque and Mahmood (1983) analytically determined that form friction depends on the steepness ratio of dune and relative roughness.

Bathurst (1978) proposed an empirical relationship for Darcy-Weisbach friction factor f. In steep mountain streams the individual roughness elements are large and may even protrude through the water surface. Bathurst (1985) delineated, with aid of data from rivers with slopes steeper than 0.4%, the lower limit of roughness of uniform flows. A detailed discussion of the problem of resistance to flow in gravel rivers can be found in Hey (1979). Brushin (1985) expressed the roughness of alluvial streams on the basis of Manning's roughness coefficient as a function of Nikuradse roughness and gravitational acceleration.

White et al. (1979) method for predicting the frictional resistance of alluvial channels is developed from experimental data. The method established a relationship between the total shear and effective shear stress for different sediment sizes for flows with Froude number less than 0.8, i.e. for lower flow regimes in which bed features are plain bed, ripples and dunes. They provided both graphical and equational representations of their technique, as well as a statistical analysis of the errors. White et al. (1987) extended White et al. (1979) method for upper flow regime flows in which the bed features are plane-bed or anti-dunes.

Brownlie (1983) developed a method for predicting alluvial roughness with the objective that it would be easily adapted to computer model applications. In this method sediment properties were described by the median particle size, specific weight and the geometric standard deviation under the assumption of log-normal distribution. Julian and Roman (1990) proposed resistance predictors for flow in steep channels. In Shen et al. (1990) study, the skin resistance, the form resistance and the total resistances were independently measured for a series of rigid bed forms in the laboratory. The data showed that the sums of skin resistance and form resistance are close to the total resistance for all experimental runs conducted.

Van Rijn (1982) has shown that the equivalent roughness or effective grain roughness of a flat bed can be related to the D_{90} of the bed material. Based on flume and field data he expressed form roughness as a function of dune height and length. Van Rijn (1984, 1989) based on the analyses of dune data, introduced form factor in the functional relationship of form roughness. Van Rijn (1993) proposed a separate expression for ripple bed for form roughnesses.

Vajda (1990) developed a method for the rationalization of local resistance calculation both due to boundary friction and combined alluvial grain and bed form roughnesses. Yalin (1992) predicted ripple and dune steepnesses separately and expressed Chezy resistance coefficient as the sum of grain roughness and form roughness. Karim and Kennedy (1983) formulated a friction-factor relation to account for the effect of bed configuration through the introduction of sediment discharge as one of the independent variables in a velocity function. Karim (1995) proposed a relation for Manning roughness coefficient that incorporates directly the role of bed configuration in determining alluvial bed resistance. A relation for bed-form height is developed, which is then used as an independent variable to account for the resistance due to bed form drag.

CHAPTER 3

METHODOLOGY

3.1 Introduction

Alluvial bed roughness has been an area of extensive investigation. Since the approach proposed by Einstein and Barbarossa (1952), many techniques have been proposed. Existing resistance relationships follow two different approaches: those that divide resistance into grain resistance and form resistance and those that do not. Six methods by which grain and form roughnesses are predicted separately and five methods for predicting overall alluvial roughness directly have been considered in this study. Brief discussions of the eleven selected methods are given in following articles.

3.2 Roughness Predictors Based on Division of Roughness

A. Einstein and Barbarossa (1952) method

According to Einstein and Barbarossa (1952), the frictional loss due to grain roughness may be evaluated by use of Manning's equation or any other suitable open channel formula. Einstein and Barbarossa recommended use of the Manning-Strickler equation

$$\frac{U}{U_*} = 7.66\left(\frac{R}{K_s}\right)^{\frac{1}{6}} \tag{3.1}$$

where U/U_* is a dimensionless quantity and R is the hydraulic radius pertaining to the grain roughness. If R′ is the hydraulic radius then abbreviation U_*' is introduced for $\sqrt{SR'g}$, and Eq. (3.1) becomes

$$\frac{U}{U_*'} = 7.66\left(\frac{R'}{K_s}\right)^{\frac{1}{6}} \tag{3.2}$$

This is the Manning-Strickler equation. The above equation can be replaced by the following logarithmic relation since it is better supported by theory:

$$\frac{U}{U_*} = 5.75\log(12.27\frac{R}{K_s}) \tag{3.3}$$

Like Manning-Stricker equation the above equation can be written as

$$\frac{U}{U_*'} = 5.75 \log(12.27 \frac{R'}{K_s}) \tag{3.4}$$

Equation (3.4) is valid for a rough boundary and hence a correction has to be introduced to make the equation applicable for fine materials. Hence Eq. (3.4) is written as

$$\frac{U}{U_*'} = 5.75 \log\left(12.27 \frac{R'}{K_s} x\right) \tag{3.5}$$

Where,

x is the correction factor, given in Fig. 3.1 and the correction is to be applied if

$$\frac{K_s}{\delta'} = \frac{K_s U_*'}{11.6 v} \leq 5 \tag{3.6}$$

If $\frac{K_s}{\delta'}$ is greater than 5, no correction is required.

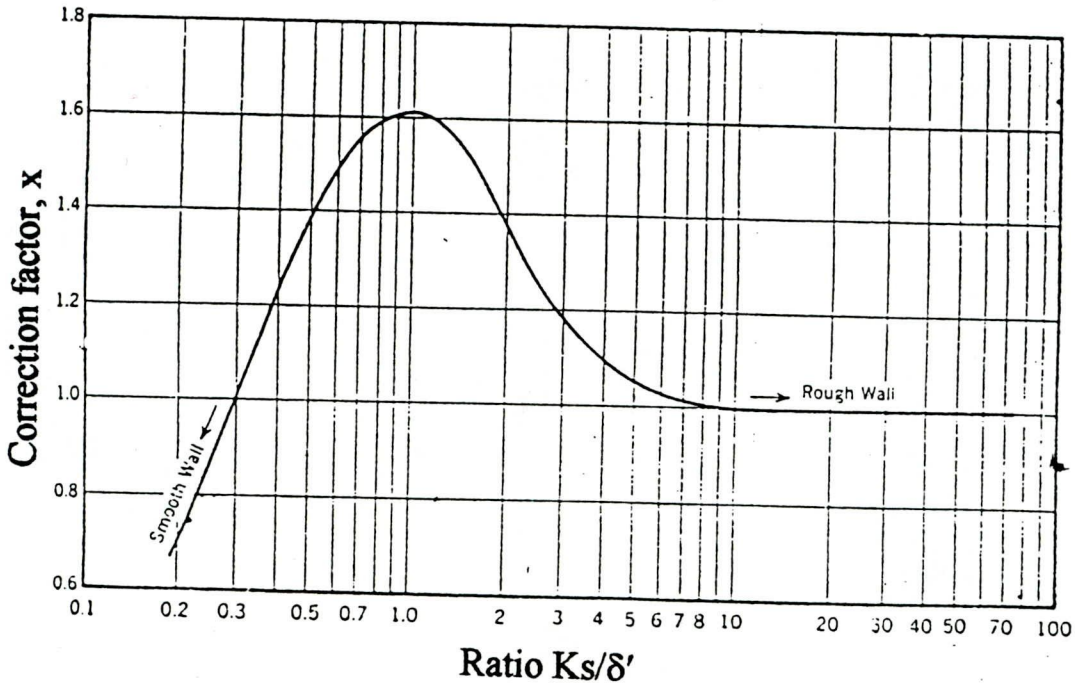

Fig.3.1 Correction x in the logarithmic friction formula (after Einstein, 1952)

Einstein and Barbarossa (1952) then argued that the bed form contribution to the friction factor

$$f'' = 8\left(\frac{U_*''}{U}\right)^2 = \frac{8gR''S}{U^2} \tag{3.7}$$

But according to Einstein's (1942) concept, the bed load transport depends solely upon the dimensionless variable ψ′ given by

$$\psi' = \frac{\gamma_s - \gamma}{\gamma}\frac{D_{35}}{SR'} = \frac{\Delta D_{35}}{SR'} \tag{3.8}$$

where, $\Delta = \frac{\gamma_s - \gamma}{\gamma} = \frac{\rho_s - \rho}{\rho}$ = relative submerged density.

Once the velocity of flow, bed level slope and bed sediment size distribution are known, then using Eqs.(2.38),(2.39) and (2.40) and also the relations $U_*' = \sqrt{gR'S}$ *and* $U_*'' = \sqrt{gR''s}$, the hydraulic radius R= (R′ + R″) and friction factor f can be predicted.

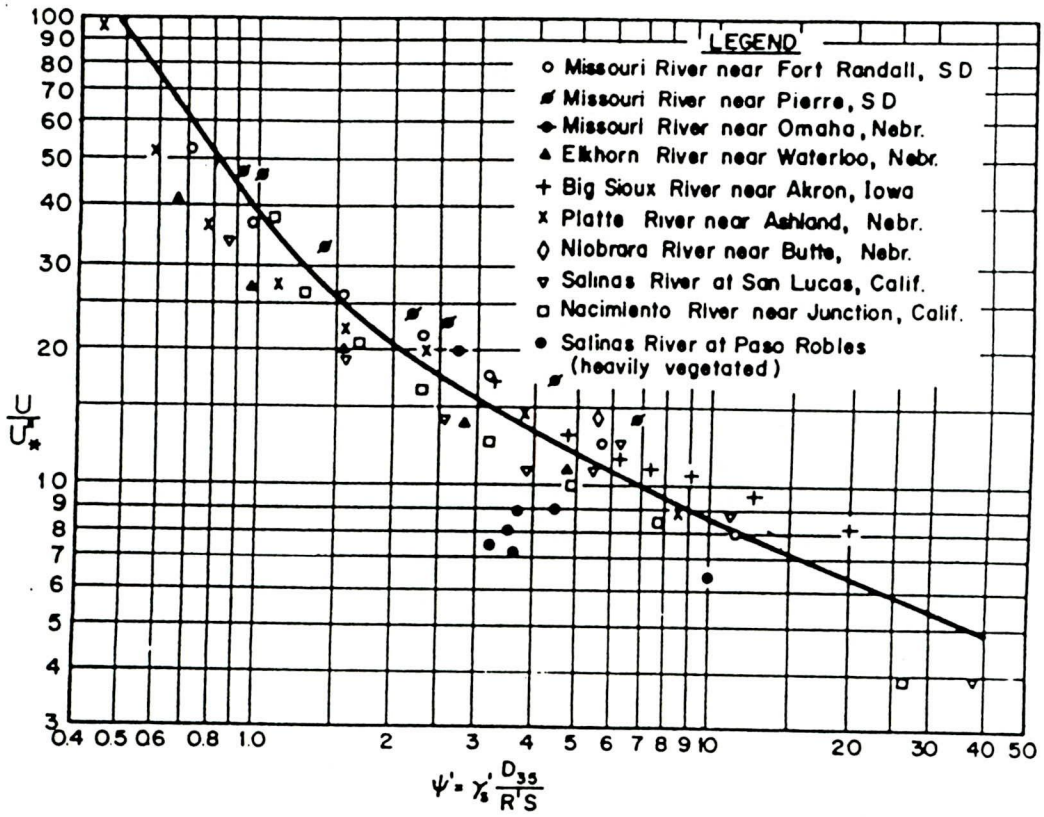

Fig.3.2 Friction loss due to channel irregularities as a function of sediment transport (after Einstein and Barbarossa, 1952)

Procedure

For a given set of U, D and S the procedure for obtaining the depth of flow and the roughness parameters is as follows:

(i) A value of R′ is selected and the value of $U_*' = \sqrt{gR'S}$ is computed.

(ii) The theoretical thickness δ′ of the laminar sub-layer is calculated by using Eq.(3.6).

(iii) If the left-hand side of the Eq. (3.6) is less than 5 then the mean velocity U is calculated by using Eq. (3.5), otherwise U is calculated by using Eq. (3.4).

(iv) Step (i) to (iii) are repeated until predicted and measured values of U are same.

(v) The dimensionless quantity ψ′ is calculated using Eq. (3.8).

(vi) The shear velocity U''_* is obtained using graphical relationship between U/U''_* and ψ′ as shown in Fig. 3.2.

(vii) The value of R″ is calculated using the equation $U_*'' = \sqrt{gR''S}$.

(viii) The hydraulic radius R is computed using Eq. (2.33).

B. Shen (1962) method

In Shen (1962) study, the emphasis is placed on the variation of the form resistance of the regularities under various flow conditions, as was proposed by Einstein and Barbarossa (1952). So Shen (1962) method is an improvement of the Einstein and Barbarossa (1952) method in particular to eliminate the systematic deviation that had been found to Einstein and Barbarossa (1952) curve for the functional relationship between U/U_*'' vs. ψ′ as shown in Fig. 3.2.

In open channel flows for which the gravitational force is predominated, the Froude number must be considered. The Froude number 1/ψ′ (a dimensionless parameter representing the intensity of shear on sediment particle) is quite important in the development of different bed patterns. But the parameter ψ′ is not sufficient to characterize and predict bed forms. Shen (1962) method defined a parameter λ that varies with the particle Reynolds number $\frac{w_s D}{\nu}$ as shown by Fig.3.3.

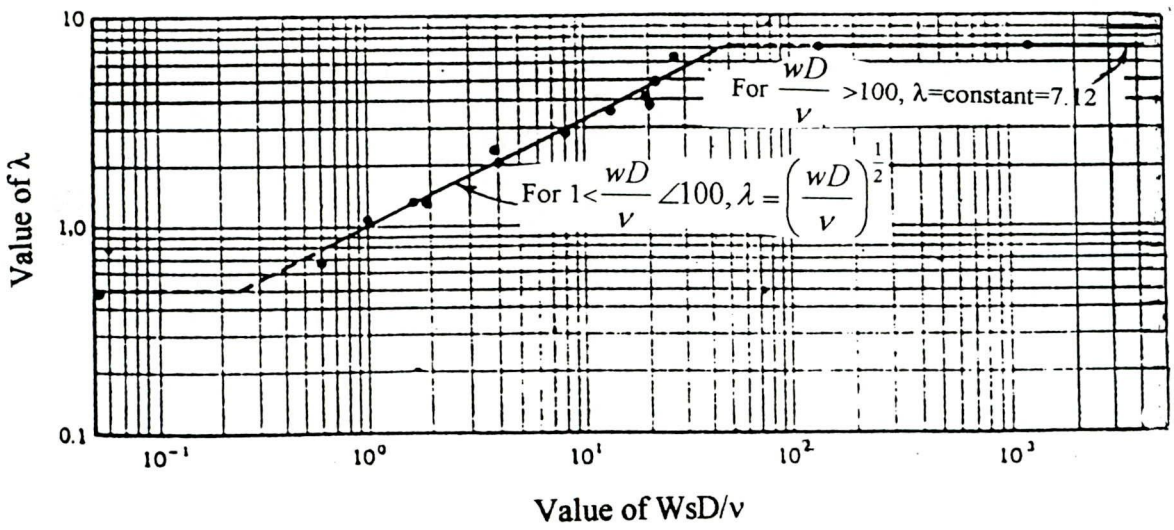

Fig.3.3 Variation of the correction parameter λ with w_sD/ν (after Shen, 1962)

The final results of Shen (1962) study was represented as follows:

1. The form resistance $\frac{U_*''}{U}$ is expressed as $A_1 + B_1 \log \frac{\psi'}{\lambda}$ in which A_1 and B_1 are constants and $\lambda = \text{Func}\left(\frac{w_s D}{\nu}\right)$. This resistance resembles the logarithmic friction law

$$\frac{U}{U_*''} = A_2 + B_2 \log\left(R' \frac{x}{K_s}\right) \tag{3.9}$$

in which A_2 and B_2 are constants and $x = Func\left(\frac{U_*' D}{\nu}\right)$.

2. For fine sediments with a value of $\frac{w_s D}{\nu} < 100$, the magnitude of f″ always decreases as ψ′ decreases. For fine sediment the form resistance is given by

$$\frac{U_*''}{U} = 0.03 + 0.11 \log\left(\frac{\psi'}{\lambda}\right) \tag{3.10}$$

As the value of ψ′ decreases, the bed configuration changes from ripple to plane bed, through stages of dune and transition.

3. For coarser material with value $\frac{w_s D}{\nu} > 100$, the variation of f″ is given by

$$\frac{U_*''}{U} = 0.03 + 0.11 \log\left(\frac{\psi'}{7.12}\right) \text{ for } \psi'<10, \text{ and} \tag{3.11}$$

$$\frac{U_*''}{U} = 0.064 - 0.0909 \log\left(\frac{\psi'}{7.12}\right) \text{ for } \psi'>10 \tag{3.12}$$

4. The values of λ were obtained as follows: First, all the experimental data were plotted on a graph with U_*''/U as the ordinate and ψ′ as the abscissa. It was found that if a certain value of λ is chosen for each experiment with a particular sediment size, then the values of U_*''/U and ψ'/λ have a definite relationship. These chosen values of λ for each sediment size are then plotted against their $\frac{w_s D}{\nu}$ as given in Fig. 3.3.

Procedure

For a given set of U, D and S, the procedure for obtaining the depth of flow and the roughness parameters is as follows:

(i) A value of R′ is selected and the value of U_*' is calculated using the equation $U_*' = \sqrt{gR'S}$.

(ii) The theoretical thickness δ′ of the laminar sub-layer is calculated using Eq. (3.6).

(iii) If the left-hand side of the Eq. (3.6) is less than 5 then the mean velocity U is calculated by using Eq. (3.5), otherwise U is calculated by using Eq. (3.4).

(iv) Step (i) to (iii) are repeated until predicted and measured values of U are same.

(v) The values of dimensionless quantity ψ′ and $w_s D/\nu$ are calculated.

(vi) The value of U_*''/U is calculated using Eqs. (3.10) or (3.11) or (3.12).

(vii) The value of R″ is calculated using the equation $U_*'' = \sqrt{gR''S}$.

(viii) The hydraulic radius R is computed using Eq. (2.33).

(ix) The friction factors f′ and f″ are obtained using Eqs. (2.38) and (2.39), respectively.

C. Engelund and Hansen (1967) method

As done by Einstein and Barbarossa (1952), Engelund and Hansen (1967) also proposed the splitting up the shear and hydraulic radius. Engelund and Hansen found in their experiment that the resistance of a plane bed with motion is larger than that of a plane bed with no sediment motion. The total mean energy gradient S per unit length of the channel is partly due to a certain friction loss S′ occurring along the stream side of the dunes and partly to a series of expansion losses that may in principle be accounted for by Carnot's formula

$$S'' = \frac{\Delta H}{L''} = \frac{q^2}{2gL}\left[\frac{1}{d-\frac{1}{2}H} - \frac{1}{d+\frac{1}{2}H}\right]^2 \approx \frac{U^2}{2gL}\left(\frac{H}{d}\right)^2 \tag{3.13}$$

Substituting Eq. (3.13) into S=S′+S″, we obtain

$$S = S' + \frac{U^2}{2gL}\left(\frac{H}{d}\right)^2 = S' + \frac{H^2}{2Ld}(F_r)^2 \tag{3.14}$$

The depth d may be replaced by the hydraulic radius R. Multiplying both sides of Eq. (3.14) by $\gamma R/(\gamma_s - \gamma)\, D_{50}$ gives

$$\frac{\gamma RS}{(\gamma_s-\gamma)D_{50}} = \frac{\gamma R'S}{(\gamma_s-\gamma)D_{50}} + \frac{\gamma}{(\gamma_s-\gamma)}\frac{H^2}{2LD_{50}}Fr^2 \tag{3.15}$$

Engelund and Hansen introduced into their analysis the dimensionless shear stress τ_* and its components τ_*' and τ_*''. Using

$$\tau_* = \frac{\gamma RS}{(\gamma_s-\gamma)D_{50}} \tag{3.16}$$

$$\tau_*' = \frac{\gamma R'S}{(\gamma_s-\gamma)D_{50}} \tag{3.17}$$

and

$$\tau_*'' = \frac{\gamma H^2}{(\gamma_s-\gamma)LD_{50}}\frac{U^2}{2gd} = \frac{\gamma H^2}{2(\gamma_s-\gamma)LD_{50}}Fr^2 \tag{3.18}$$

Equation (3.15) can be written as

$$\tau_* = \tau_*' + \tau_*'' \tag{3.19}$$

To obtain R′, Engelund and Hansen proposed the following logarithmic resistance formula:

$$\frac{U}{U_*'} = \frac{U}{\sqrt{gR's}} = 6 + 2.5\ln\frac{R'}{K_s} \tag{3.20}$$

where $K_s = 2D_{65}$

Using flume data, Engelund and Hansen (1967) obtained the following relationship for lower flow regime with a dune or ripple bed:

$$\tau_*' = 0.06 + 0.40\tau_*^2 \tag{3.21}$$

or

$$\tau_* = 1.581(\tau_*' - 0.06)^{1/2} \tag{3.22}$$

for $\tau_*' < 0.55$

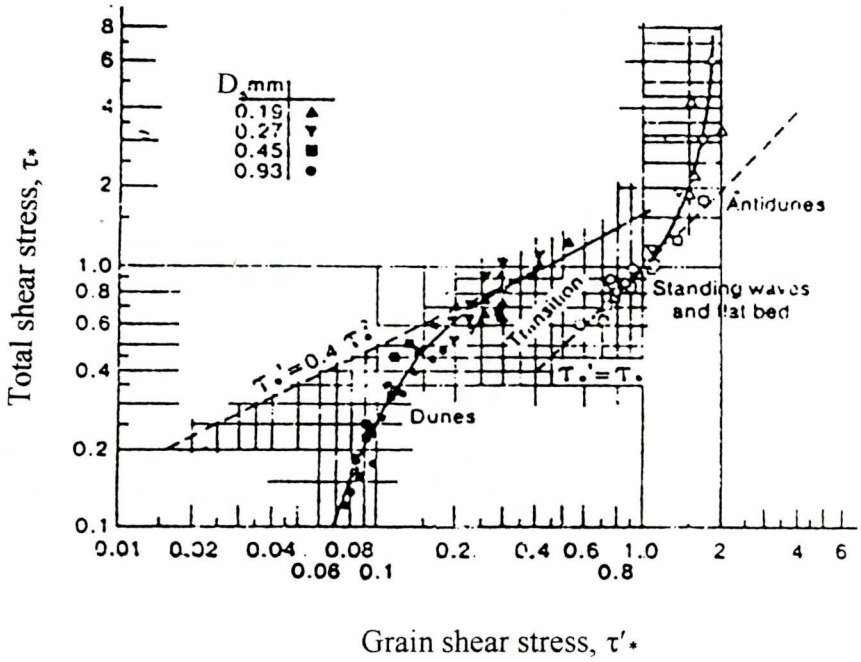

Fig.3.4 Engelund's universal relationship between dimensionless grain shear stress τ'_* and total shear stress τ_* (after Engelund et al., 1967)

For upper flow regime with $0.55 < \tau_*' < 1$, the suggested relationship is

$$\tau_*' = \tau_* \tag{3.23}$$

These relations are shown in Fig. 3.4 with a discontinuity between lower and upper flow regimes occurring at about $\tau_*' = 0.55$.

Brownlie (1983) extended Eq. (3.21) from the curve in Fig. 3.4 into higher regions of the upper flow regime for $\tau_*' > 1$ and obtained the relationship

$$\tau_*' = [0.702\,(\tau_*)^{-1.8} + 0.298]^{-1/1.8} \tag{3.24}$$

or

$$\tau_* = [1.425\,(\tau_*')^{-1.8} - 0.425]^{-1/1.8} \tag{3.25}$$

Procedure

Engelund and Hansen (1967) method can be used in predicting depth of flow for a given set of q, D and S and sediment characteristics. The procedure is as follows:

(i) A value of depth d′ or hydraulic radius R′ is assumed.

(ii) The dimensionless grain shear stress τ_*' and the mean velocity of flow U are computed uisng Eqs. (3.17) and (3.20), respectively.

(iii) If τ_*' is less than 0.55, the bed is in the lower flow regime. The total shear τ_* is computed by using Eq. (3.22). If τ_*' is greater than 0.55 and less than 1.00 then the flow in upper regime and τ_* is computed by using Eq. (3.23). For τ_*' greater than 1.00, the flow in higher regions of upper flow regime and τ_* is computed by using Eq. (3.25).

(iv) Using Eq. (3.16) the hydraulic radius R is computed.

(v) The discharge per unit width q is computed on the basis of calculated U and R.

(vi) The computed q is compared with the given q and the steps are repeated until these two q values are same.

F. Haque and Mahmood (1983) method

Bed features, such as ripples and dunes, play a significant role in the determination of hydraulic resistance to flow in alluvial streams. Haque and Mahmood (1983) recognized that the total drag at the bottom of the channel consists of two distinct parts: (1) the surface drag, and (2) the form drag. They took an attempt to calculate the form friction factor by integrating the pressure over a ripple or dune surface, and, thus, to discover the relationship between the form friction factor and the geometric parameters, such as steepness ratio and the relative roughness of the bed forms.

The observational fact is that the form friction factor is independent of the actual size of the bed form and depends only on the geometric shape of the flow region. Thus, only two parameters - one defining the shape of the lower boundary and the other defining the aspect ratio of the region are needed for the computation of f″. The numerical results of the parametric study are shown graphically in Fig. 3.5.

In this figure, a solid curve corresponds to a constant steepness ratio H/L of the bed form, and each dotted line represents a constant aspect ratio, $(d - \frac{H}{2})/L$, of the flow region. The points denoted by open circles represent the solutions obtained by the finite element analysis. The curve for H/L = 0.05 is, however, obtained by interpolation. The position of a typical interpolation point is determined from the following identify

$$\frac{H}{L} = \left(\frac{H}{d - \frac{H}{2}}\right)\left(\frac{d - \frac{H}{2}}{L}\right) \tag{3.26}$$

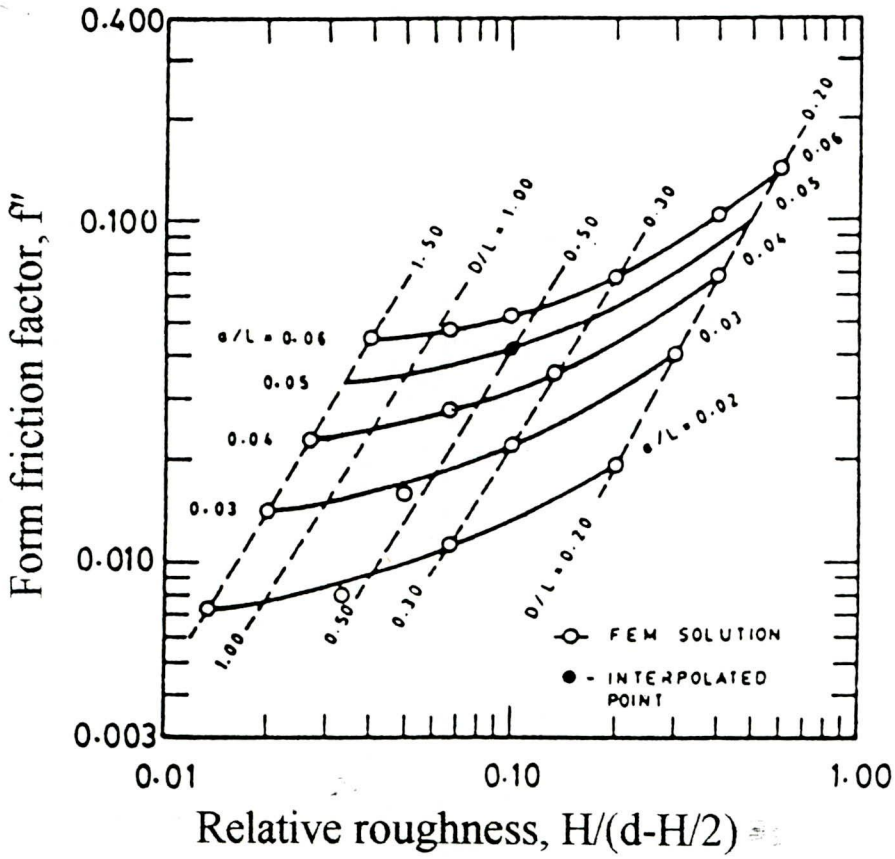

Fig.3.5 Relation between form friction factor f″ and relative roughness H/(d-H/2) (after Haque and mahmood, 1983

From Fig. 3.5, it may be noted that the dotted lines for $(d - \frac{H}{2}) / L > 0.5$ are almost parallel, and the family of curves in this region can be expressed in the mathematical form

$$f'' = \phi\left(\frac{d - \frac{H}{2}}{L}\right) \cdot \left(\frac{H}{d - \frac{H}{2}}\right)^{m} \tag{3.27}$$

in which m is the slope of dotted lines and is found to have a numerical value of 1.653. The function f depends on $(d - \frac{H}{2}) / L$ and its behavior is obtained by studying the variation of f'' along $\left(\frac{H}{d - \frac{H}{2}}\right) = 1$. The variation of ϕ is shown in Fig. 3.6.

From this figure, it is evident that for $\left(d - \frac{H}{2}\right) / L > 0.5$, the function ϕ can be expressed quite accurately by

$$\phi = 4.9\left(\frac{d - \frac{H}{2}}{L}\right)^{1.477} \tag{3.28}$$

and the form friction factor can be expressed by the equation

$$f'' = 4.9\left(\frac{d - \frac{H}{2}}{L}\right)^{1.477} \left(\frac{H}{d - \frac{H}{2}}\right)^{1.653} \tag{3.29}$$

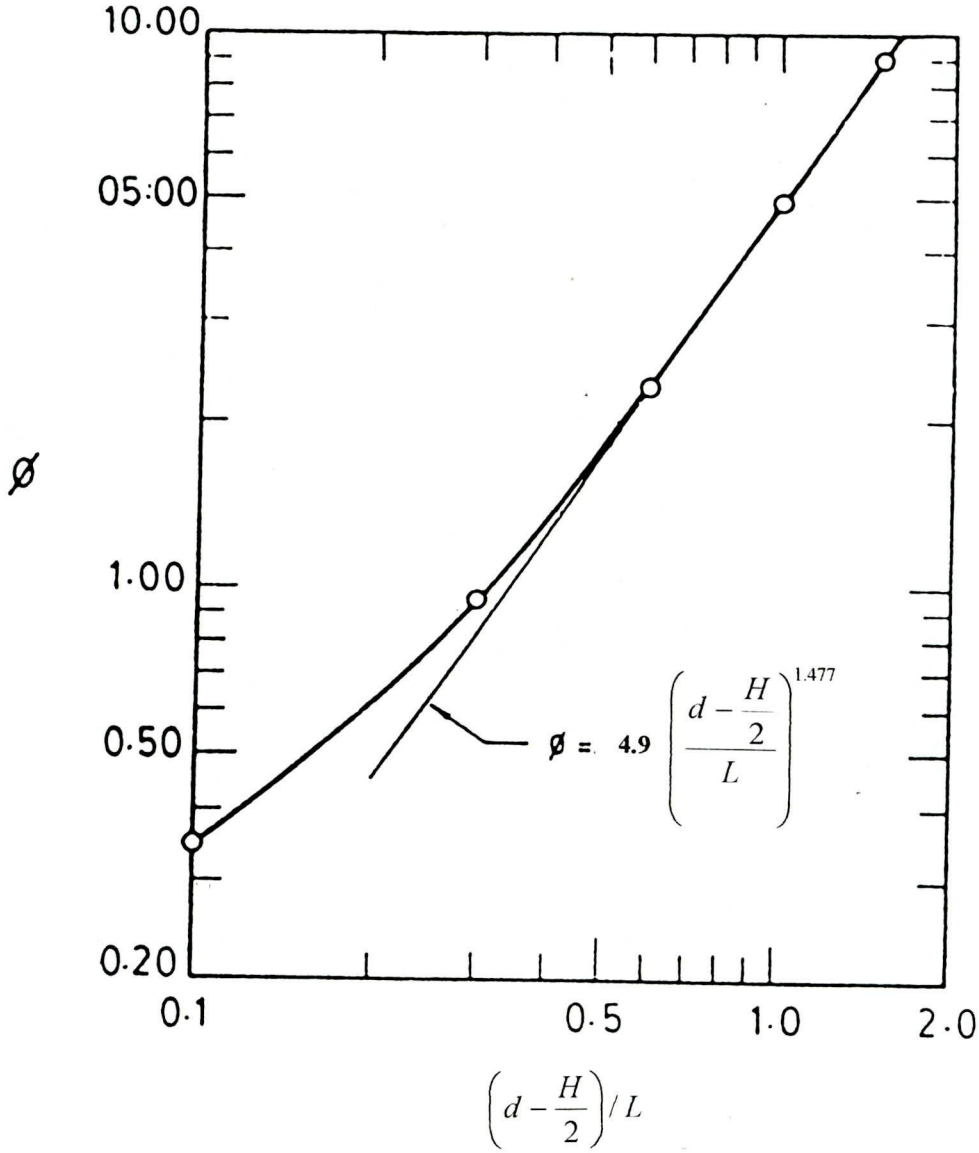

Fig.3.6 Variation of θ with relative depth (d-H/2)/L (after Haque and Mahmood, 1983)

Procedure

For a given set of bed form height and length, the form friction factor f″ can be obtained by using the Haque and Mahmood method. The dimensions of the bed forms can be predicted by using Vajda (1990) or Yalin (1992) method. The grain roughness can be predicted by using White-Colebrook formula. The procedure is as follows:

(i) Bed form dimensions are computed by using Vajda (1990) or Yalin (1992) method.

(ii) The grain roughness is predicted by using formula for grain roughness presented by Eq. (2.12).

(iii) For relative depth $\left(d - \frac{H}{2}\right)/L$, the corresponding functional value of φ is predicted using Fig. 3.6.

(iv) The form friction factor f″ is computed using Eq. (3.29).

(v) The overall friction factor f is computed using Eq. (2.40).

G. Shen et al. (1990) method

Shen et al. (1990) derived a relationship to predict the resistance as a function of flow Reynolds number, relative roughness, and relative height of dunes. According to Shen et al. (1990) method the variation of shear stress along the upstream face of a bed form may be studied through the use of a local skin shear coefficient C_f, defined as

$$C_f = \frac{\tau_x}{\frac{1}{2}\rho U_x 2} = \frac{8gRS}{4U_*^2} = \frac{f_x'}{4} \tag{3.30}$$

where flow is along the longitudinal direction x along the base of dunes and parallel to the flow direction and

$$f_x' = \frac{8\tau_x'}{\rho U_x^2} \tag{3.31}$$

When boundaries are hydraulically smooth and the Reynolds number Re<100,000, the local friction factor f_x' is given by the Blasius formula

$$f_x' = \frac{0.316}{Re^{0.25}} \tag{3.32}$$

which can be rewritten in terms of C_f as

$$C_f = \frac{0.079}{Re^{0.25}} \tag{3.33}$$

where, the Reynolds number Re = q/ν and q= discharge per unit width.

The local skin shear coefficients were computed for each measured shear stress using Eq. (3.30). The local skin shear coefficients were plotted against Reynolds number Re for different longitudinal distance from the upstream toe.

For the bed form investigated, the measured data fit the relationship

$$C_f = \frac{0.041\frac{x}{A} - 0.085}{(Re)^{0.25}} \tag{3.34}$$

where, x is the longitudinal distance from the upstream toe, and H is the bed form height.

Neglecting the small shear stress present within the separation zone, the longitudinal force per unit width due to skin shear F_s is given by

$$F_s = \int \tau_x dx \tag{3.35}$$

The bed skin friction factor f' is computed from

$$f' = \frac{8\tau'}{\rho U^2} = \frac{8}{\rho U^2}\left(\frac{F_s}{L}\right) \tag{3.36}$$

Shen et al. (1990) method suggested that the equation

$$f' = \left(\frac{0.106d}{d_c}\right)\frac{1}{(Re)^{0.25}} \tag{3.37}$$

fits the measured skin friction factor f' values very well.

The form friction factor f'' is computed as

$$f'' = \frac{8gR''S}{U^2} = \frac{8\tau''}{\rho U^2} = \frac{8F_p}{L\rho U^2} \tag{3.38}$$

where the form shear stress $\tau'' = \frac{F_p}{L}$, pressure drag coefficient F_p is taken as positive in the downstream direction.

The pressure drag force consists of two components: a viscous shear force and a turbulent pressure force. Generally the drag force consists of both components and can be expressed as

$$F_P = \frac{1}{2}\rho U^2 C_D H \tag{3.39}$$

or

$$C_D = \frac{F_P}{\frac{1}{2}\rho U^2 H} \tag{3.40}$$

Combining Eqs. (3.38) and (3.40) yields

$$C_D = \frac{1}{4} f'' \frac{L}{H} \tag{3.41}$$

The pressure drag coefficient C_D was plotted against the relative roughness H/d. These investigators proposed a relationship between C_D and H/d of the form

$$C_D = m\left(\frac{H}{d}\right)^{3/8}$$

where m = a coefficient dependent only on the bed form steepness ratio with a mid range value of m = 4/9 for H/d≤ 0.35. For relative roughness H/d≥ 0.35, a power relation with an exponent of 3/2 and m equal to 3/2 fits the data well. The pressure drag coefficient C_D may be related to the relative roughness as follows:

$$C_D = \frac{4}{9}\left(\frac{H}{d}\right)^{3/8} \; for \; \frac{H}{d} \le 0.35 \tag{3.42}$$

$$C_D = \frac{3}{2}\left(\frac{H}{d}\right)^{3/2} \; for \; \frac{H}{d} > 0.35 \tag{3.43}$$

The form friction factor f″ can be computed from the pressure drag coefficient and the bed form steepness ratio as

$$f'' = 4 C_D \frac{H}{L} \tag{3.44}$$

Procedure

For a given set of bed form height and length the form friction factor f″ can be computed by using Shen et al. (1990) method. The dimensions of bed form can be obtained by using Allen (1963) or Vajda (1990) or Yalin (1992) method. Other known parameters are R, S, and D. The procedure is as follows:

(i) Bed form dimension is predicted by using Allen (1963) or Vajda (1990) or Yalin (1992) method.

(ii) Pressure drag coefficient C_D is computed by using Eq. (3.42) for H/d ≤0.35 or by using Eq. (3.43) for H/d>0.35.

(iii) The form friction factor f″ is computed using Eq. (3.44).

(iv) The grain friction factor f′ is computed using Eq. (3.36).

(v) The overall friction factor f is computed using Eq. (2.40).

H. Van Rijn (1993) method

Van Rijn (1993) presented a method that makes the classification of bed forms, the prediction of the bed forms dimensions and the effective hudraulic roughness of the bed forms feasible. Van Rijn (1993) divided the total alluvial roughness as

$$K_s = K_s' + K_s'' \tag{3.45}$$

and expressed the White and Colebrook equation for Chezy coefficient found from Eq. (2.12) as

$$C = 18\,\mathrm{Log}\left(\frac{12R}{K_s}\right)$$

in which

$K_s' = 3D_{90}$ for $\theta<1$ (lower regime)

and

$K_s' = 3\theta D_{90}$ for $\theta>1$ (upper regime), where the mobility parameter θ is given by

$$\theta = \frac{U_*^2}{\frac{(\rho_s - \rho)}{\rho} g D_{50}}$$

The effective form roughness is related to bed form height H, the bed form steepness H/L and the bed form shape γ. The functional relationship is as follows:

$$K_s'' = F(H, \frac{H}{L}, \gamma) \tag{3.46}$$

The most general case is that of a bed consisting of mega-ripples superimposed on dunes and sand waves. It is proposed to determine the overall form roughness by summation of individual values, i.e.

$$K_s'' = K_{sr}'' + K_{sd}'' + K_{ssw}'' \tag{3.47}$$

in which

K_{sr}''= form roughness related to ripples.

K_{sd}''= form roughness related to dunes

K''_{ssw} = form roughness related to sand waves.

Van Rijn proposed the following equation to compute K''_{sr}, K''_{sd} and K''_{ssw}:

$$K_{sr}' = 20\gamma_r H_r\left(\frac{H_r}{L_r}\right) \tag{3.48}$$

where, γ_r = ripple presence factor 1 for ripple alone and 0.7 for ripples superimposed on dunes or sand waves.

$$K_{sd}'' = 1.1\gamma_d H_d\left(1 - e^{-25\frac{H_r}{L_r}}\right) \tag{3.49}$$

where, γ_d = form factor (=0.7 for field conditions).

Sand waves are defined as symmetrical bed forms with a length much larger than the water depth. The lee side slopes of symmetrical sand waves are relatively mild. Hence flow separation will not occur.

Therefore, the form roughness of symmetrical sand waves is assumed to be zero. Thus

$$K''_{ssw} = 0. \tag{3.50}$$

Procedure

When the velocity of flow U, slope of the channel S, and particle diameter D are known, the hydraulic radius and roughness parameters can be predicted as follows:

(i) A hydraulic radius R is assumed.

(ii) With assumed hydraulic radius the transport stage parameter T is calculated using Eq. (2.44).

(iii) The regime of flow is obtained knowing the transport stage parameter T.

(iv) The respective bed form length is computed using Eq. (2.83), (2.85), (2.87) or (2.89) depending on whether the bed form is ripple, mega-ripple, dune or anti-dune.

(v) The bed form height is computed using Eqs. (2.82), (2.84), (2.86) or (2.88) depending on regime of flow.

(vi) The Nikuradse grain roughness value is obtained by $K_s' = 3D_{90}$ if θ is less than 1.00 and by $K_s' = 3\theta D_{90}$ if θ is greater than 1.00.

(vii) Nikuradse form roughness related to ripple K_{sr}'' is computed by using Eq. (3.48). Nikuradse form roughness that related to dune K_{sd}'' and standing wave K_{ssw}'' are calculated using Eqs. (3.49) and (3.50), respectively.

(viii) The K_s'' value is the sum of K_{sr}'' and K_{sd}''.

(ix) The Chezy's resistance factor C is computed by using Eq. (2.12).

(x) The hydraulic radius is computed by using Eq. (2.4).

(xi) If the computed and assumed values of R are not same the above procedure is repeated.

3.3 Roughness Predictors Based on Overall Roughness

A. Simons and Richardson (1966) method

The United State Geological Survey conducted an extended laboratory study of alluvial channel flow, using the 8-ft wide, 150-ft long and 2-ft wide, 60-ft long laboratory flumes at Colorado State University. Simons and Richardson (1966) analyzed those data together with those obtained in natural streams and irrigation canals by several others investigators with the goal of obtaining an improved friction factor for alluvial channels. Simons and Richardson (1966) suggested a set of resistance equations for the computation of Chezy's C depending on each type of bed configuration. A graphical relation, developed by Simons and Richardson (1966), that relates stream power, median fall diameter of bed material and bed form type is shown in Fig. 3.7.

Once the bed configuration can be confirmed by trial and error, then the Chezy's roughness factor C can be computed from depth of flow, bed level slope and grain size D_{85}.

The following form resistance equations (in English units) were suggested

i) For a plane bed without sediment transport

$$C/\sqrt{g} = 5.9 \log (d/D_{85}) + 5.44 \tag{3.51}$$

ii) For a plane bed with sediment transport

$$C/\sqrt{g} = 7.4 \log (d/D_{85}) \tag{3.52}$$

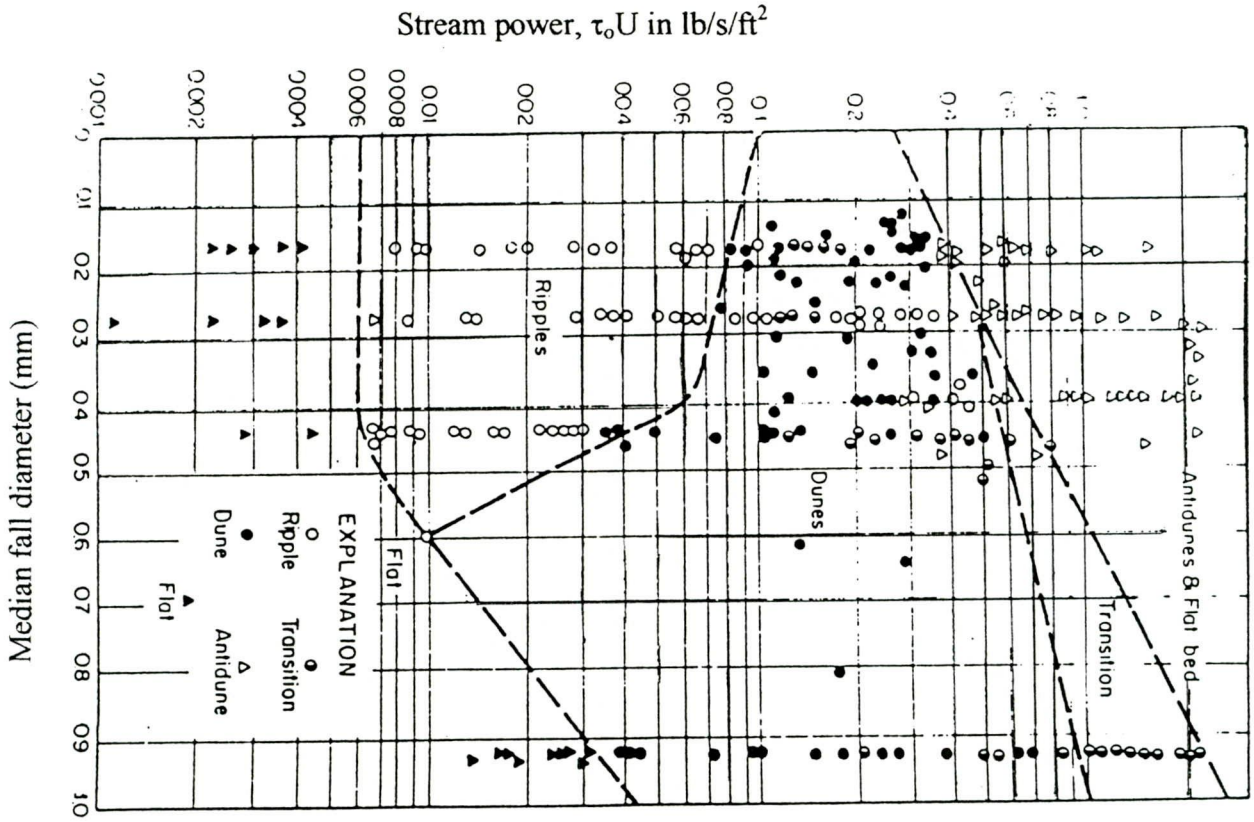

Fig.3.7 Relation of bed form to stream power and median fall diameter of bed sediment (after Simons and Richardson, 1966)

iii) For ripples

$$C/\sqrt{g} = (7.66 - 0.3/U_*) \log d + 0.13/U_* + 11 \tag{3.53}$$

iv) For dunes and anti-dunes

$$C/\sqrt{g} = 7.4 \log(d/D_{85}) \sqrt{\left(1 - \frac{\Delta RS}{RS}\right)} \tag{3.54}$$

The term ΔRS is an adjustment for RS to compensate for the form roughness. For dunes, ΔRS is a function of RS as shown in Fig. 3.8. For anti-dunes, the correction term $\sqrt{1-\frac{\Delta RS}{RS}}$ is a function of sand size, shear stress and depth, as shown in Figs. 3.8 and 3.9.

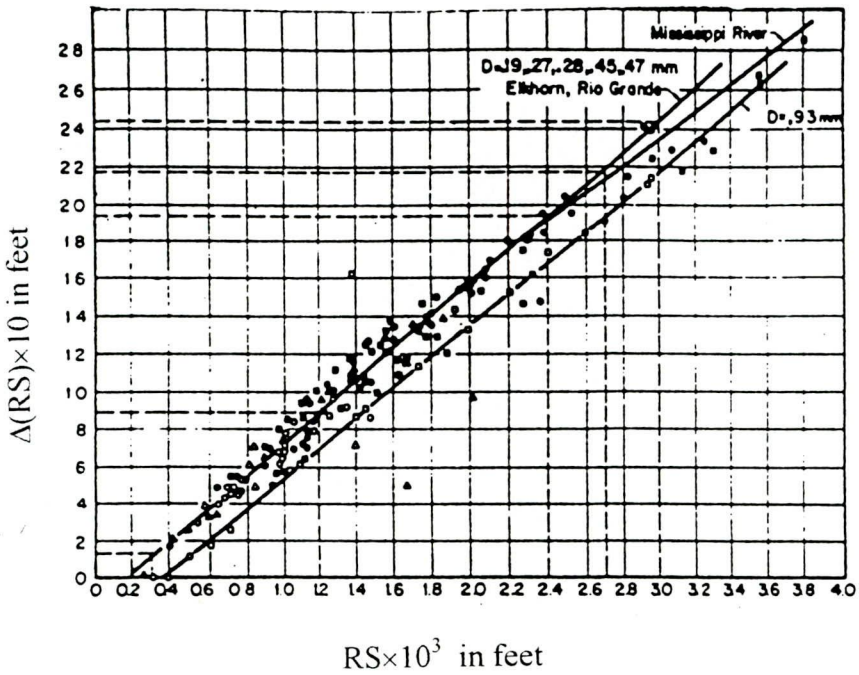

Fig.3.8 Relation between ΔRS and RS for the dune bed configuration (after Simons and Richardson, 1966)

Procedure

Simons and Richardson to determine the average velocity of flow in a sand bed channel with known values of R, S, D_{50} and D_{85} suggest the following procedure.

(i) A bed configuration is assumed.

(ii) The value of $C/\sqrt{g}$ is calculated using the appropriate equation (Eqs. 3.51 - 3.54).

(iii) Mean velocity of flow, U, is computed using Eq. $U = C\sqrt{RS}$.

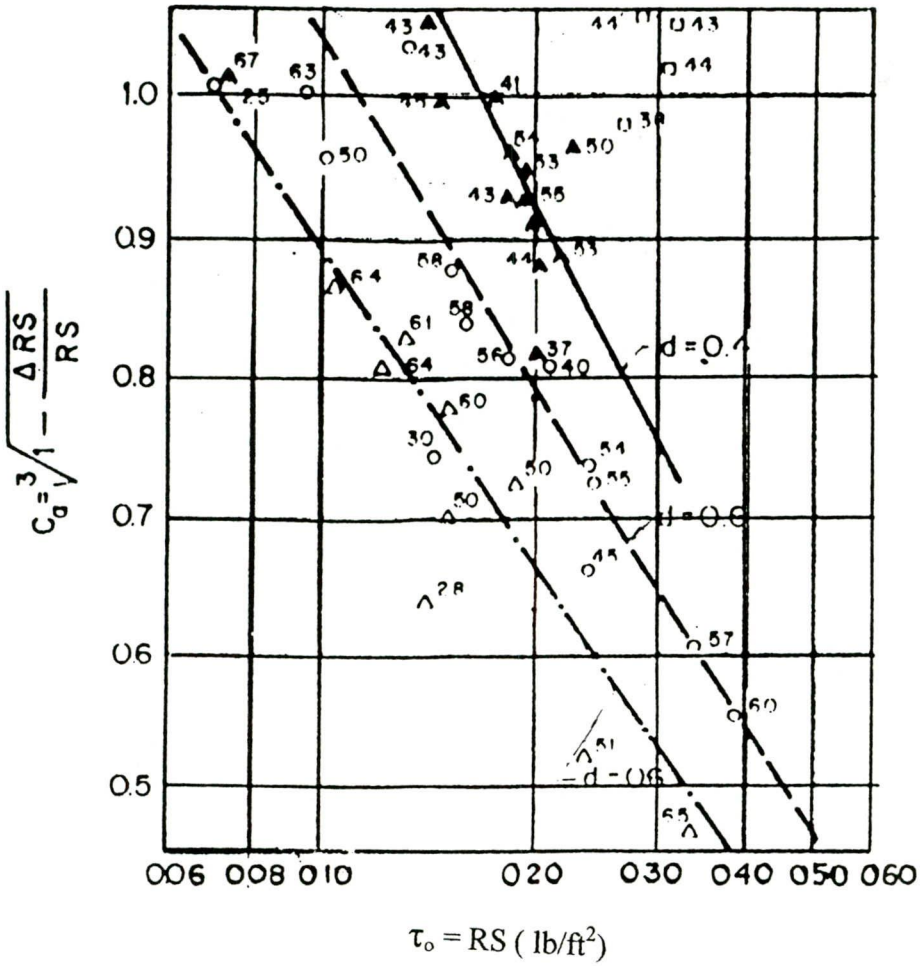

Fig.3.9 Relation between the Chezy coefficient correction term, shear stress and depth for antidune bed configuration (after Simons and Richardson, 1966)

(iv) Stream power $\tau_0 U$ is computed using $\tau_0 = \gamma RS$.

(v) Using the graphical relationship between stream power and median fall diameter shown in Fig. 3.7, the bed form is predicted and checked with the assumed one.

(vi) If the assumed and predicted bed forms are different, steps from 1 to 5 are repeated using the bed form found.

B. Garde and Ranga Raju (1977) method

The Garde and Ranga Raju (1977) method is the revision of Garde and Ranga Raju (1966) and Ranga Raju (1970) methods. Garde and Ranga Raju (1977) analyzed data from flume studies,

canals and natural streams and proposed an empirical equation for the mean velocity. Using the principles of dimensional analysis, the functional relationship was written as

$$\frac{U}{\sqrt{\frac{\rho_s - \rho}{\rho} g D_{50}}} = f\left[\frac{R}{D_{50}}, \frac{S}{\frac{\rho_s - \rho}{\rho}}, \frac{g^{1/2} D_{50}^{3/2}}{\nu}\right] \tag{3.55}$$

The functional relationship given in Eq. (3.55) may be rewritten after some modifications as

$$\frac{U}{\sqrt{g \frac{\rho_s - \rho}{\rho} R}} = f\left[\frac{R}{D_{50}}, \frac{S}{\frac{\rho_s - \rho}{\rho}}, \frac{g^{1/2} D_{50}^{3/2}}{\nu}\right] \tag{3.56}$$

Eqs. (3.55) and (3.56) were used as the basis of analyses of alluvial channel data from canals, rivers and laboratory flumes. By the use of graphical plotting they showed that

$$\frac{U}{\sqrt{g \frac{\rho_s - \rho}{\rho} R}} = f\left[\left(\frac{R}{D_{50}}\right)^{1/3}, \frac{S}{\frac{\rho_s - \rho}{\rho}}, \frac{g^{1/2} D_{50}^{3/2}}{\nu}\right] \tag{3.57}$$

Viscous effects partly govern the resistance for the plane bed regime with fine bed material. But the role played by viscosity in the determination of resistance in the ripple and dune bed regime is not clearly understood. The parameter $\frac{g^{1/2} D_{50}^{3/2}}{\nu}$ was considered insignificant as a first approximation and the following relationship was obtained:

$$\frac{U}{\sqrt{g \frac{\rho_s - \rho}{\rho} R}} = f\left[\left(\frac{R}{D_{50}}\right)^{1/3}, \frac{S}{\frac{\rho_s - \rho}{\rho}}\right] \tag{3.58}$$

Data for a particular sediment size tend to follow a single curve but the curves are different for different sediment sizes. A unique curve was obtained by plotting

$$K_1 \frac{U}{\sqrt{g \frac{\rho_s - \rho}{\rho} R}} \text{ versus } K_2 \left(\frac{R}{D_{50}}\right)^{1/3} \left(\frac{S}{\frac{\rho_s - \rho}{\rho}}\right)$$

as shown in Fig. 3.10.

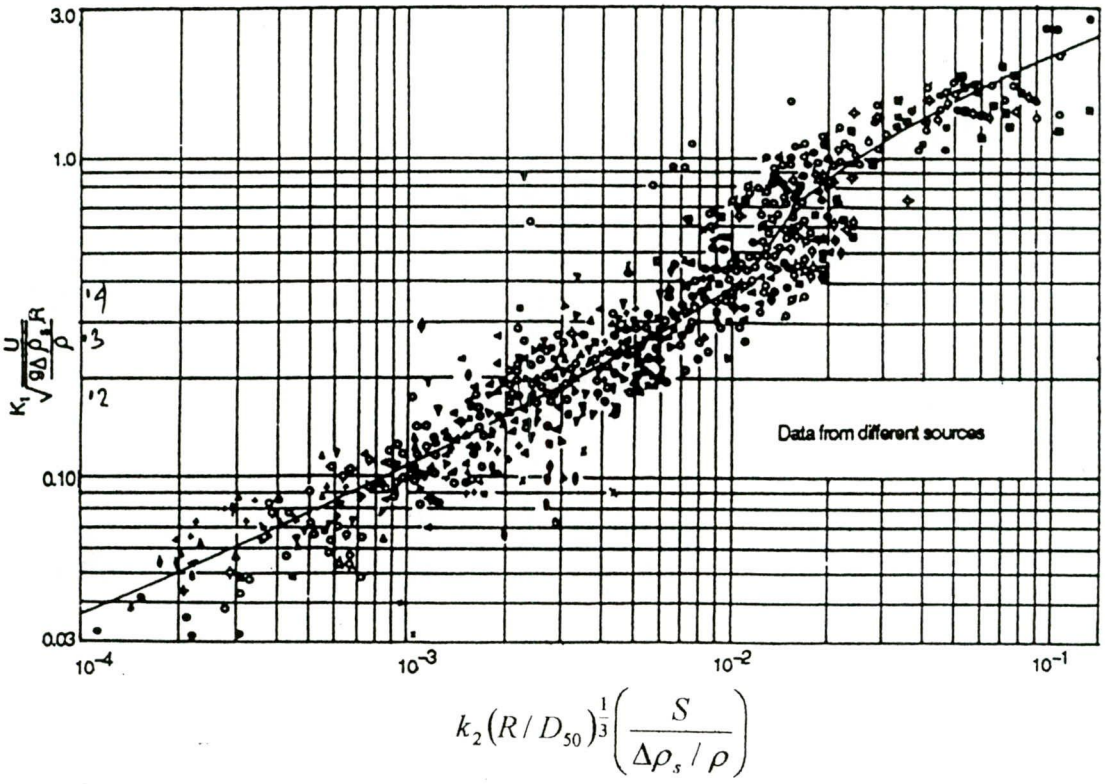

Fig.3.10 Roughness predictor for alluvial channel (after Ranga Raju, 1970)

The empirical coefficients K_1 and K_2 are functions of sediment size as shown in Fig. 3.11. For given values of R, S, ρ_s, ρ and D_{50} the coefficients K_1 and K_2 may be read and the parameter $\left(\frac{R}{D_{50}}\right)^{1/3}\left(\frac{S}{(\rho_s-\rho)/\rho}\right)$ is obtained. The Froude number $K_1 \frac{U}{\sqrt{gR\frac{\rho_s-\rho}{\rho}}}$ is read from Fig. 3.11 and the mean velocity U can be predicted.

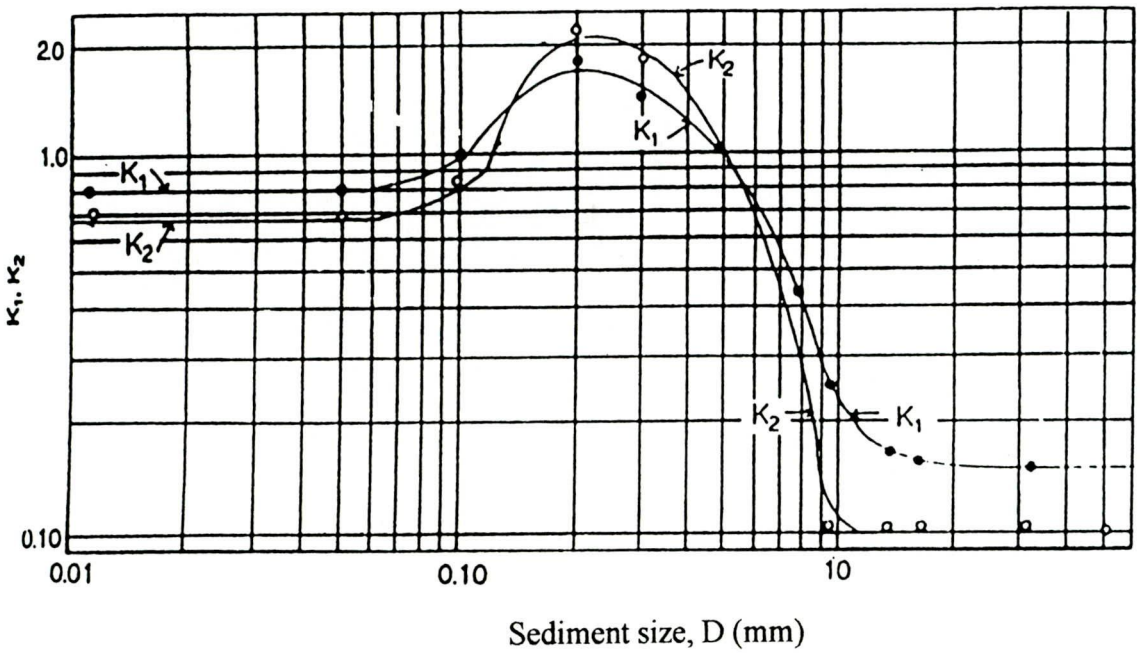

Fig.3.11 Variation of K_1 and K_2 with sediment size (after Ranga Raju, 1970)

Procedure

The following procedure is suggested by Garde and Ranga Raju (1977) to determine the average velocity in a sand-bed channel with known values of R,S and D.

(i) The values of K_1 and K_2 are computed using the graphical relation of K_1 and K_2 with the median grain size diameter D_{50} shown in Fig. 3.11.

(ii) The value of $K_2\left(\frac{R}{D_{50}}\right)^{1/3}\left(\frac{S}{\frac{\rho_s-\rho}{\rho}}\right)$ is calculated.

(iii) Using Fig 3.10, the value of $K_1\frac{U}{\sqrt{gR\frac{\rho_s-\rho}{\rho}}}$ is computed.

(iv) In the expression $K_1 \frac{U}{\sqrt{gR\frac{\rho_s - \rho}{\rho}}}$ the parameters K_1, R, ρ_s, ρ and g are known, then the velocity of flow U can be calculated.

(v) The roughness coefficients n, C and f are computed by Eqs. (2.1), (2.5) and (2.7) using the flow velocity obtained in step (iv).

C. Brownlie (1983) method

By Brownlie (1983) method, using the π-Theorem, the variables involved in alluvial channels can be arranged into six dimensionless groups in the form

$$\frac{RS}{D_{50}} = \frac{(\rho_s - \rho)}{\rho}\tau_* = f\left(q_*, S, \sigma_g, P, \frac{\rho_s - \rho}{\rho}\right) \tag{3.59}$$

where, $q_* = \frac{q}{\sqrt{gD_{50}^3}}$, $P = \frac{4q}{\nu}$ and q= discharge per unit width.

Since we are concerned with fully rough flow, P is expected to be of secondary importance. Furthermore, since only sand is under consideration, $\frac{\rho_s - \rho}{\rho}$ will be constant, and can be put aside. Therefore, Eq. (3.59) can be reduced to

$$\frac{\rho_s - \rho}{\rho}\tau_* = \text{Func}\ (q_*, S, \sigma_s) \tag{3.60}$$

The hydraulic radius R is computed for lower flow regime from

$$R/D_{50} = 0.3724(q_*)^{0.6539}\ S^{-0.2542}\ \sigma_g^{\ 0.1050} \tag{3.61}$$

and for upper flow regime from

$$R/D_{50} = 0.2836(q_*)^{0.6248}\ S^{-0.28577}\ \rho_g^{\ 0.08013} \tag{3.62}$$

Flow regime

After considering many possible dimensionless parameters, the following four were selected by Brownlie as indicators of flow regime:

$$F_g, \frac{D_{50}}{\delta}, S, \sigma_g$$

Here F_g = the grain Froude number defined as $\frac{U\sqrt{\rho}}{\sqrt{(\rho_s - \rho)gD_{50}}}$ and d is the thickness of the laminar sub-layer $= \frac{11.6\nu}{U_*'}$

The flow regime relationship between F_g and S is shown in Fig. 3.12. The first point that is obvious from Fig. 3.12 is that beyond a slope of S = 0.006 only upper flow regime exists. For lower values of slope, an approximate dividing line can be defined by

$$F_g = F^*_g = 1.74\ S^{-1/3} \tag{3.63}$$

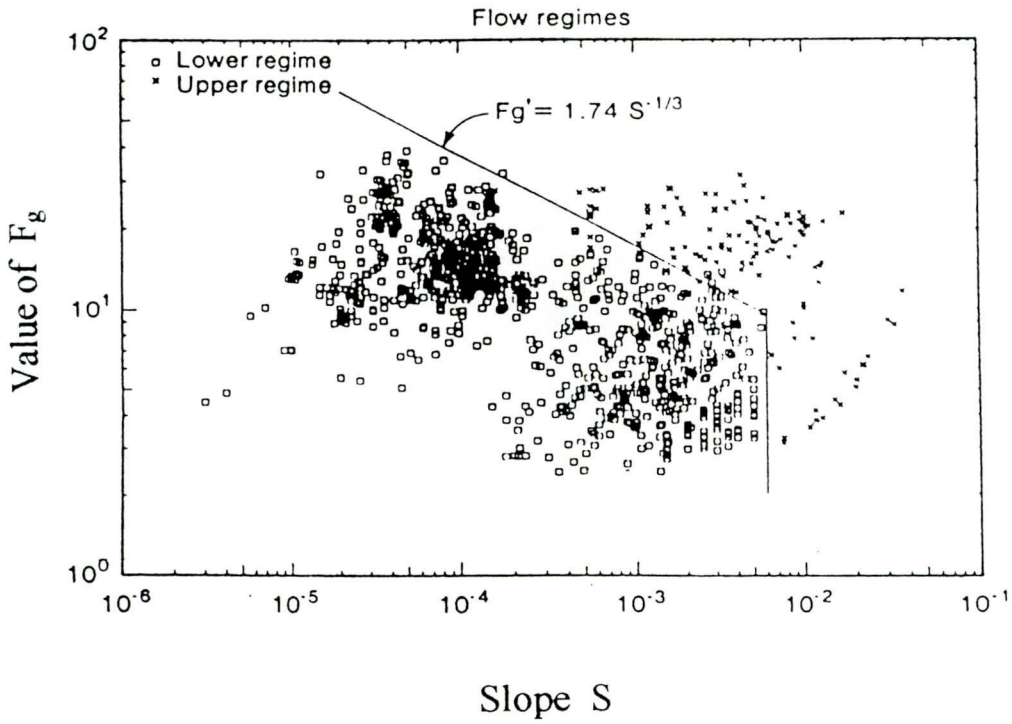

Fig.3.12 Determination of flow regimes-grain Froude number, F_g, plotted against slope, S (after Brownlie, 1983)

The overlap along this line indicates that an additional variable will be needed to better define this transition zone. The transition region can be defined by the equations for the lower limit of upper flow regime.

$$\text{Log}\frac{F_g}{F_g^*} = -0.02469 + 0.1517\log\frac{D_{50}}{\delta} + 0.8381\left(\log\frac{D_{50}}{\delta}\right)^2 \; for \; \frac{D_{50}}{\delta} < 2 \tag{3.64}$$

$$\text{Log}\frac{F_g}{F_g^*} = \log(1.25) \; for \; \frac{D_{50}}{\delta} \geq 2 \tag{3.65}$$

and for the upper limit of the lower flow regime

$$\text{Log}\frac{F_g}{F_g^*} = -0.2026 + 0.07026\log\frac{D_{50}}{\delta} + 0.9330\left(\log\frac{D_{50}}{\delta}\right)^2 \; for \; \frac{D_{50}}{\delta} < 2 \tag{3.66}$$

$$\text{Log}\frac{F_g}{F_g^*} = \log 0.8 \; for \; \frac{D_{50}}{\delta} \geq 2 \tag{3.67}$$

Between these values lies the transition regime.

Procedure

For a given set of q, S, D_{50} and σ_g, the flow regime, depth of flow and roughness parameters may be predicted according to the following steps:

(i) The ratio of the median grain size to the theoretical thickness of the laminar sub-layer D_{50}/δ is computed.

(ii) The hydraulic radius R for both lower flow regime and upper flow regime is computed by using Eqs. (3.61) and (3.62).

(iii) The flow velocity is computed by using Eq. $U = \frac{Q}{B * R}$.

(iv) The roughness coefficient n is computed by Eq. (2.1).

(v) F_g is computed both for lower flow regime and upper flow regime using equation $F_g = \frac{U\sqrt{\rho}}{\sqrt{(\rho_s - \rho)gD_{50}}}$ with U = q/R.

(vi) F_g^* is computed using Eq.(3.63).

(vii) F_g/F_g^* and D_{50}/δ are used to locate upper limit of lower flow regime and lower limit of upper flow regime using Eqs. (3.64) to (3.67).

(viii) Flow regime thus determined is either lower, transition or upper.

D. White et al. (1987) method

White et al. (1987) restricted their attention to data for which the Froude number was less than 0.8. This restriction has not prevented others from attempting to use the data for Froude numbers larger than 0.8, with success. White et al. (1987) found that there exists a relationship between the total shear mobility F_{fg} and the effective shear mobility F_{gr}. The form of relationship varies with sediment size D_*. The dimensionless general sediment mobility is given by

$$F_{gr} = \frac{(U_*)^m}{\sqrt{[gD_{35}(s-1)]}}\left[\frac{U}{\sqrt{32}\log(10d/D_{35})}\right]^{1-m} \tag{3.68}$$

and sediment mobility for fine grain is given by

$$F_{fg} = \frac{U_*}{\sqrt{gD_{35}(s-1)}} \tag{3.69}$$

Sediment motion occurs only if F_{gr} exceeds the threshold value P. For the region F_{gr} greater than P, it is found that there is a linear relationship between F_{gr} and F_{fg} for each value of D_*, given by

$$\frac{F_{gr}-P}{F_{fg}-P} = Func(D_*) \tag{3.70}$$

White et al. (1987) suggested the following equation for F_{gr} for lower flow regime

$$\frac{F_{gr}-P}{F_{fg}-P} = 1-0.76\left\{1-\frac{1}{\mathrm{Exp}[(\log D_*)^{1.7}]}\right\} \tag{3.71}$$

and the following equation for F_{gr} for upper flow regime

$$\frac{(F_{gr}-P)+0.07(F_{gr}-P)^4}{F_{fg}-P} = 1.07-0.18\log(D_*) \tag{3.72}$$

The velocity of flow can be obtained both for lower and upper flow regimes using Eq. (3.68) with

$m = 0.00$ and $p = 0.17$ for $D_*>60$

and

$m = 1.0 - 0.56 \text{ Log } D_*$ and $p = 0.23/(D_{gr})^{0.5} + 0.14$ for $1<D_*<60$.

The use of two separate relationships for lower and upper regimes create problem of determining the appropriate regime to use in particular circumstances.

White et al. suggested that the following criteria be used:

$U_E^L + U_E^U < 0.22$ lower flow regime (3.73)

$U_E^L + U_E^U > 0.22$ upper flow regime (3.74)

where,

$U_E^L = U_E$ for velocity U computed for lower flow regime

$U_E^U = U_E$ for velocity U computed for upper flow regime

and $U_E = \dfrac{US}{(gv)^{1/3} D_*}$

Procedure

For a given set of R, D_{35}, g and S, the velocity of flow U and the flow regime may be computed using the following steps:

(i) Sediment mobility for fine grains F_{fg} is calculated using Eq. (3.69).

(ii) General sediment mobility F_{gr}, for lower flow regime is calculated using Eq. (3.71).

(iii) The lower regime mean velocity U is calculated using Eq. (3.68).

(iv) The dimensionless unit stream power U_E is calculated for lower regime flow using $U_E = \dfrac{US}{(gv)^{1/3} D_*}$.

(v) Using Eq.(3.72) the general sediment mobility F_{gr} for upper flow regime is calculated.

(vi) The upper flow regime mean velocity of flow U is calculated using Eq. (3.68)

(vii) As in step (iv), the value of U_E is calculated for upper flow regime.

(viii) If the summation of U_E for both lower and upper flow regimes is greater than 0.22, then the velocity of flow is that of the upper flow regime.

(ix) If the summation of U_E is less than 0.22, then the velocity of flow is that of lower flow regime.

(x) The overall friction factor f is computed by Eq. (2.7) using computed flow velocity in steps (iii) or (iv).

E. Karim (1995) method

Karim (1995) proposed a relation for determining the Manning coefficient that accounts for the effect of bed configuration directly on hydraulic resistance for flows in alluvial channels.

From physical considerations combined with dimensional analysis, a friction factor relationship is expressed as

$$\frac{U}{\sqrt{g(s-1)D_{50}}} = \Phi\left(\frac{d}{D_{50}}, S, Q_s^*, \lambda\right) \tag{3.75}$$

where,

Q_s^* = a dimensionless sediment discharge parameter,

λ = dimensionless parameter related to bed form

φ = a function to be determined

The bed configuration parameter λ is expressed as a ratio f/f_0, where f is the Darcy-Weisbach friction factor (including contribution due to bed forms) and f_o is the friction factor due to grain roughness only. The velocity relation developed by Karim (1995) is

$$\frac{U}{\sqrt{g(s-1)D_{50}}} = 6.683\left(\frac{d}{D_{50}}\right)^{0.626} S^{0.503}\left(\frac{f}{f_0}\right)^{-0.465} \tag{3.76}$$

and the equation is similar to the familiar Manning equation

$$U = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

For natural sediment (s = 2.65) the suggested relation for n is given by

$$n = 0.037\,(D_{50})^{0.126}\,(f/f_0)^{0.465} \qquad (D_{50} \text{ in meters}) \tag{3.77}$$

The central problem in alluvial flow resistance depends on accurate formulation of relative dimensions for different bed configurations that occur during different stages of flow, and relating them to hydraulic resistance. Karim (1995) developed a relation for relative dune height. Fig.3.13 shows a plot of H/d as a function of U_*/w_s.The difficulty in formulating a relation for H/d is apparent from the scatter of data points in Fig.3.13. The best-fit curve through the data points in Fig.3.13 can be expressed by the following equations:

$$\frac{H}{d} = -0.04 + 0.294\left(\frac{U_*}{w_s}\right) + 0.00316\left(\frac{U_*}{w_s}\right)^2 - 0.0319\left(\frac{U_*}{w_s}\right)^3 + 0.00272\left(\frac{U_*}{w_s}\right)^4 \tag{3.78}$$

for $0.15 < \frac{U_*}{w_s} < 3.64$, and

$$\frac{H}{d} = 0 \tag{3.79}$$

for $\frac{U_*}{w_s} < 0.15$ or $\frac{U_*}{w_s} > 3.64$

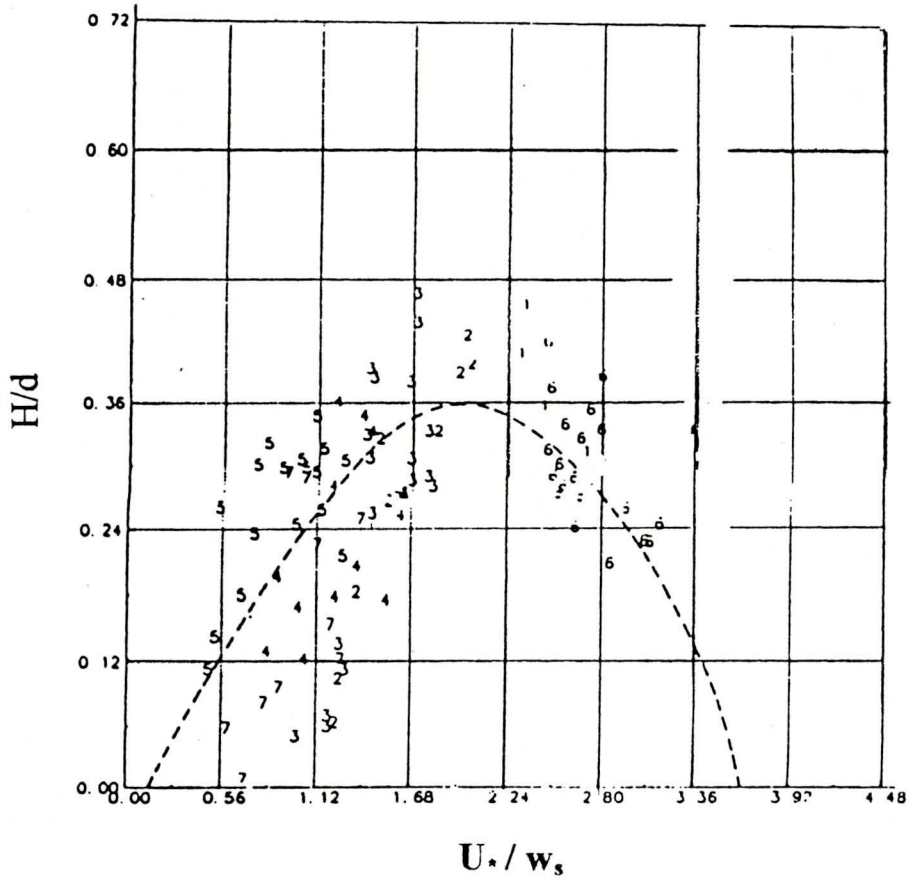

Fig.3.13 Relative bed form height as function of U_*/w_s (after Karim, 1995)

A linear relation is assumed between the friction factor ratio f/f_0 and relative bed form height, H/d and is given by

$$\frac{f}{f_0} = 1.20 + 8.92\left(\frac{H}{d}\right) \tag{3.80}$$

Combination of Eqs. (3.78) and (3.79) with Eq. (3.80) yields

$$\frac{f}{f_0} = 1.20 + 8.92[-0.04 + 0.294(\frac{U_*}{w_s}) + 0.00316(\frac{U_*}{w_s})^2 - 0.0319(\frac{U_*}{w_s})^3 + 0.00272(\frac{U_*}{w_s})^4] \tag{3.81}$$

when $0.15 < \frac{U_*}{w_s} < 3.64$ and

$$\frac{f}{f_0} = 1.20 \tag{3.82}$$

when $\frac{U_*}{w_s} < 0.15$ or $\frac{U_*}{w_s} > 3.64$

Equations (3.81) and (3.82), which determine the parameter f/f_0, reflect the role of bed configuration on hydraulic resistance.

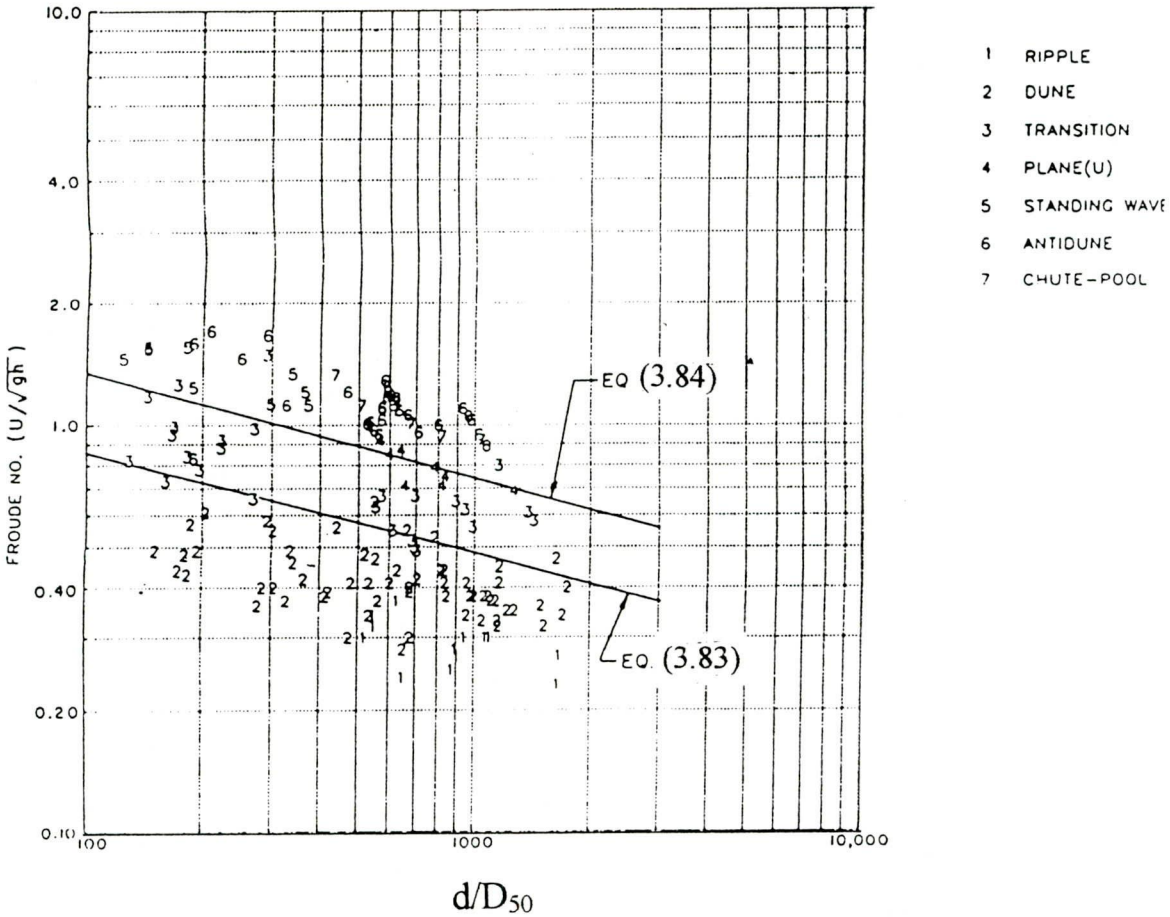

Fig.3.14 Froude number criteria for bed regime prediction (Karim, 1995)

The lack of available bed-form height data during transitional bed regime (i.e., after the bed-form height begins to decrease at $U_* / w_s \approx 2.0$ until it is washed out completely at $U_* / w_s \approx 3.6$) is evident in Fig.3.13. The following special procedure is used for the transition regime. A graphical relation for bed regime prediction is shown in Fig. 3.14. which plots experimental

data in the form of Froude number versus relative depth $\left(\frac{d}{D_{50}}\right)$. The two lines in Fig.3.14. which separate the transition regime from lower and upper flow regimes, are given respectively by

$$Fr_t = 2.716\left(\frac{d}{D_{50}}\right)^{-0.25} \tag{3.83}$$

and

$$Fr_u = 4.785\left(\frac{d}{D_{50}}\right)^{-0.27} \tag{3.84}$$

where Fr_t = Froude number at the beginning of transition regime and Fr_u = Froude number at the beginning of upper regime. In this analysis Eqs. (3.83) and (3.84) are used to determine bed regime as follows:

Lower regime	: $Fr<Fr_t$
Transition regime	: $Fr_t<Fr<Fr_u$
Upper regime	: $Fr>Fr_u$
where,	Fr = Froude number of flow.

For the transition regime, it is assumed that H/d is maximum (0.20) at $Fr = Fr_t$ and decreases to zero at $Fr = Fr_u$, i.e.

$$\frac{H}{d} = 0.20\left(\frac{Fr_u - Fr}{Fr_u - Fr_t}\right) \tag{3.85}$$

For the transition regime, H/d is determined by the minimum values given by Eqs. (3.78). (3.79) and (3.85) while H/d is determined for lower flow regime by Eqs. (3.78) and (3.79). For upper flow regime, H/d = 0.

Procedure

For given values of discharge Q, energy slope S and sediment size D_{50}, the values of hydraulic radius R and the mean velocity of flow U are computed in an iteration scheme as follows:

(i) A water surface elevation is assumed.

(ii) The values of hydraulic radius R, flow area A, mean velocity of flow U and the Froude number F_r are computed.

(iii) The values of U_*, Fr_t and Fr_u are calculated using Eqs. (2.15),(3.83) and (3.84), respectively.

(iv) If $Fr < Fr_t$, then H/d is calculated using Eq.(3.78) or (3.79).

(v) For $Fr_t < Fr < Fr_u$, H/d is assumed as the minimum of the values given by Eqs.(3.78),(3.79) and (3.85). H/d = 0 for $Fr > Fr_u$.

(vi) Using Eq. (3.80) the value of f/f_o is computed, value of n is computed by using Eq.(3.77) and U_2 by Eq (3.76) assuming $U=U_2$.

(vii) If U_2 is not equal to U, within the specific criteria, another water surface elevation is assumed and the steps from (ii) to (vi) are continued until $U_2 = U$ is satisfied.

(viii) The overall friction factor f is computed by using equation Eq. (2.7)

3.4 Prediction Accuracy

The reliability for predicting friction coefficients and other hydraulic parameters for flow in alluvial channels were tested by different methods by different workers. Van Rijn (1984, 1993) compared his method with others, on the basis of percentage of predicted depth and Chezy's C that lies within ±10%, ±20% and ±30% error ranges. Brownlie (1983) verified his proposed method with other several methods graphically and numerically. Karim (1995) tested the accuracy of his method on the basis of the mean normalized error (MNE) and the percentage of error on the predicted depth and velocity. Karim (1995) also proposed that the prediction accuracy could also be tested by the standard deviation (SD) of the MNE, which gives an indication of the range of errors for the predicted quantities in each data set. When both the MNE and the SD of MNE of a predicted parameter for a particular method will be low than the method is said good.

In this study, accuracy of different methods is determined on the basis of

1. MNE of the predicted roughness coefficients, depth and velocity.
2. Standard deviation of the MNE of the roughness coefficients, depth and velocity.
3. Graphical (bar chart) comparison of calculated and predicted manning's n and Chezy's C.
4. Graphical variation of measured and predicted Manning's n obtained by different methods.

The accuracy of prediction measured by the mean normalized error (MNE) in predicted depths or velocities expressed in percent is defined as

$$MNE = \frac{100}{N} \sum_{i=1}^{N} \frac{X_{pi} - X_{mi}}{X_{mi}} \tag{3.97}$$

where,

X_{pi} = predicted depths (or velocities or roughness parameters) for ith flow

X_{mi} = measured/calculated depths (or velocities or roughness parameters) for ith flow

N = total number of data of a station.

And the SD of the MNE= $\sqrt{\frac{\sum_{i=1}^{N} (Xi - \overline{X})^2}{N}}$

Where,

Xi = MNE of each data

$\overline{X}$ = Mean value of MNE

N = total number of data.

CHAPTER 4

DATA COLLECTION AND ANALYSIS

4.1 Introduction

It is stated in chapter 1 that the main objective of the present study is to determine the applicability of some alluvial roughness predictors for the rivers Gumti, Kushiyara and Mohananda and to identify the method (s) that seem to predict roughness characteristics of the above mentioned rivers satisfactorily. In this chapter, a brief description of the Gumti, Kushiyara and Mohananda rivers is presented first, thereafter, data required for the study, sources of data, collection of data etc. are described.

4.2 Brief Description of the Rivers

4.2.1 The Gumti river

The Gumti is a flashy river. It rises in the hill of Tipura in India and enters the plains just before crossing into Bangladesh at Katakbazar , a few kilometers east of Comilla town. It meanders into north- westernly direction to Companiganj. About 1.5 km upstream of this village, the Buri Nadi river branches off from the Gumti river at Jibanpur and flows in a northern direction falling into the Titas river at Nabinagar. The Gumti drains into the Meghna river at Daudkandi. The catchment area of the river Gumti is about 2150 sq.km. The river length is 122 km from Katakbazer to Daudkandi.

4.2.2 The Kushiyara river

The Kushiyara is a major tributary of the Meghna, Bangladesh's largest river after the Brahmaputra and the Ganges. The Kushiyara bifurcates on meeting the Bangladesh border about 15 km east of Karimganj. Until it enters Bangladesh it is called the Barak. A channel of the main river follows the international boundary to the north to link up with the Surma. This channel takes off about one third of the Kushiyara flood flow, which represents about three-quarters of the total Surma flow. From this bifurcation the main Kushiyara continues

westwards, also along the international boundary to Karimganj. The Kushiyara is one of the two major upper branches of the Meghna river becoming, further downstream, the Kalni. It has a large catchment area in India in the region of 26000 sq. km. The Kushiyara drains a large area of north-south hills in west Manipur, north Mizoram and south Assam.

4.2.2 The Mohananda river

The Mohnanda is a major tributary of the Ganges river and rises near Mohaldiram in Darjeeling district in India. It forms the western boundary of Bangladesh at Tetulia and then again at Ranishankoil. Flowing through India, it again forms the boundary of Bangladesh near Bhadahat, then flows through Chapai Nawabganj and joins the Ganges near Godagari. The Punarbhaba joins the Mohananda river just south of Rohanpur. Between the Punarbhaba and the Mohananda rivers there are three other tributaries of the later river, the Tangon, the Kulik and the Nagor. The Mohananda is a meandering river; the average width of this river varying from 300 m to 600 m. The maximum and minimum flows of the Mohananda river is 12 m^3/s and 3660 m^3/s respectively.

4.3 Data Requirements

To predict roughness coefficients n, C and f the flow of the river is assumed as quasi-uniform and the required data are the observed discharge Q, the energy line slope S, the hydraulic radius R and the grain size of the bed materials should be known. Since the flow in an alluvial channel is assumed as quasi-uniform, data required calculating roughness parameters n, f and C are Q, R, S and grain size of the bed materials. The median particle size (D_{50}) is commonly used for the grain size. However, Einstein and Barbarrosa (1952) used D_{65}, White et al. (1987) used D_{35}, Haque and Mahmood (1983) used D_{84} and Van Rijn used D_{90} in their respective roughness predictors. Brownlie (1983) used geometric standard deviation that may be calculated from D_{16}, D_{50} and D_{84}.

Particle fall velocity w_s was used by Shen (1962) method which can be calculated if the kinematic viscosity of water ν and the acceleration due to gravity g are known. Water density ρ and particle density ρ_s are also used in the roughness prediction.

It can be concluded from above discussion that data required for this study are: discharge Q, energy line slope S, top width B, cross-sectional area A, kinematic viscosity of water ν, acceleration due to gravity g, density of water ρ, density of sediment ρ_s and particle sizes D_{16}, D_{35}, D_{50}, D_{65}, D_{84} and D_{90}. Since the energy line slope S is not available, there the water surface slope S_w is taken equal to the energy line slope S.

4.4 Data Collection

The hydrological data such as the cross-sectional area, discharge, velocity, depth, width of the river etc. are collected of the station Jibanpur for the Gumti river, station Sherpur for the Kushiyara river and station Chapai Nawabganj for the Mohananda river from the Hydrology Directorate, Bangladesh Water Development Board (BWDB). The water surface slopes of the rivers Gumti and Mohananda are collected from BWDB and of the river Kushiyara is collected from River Survey Project (FAP-6). Grain size data for Gumti, Kushiyara and Mohananda rivers are collected from Surface Water Modelling Center (SWMC).

The hydrological data of the station Jibanpur for river Gumti are available from 12.02.81 to 27.11.96 (352 records), of the station Sherpur for river Kushiyara are available from 25.03.82 to 23.03.96 (349 records), of the station Chapai Nawabganj for river Mohananda are available 06.04.82 to 24.03.92 (288 records).

The Hydrology Division of BWDB measures discharges of the three selected rivers fortnighty. BWDB also collects sediment data at the stations Jibanpur and Sherpur of the rivers Gumti and Kushiyara, respectively. Sediment sample is not collected of the river Mohananda.

The grain size of the bed material and the water surface slope of stations Jibanpur for the Gumti river, of station Sherpur for the Kushiyara river and of station Chapai Nawabganj for the Mohananda river are shown in Table 4.1.

Table 4.1 Grain size of bed material and water surface slope of the river Gumti at Jibanpur, of Kushiyara at Sherpur and of Mohananda at Chapai Nawabganj stations

Station	Grain size of the bed material in mm						Water surface slope
	D_{16}	D_{35}	D_{50}	D_{65}	D_{84}	D_{90}	
Jibanpur	0.100	0.129	0.151	0.176	0.220	0.240	0.000079
Sherpur	0.110	0.139	0.159	0.182	0.350	0.400	0.000022
Chapai Nawabganj	0.080	0.110	0.132	0.160	0.210	0.230	0.000110

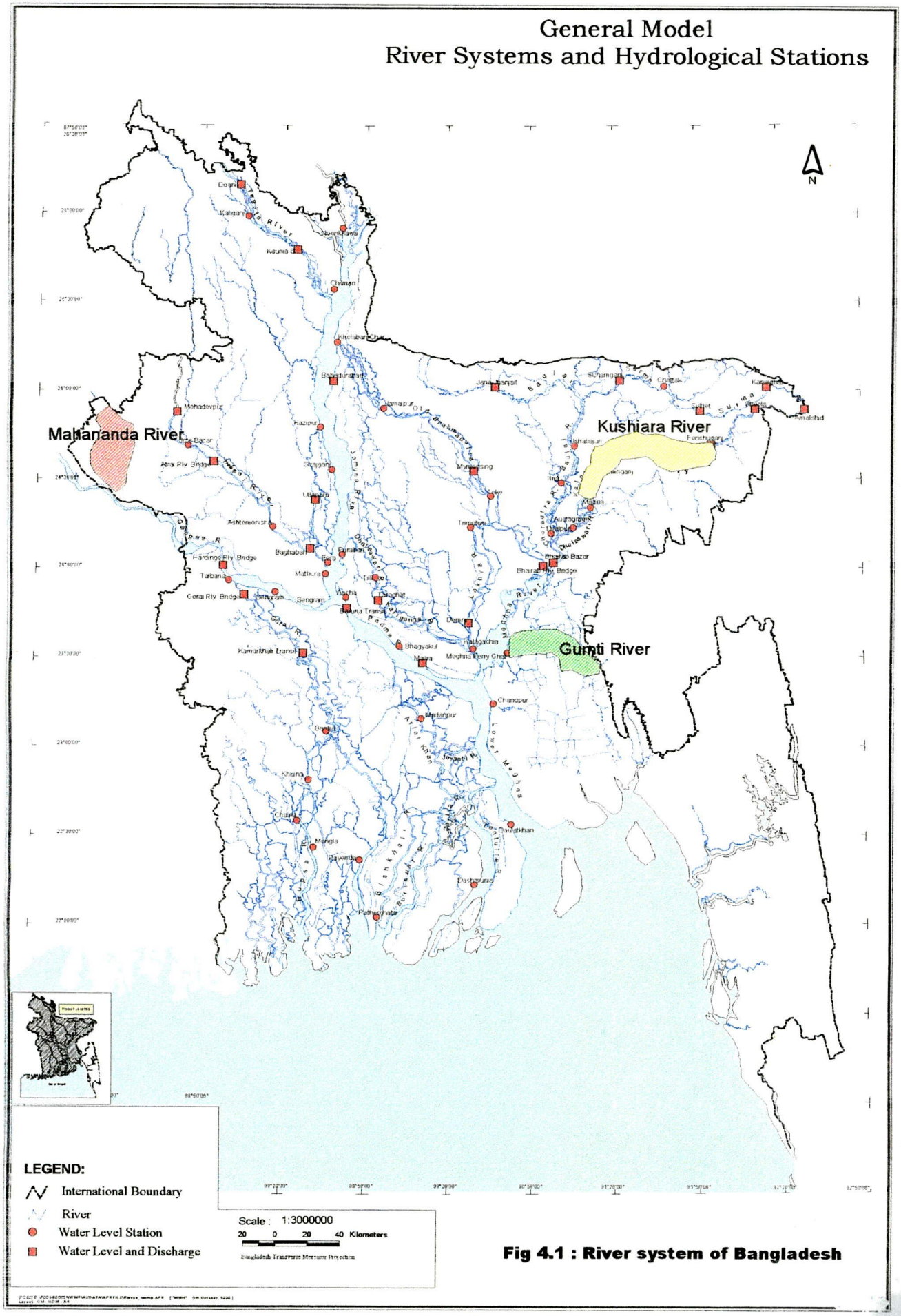

Fig 4.1 : River system of Bangladesh

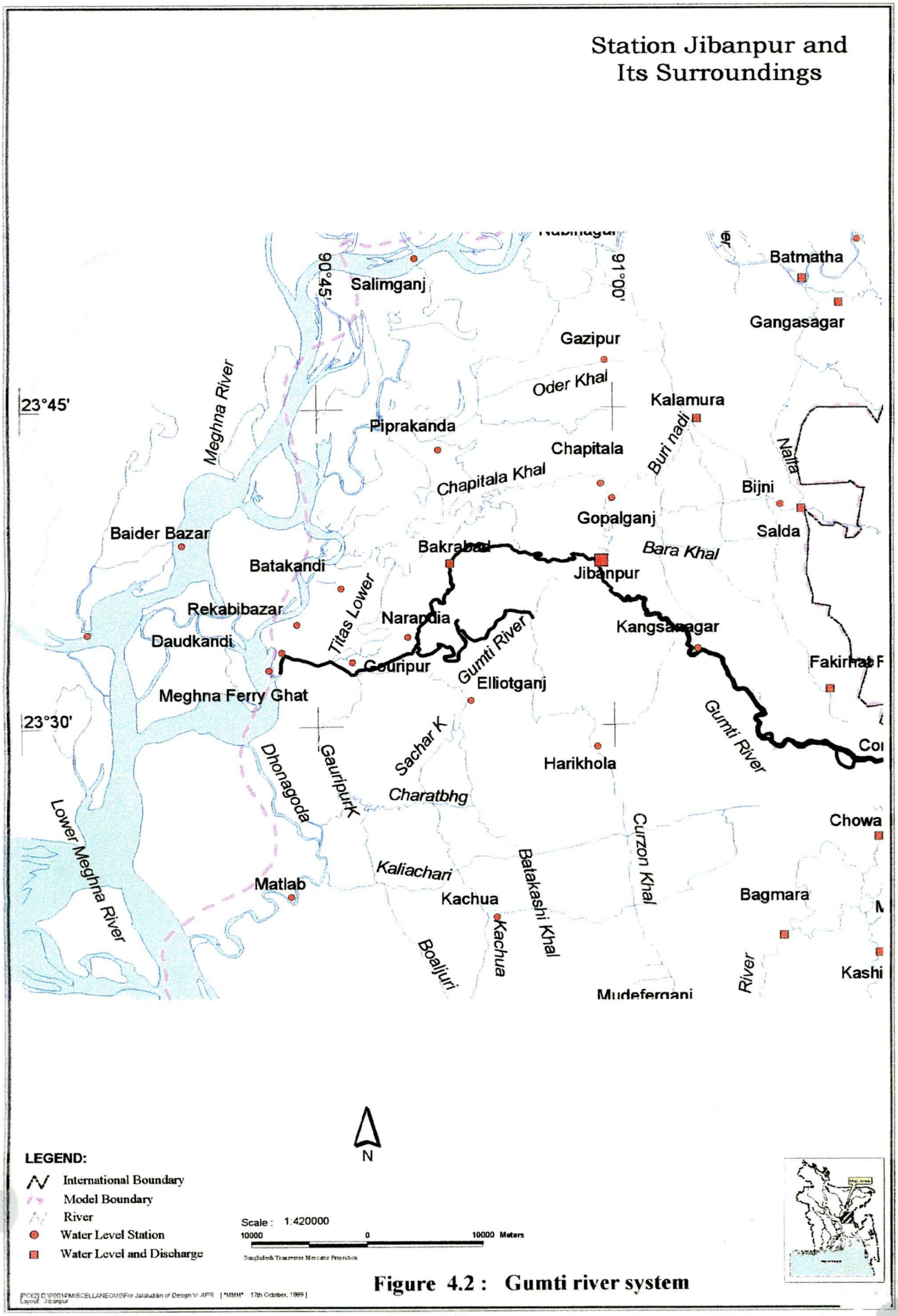

Figure 4.2 : Gumti river system

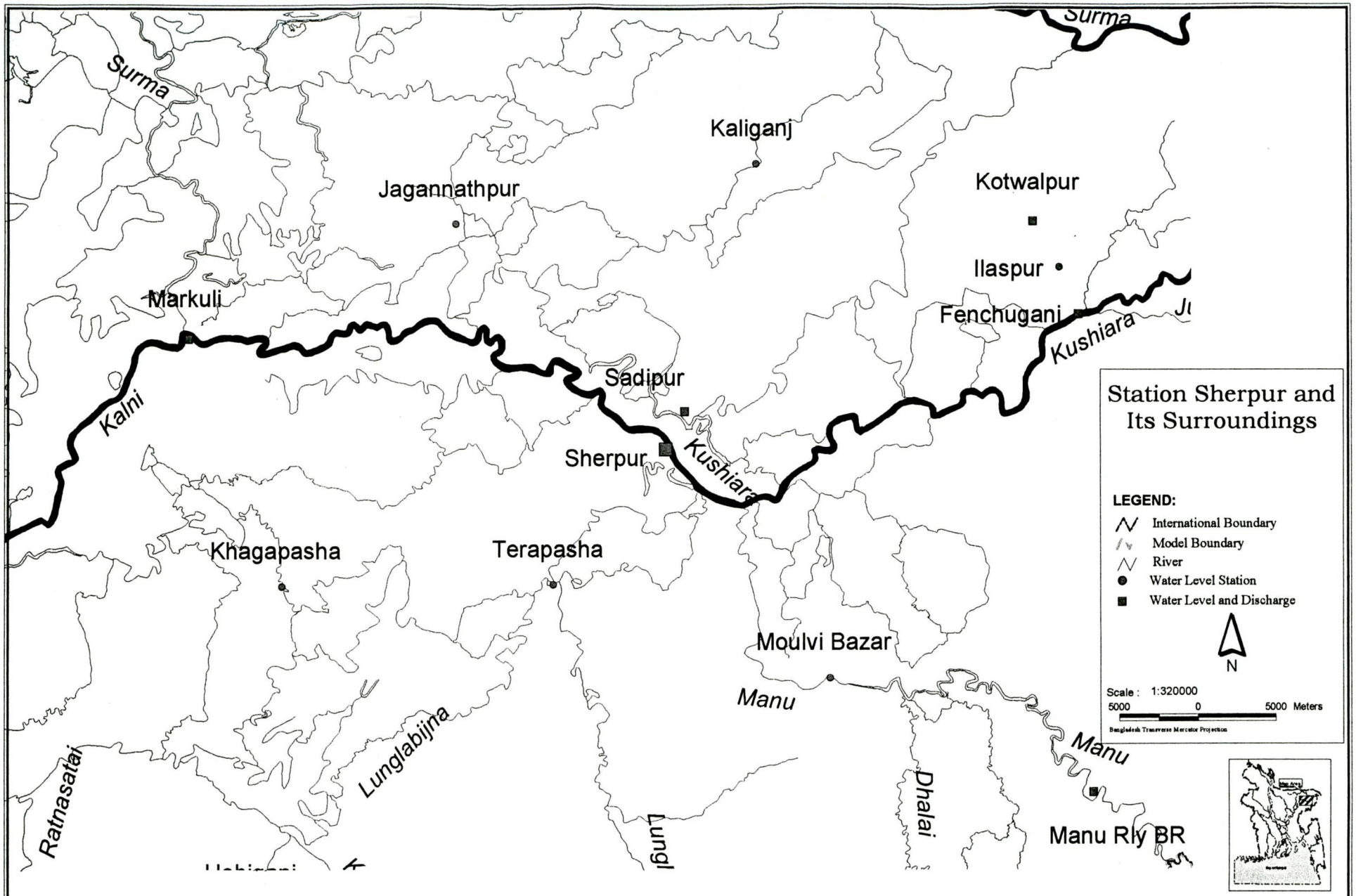

Figure 4.3 : Kushiara river system

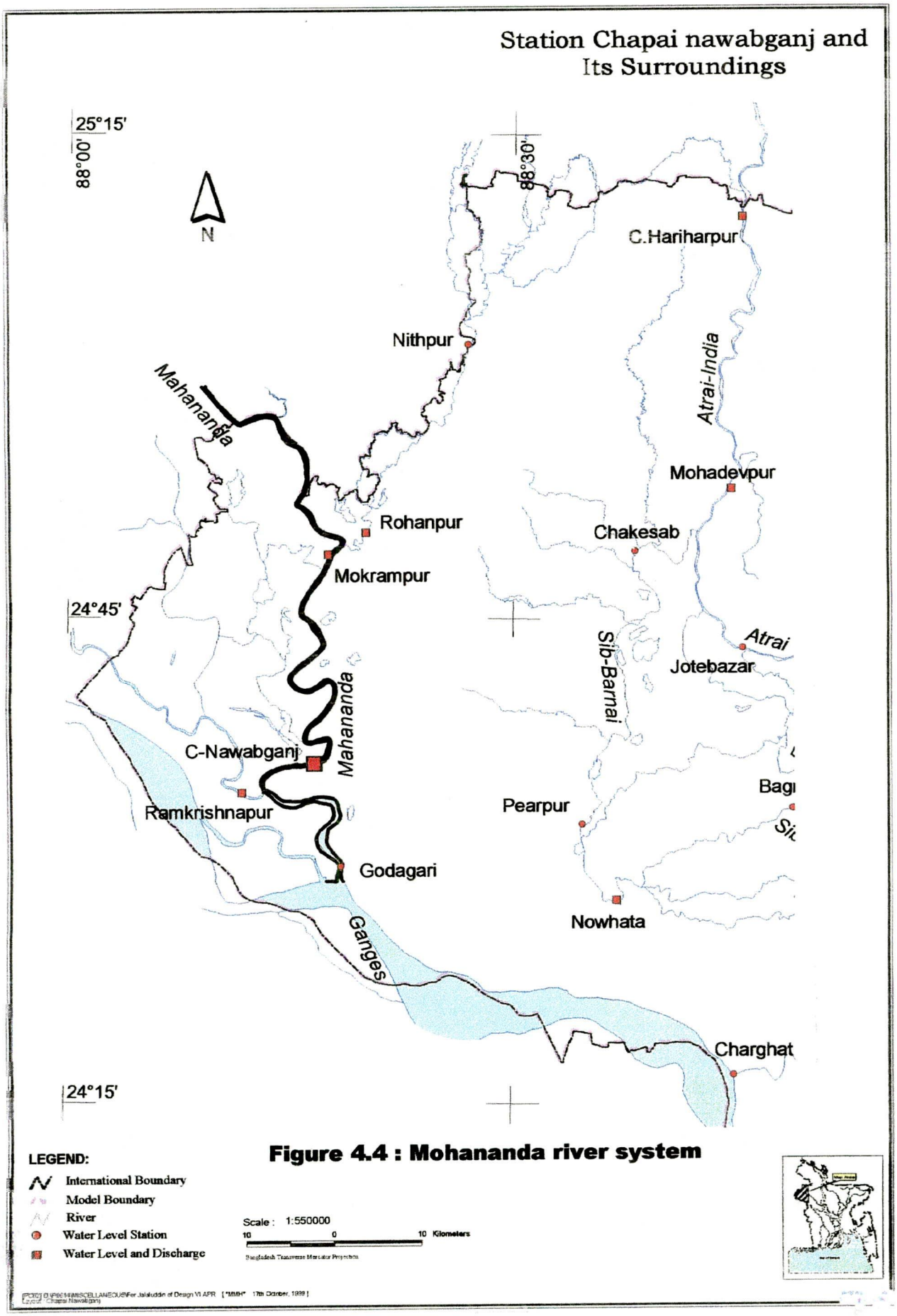

Figure 4.4 : Mohananda river system

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 Introduction

To determine the applicability of the roughness predictors in the Gumti, Kushiyara and Mohananda rivers, eleven roughness predictors are used. These methods are used to predict the roughness coefficients of the above mentioned rivers by using FORTRAN-77 computer Programme. Out of these eleven roughness predictors some methods predict depth of flow, some predict velocity and some can predict both depth and velocity. Some methods can predict the roughness coefficients directly, without predicting depth or velocity.

In this study some methods are based on total roughness and some are based on division of roughnesses. The Manning, Chezy and Darcy-Weisbach formulae give only the total roughness, and hence comparison with these formulae can only be made on the basis of total roughness. In the following sections the roughness parameters predicted by different methods are compared with the values obtained by direct calculation using the Manning, Chezy and Darcy-Weisbach formulae. For brevity, the comparison is made on the basis of Manning's n only. The numerical values of n, C and f obtained by different methods indicate that if Manning's n is compatible, Chezy and Darcy-Weisbach formulae also give compatible results. That is why detailed comparisons on the basis of Manning's n have been done. Chezy's C and the Darcy-Weisbach friction factor f have not been compared for all the cases.

5.2 Roughness Parameters by Manning, Chezy and Darcy-Weisbach formulae

The roughness parameters n, C and f are calculated by the Manning, the Chezy and the Darcy-Weisbach formulae, respectively, for all the data sets of the Gumti, Kushiyara and Mohananda rivers and are termed as **calculated** roughness parameters. After analyses the maximum values of the roughness parameters n and f and the minimum value of C and vice versa of the rivers Gumti, Kushiyara and Mohananda are given in Table 5.1.

Table 5.1 Maximum & minimum values of roughness coefficients for the Gumti, Kushiyara and Mohananda rivers

Name of the river	Maximum and minimum value of the roughness coefficients					
	For low discharge			For high discharge		
	n_{max}	f_{max}	C_{min}	n_{min}	f_{min}	C_{max}
Gumti	0.068	0.437	13.40	0.023	0.024	57.40
Kushiyara	0.096	0.587	11.56	0.015	0.008	96.98
Mohananda	2.316	0.038	45.29	0.081	0.009	92.34

The values of n and f decreases with increasing discharge while the values of C increases with increasing discharge. For the Gumti river the ratios of maximum to minimum values of n, f and C values are 2.96, 18.21 and 4.28. For the Kushiyara river these ratios are 6.4, 73.4 and 8.39 and for the Mohananda river the ratios are 28.6, 4.2 and 34.67.

The variation of Manning's n, Chezy's C, Darcy-Weisbach f, grain roughness n′, and form roughness n″ with discharges are shown in the Figs. 5.37, 5.38 and 5.39 for the rivers Gumti, Kushiyara and Mohananda, respectively. The Strickler formula (Strickler, 1923) as discussed in Art. 2.3.5, is used to calculate the values of Manning's grain roughness n′. There is a consensus among the many researchers that this formula gives the grain roughness value n′ for both rigid and mobile bed channels, although there is considerable controversy regarding the value of K_s in this formula. Various researchers suggested values of K_s ranging from D_{50} to 5.1 D_{84} (as mentioned in Art. 2.3.7). Hopefully, the numerical value of n′ is not very much sensitive to K_s since $n' \propto K_s^{1/6}$. The range of grain roughness n′ by this formula is from 0.011 to 0.015 for the Gumti river, from 0.011 to 0.017 for the Kushiyara river and from 0.011 to 0.015 for the Mohananda river. The form roughness n″ is obtained by subtracting the grain roughness n′ from total roughness n.

Calculated values of the roughness coefficients n, C and f are reasonable limit and within the normal range for the Gumti and the Kushiyara rivers. But for the Mohananda river, these values are

out of normal range (Chow, 1959, Table 5.6). Therefore, the data of the Mohananda river don't seem reliable.

5.3 Performance of the Roughness Predictors

5.3.1 General

A brief discussion on results of calculated and predicted roughness parameters and depth are given in Art.5.4. Using water surface slope and measured depth and velocity, the values of roughness parameters n, C and f are calculated using Manning, Chezy and Darcy-Weisbach formulae, respectively. Out of the eleven roughness predictors that have been used, methods by Engelund and Hansen (1967), Brownlie (1983) and Karim (1995) predict both depth and velocity. Methods by Einstein and Barbarossa (1952), Shen (1962) and Van Rijn (1993) predict only depth; methods by Simons and Richardson (1966), Garde and Ranga Raju (1970) and White et al. (1987) predict only velocity; and those by Haque and Mahmood (1983) and Shen et al. (1990) predict neither depth nor velocity but they predict the roughness parameters directly. These predicted roughness coefficients are then compared with the calculated roughness coefficients. The Mean Normalised Error (MNE) and the standard deviation (SD) of the MNE of the predicted n, C, f, depth and velocity with the corresponding calculated values are used to evaluate the applicability of the roughness predictors in the particular river (s).

The MNE and the standard deviation of the MNE of the predicted parameters are shown in Tables 5.3, 5.4 and 5.5 and in the Figs. 5.43, 5.44, 5.45, 5.46, 5.47 and 5.48 for the rivers Gumti, Kushiyara and Mohananda.

5.3.2 Roughness parameters predicted by different methods based on division of roughness

5.3.2.1 Einstein and Barbarossa (1952) method

The Einstein and Barbarossa (1952) method seems to predict the roughness coefficients of the Gumti river with good accuracy. The range and variation of the predicted Manning's n and Chezy's

C are quite close to the calculated values for Gumti river. For Gumti river, the range of calculated Manning's n by Manning formula varies from 0.068 to 0.023 and the corresponding predicted Manning's n by the Einstein and Barbarossa method varies from 0.052 to 0.019. Range of the calculated Chezy's C by Chezy's formula and corresponding range of predicted Chezy's C by this method varies from 13.40 to 57.40 and from 16.34 to 67.12, respectively. For the Gumti river the grain roughness remains constant for all discharges and this value is 0.011.

The Einstein and Barbarossa (1952) method predicts roughness coefficients satisfactorily for the Kushiyara river also. The range and variation of the Manning's n and Chezy's C are close to the values obtained by the Manning and the Chezy formulae. The range of predicted Manning's n and Chezy's C are 0.078 to 0.022 and 13.50 to 71.47, respectively, corresponding to the low and high discharges. The grain roughness varies from 0.011 to 0.014 for the Kushiyara river.

For Mohananda river, the measured data do not seem to be reliable and the numerical values of Manning's n calculated by the Manning formula are out of normal range. The range of Manning's n by Manning formula and those predicted by Einstein and Barbarossa (1952) vary from 2.316 to 0.081 and from 0.166 to 0.015, and the corresponding Chezy's C is from 0.52 to 18.03 and from 4.63 to 77.83, respectively. The Einstein and Barbarossa (1952) method predicted Manning's n close to the normal range. For the Mohananda river the grain roughness values varies from 0.011 to 0.012 for ripple and dune bed configurations, respectively.

The mean normalised error (MNE) and the standard deviation (SD) of MNE of Manning's n is 1.92% and 32.30% for Gumti river, 38.1% and 38.0% for the Kushiyara river and - 99.85% and 0.10% for Mohananda river.

The MNE and the standard deviation of MNE of Chezy's C is 5.16% and 27.30% for Gumti river, -16.2% and 23.0% for the Kushiyara river and 556.68% and 185.70for Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.1, 5.2 and 5.3, respectively.

5.3.2.2 Shen (1962) method

Like Einstein and Barbarossa (1952) method, the Shen (1962) method also predicts the roughness coefficients with good accuracy, but the values by Shen (1962) method are slightly higher than the values obtained by direct calculation. The trend of variation of the predicted roughness coefficients by Shen (1962) method is similar to that of direct calculation. The respective range of Manning's n and Chezy's C are 0.076 to 0.024 and 12.30 to 55.80 for the Gumti river; 0.080 to 0.030 and 11.20 to 57.70 for the Kushiyara river; and 0.069 to 0.016 and 8.90 to 73.10 for the Mohananda river. For the Mohananda river the values of Manning's n predicted by the Shen (1962) are very close to the corresponding values predicted by the Einstein and Barbarossa method.

The predicted grain roughness value 0.011 is constant for all discharges for the Gumti river. For Kushiyara river the value of grain roughness increases with increasing discharge. For low discharge the value of grain roughness is 0.011, for medium and high discharges it is 0.012 and 0.013, respectively.

The MNE and the standard deviation of MNE of Manning's n is 45.00% and 45.70% for Gumti river, 130.85% and 47.70% for Kushiyara river and – 83.54% and 7.40% for Mohananda river.

The MNE and the standard deviation MNE of Chezy's C are – 19.37% and 43.50% for Gumti river, -38.73% and 13.50% for Kushiyara river and 472.40% and 277.00% for Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.4, 5.5 and 5.6, respectively.

5.3.2.3 Engelund and Hansen (1967) method

The results obtained by the Engelund and Hansen (1967) method seem to be very poor for all the three rivers. The values of the predicted roughness coefficients are nearly constant for all flow regimes. The variations of the Manning's n and Chezy's C corresponding to low and high discharges are 0.013 to 0.015 and from 85.1 to 85.1 for the Gumti river; from 0.014 to 0.015 and from 95.1 to 97.5 for the Kushiyara river; and from 0.012 to 0.026 and from 56.7 to 83.0 for the

Mohananda river, respectively. The value of grain roughness n′ is 0.014 which remains constant for all discharges and for all the three rivers.

The MNE and the standard deviation of MNE of Manning's n is –60.30% and 2.80% for Gumti river, -43.30% and 29.30% for Kushiyara river and -94.70% and 5.20% for Mohananda river.

The MNE and the standard deviation MNE of Chezy's C are 197.70% and 154.00% for Gumti river, 152.60% and 173.30% for Kushiyara river and 3297.00% and 2922.00% for Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.7, 5.8 and 5.9, respectively.

5.3.2.4 Haque and Mahmood(1983) method

Bed form dimensions by Vajda (1990) method

Predicted values of roughness coefficients by applying the Haque and Mahmood (1983) method to the Gumti river seem to be of poor quality. The values of Manning's n are low for low discharges and high for high discharges, a behavior just reverse to the normal trend. In plane or ripple beds the Manning's n varies from 0.035 to 0.015 and in dune or anti-dune beds the variation of Manning's n is from 0.05 to 0.040. In the transition regime the values of Manning's n remain constant at 0.050. So in plane or ripple beds the predicted Manning's n values are lower than that of calculated values by Manning formula where as in dune or anti-dune beds the predicted Manning's n values are higher than of corresponding calculated values.

The qualities of the predicted roughness coefficients for the Kushiyara and Mohananda rivers seem to be quite better than that of Gumti river. For both the Kushiyara and Mohananda rivers, the predicted Manning's n values are lower for low discharges than the corresponding values obtained the by Manning formula, but for medium to high discharges the predicted n values are very close to the respective calculated values. The predicted Manning's n and Chezy's C vary from 0.035 to

0.018 and from 31.12 to 81.16 for the Kushiyara river and from 0.043 to 0.017 and 27.94 to 85.50 for the Mohananda river.

The value of grain roughness n′ is 0.014 for lower flow regime, and 0.015 for both the transition and upper regimes for all the three rivers.

The MNE and the standard deviation of MNE of Manning's n are -16.55% and 53.00% for Gumti river, - 5.61% and 40.30% for Kushiyara river and - 90.50% and 5.40% for Mohananda river.

The MNE and the standard deviation of MNE of Chezy's C are 84.42% and 134.10% for Gumti river, 24.96% and 55.00% for Kushiyara river and 1577% and 1634% for Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.10, 5.11 and 5.12, respectively.

5.3.2.5 Shen et al. (1990) method

Bed form dimensions by Allen (1962) method

The Shen et al. (1990) method predicts inconsistent roughness coefficients for the Gumti river. Although the maximum and minimum Manning's n values predicted by Shen et al. (1990) method are 0.098 and 0.024, there are many values which lie in between 0.008 and 0.003 in all flow regimes. The range of Chezy's C of Gumti river is from 29.20 to 57.00.
For the Kushiyara and the Mohananda rivers the quality of the predicted roughness coefficients is good for transition and upper flow regimes; but for low flow regime it predicts low values of Manning's n.

For the Kushiyara river, the range of Manning's n and Chezy's C by direct calculation and by Shen (1990) method are 0.035 to 0.012 and 60.50 to 81.00, respectively. For the Mohananda river these ranges are from 0.027 to 0.008 and from 46.60 to 172.10, respectively.

The value of grain roughness n′ is 0.011 in lower regime, 0.012 in transition regime and 0.013 to 0.014 in upper regime for all the three rivers.

The MNE and the standard deviation of MNE of Manning's n is -37.16% and 46.00% for Gumti river, -12.74% and 35.70% for Kushiyara river and - 94.42% and 3.20% for Mohananda river.

The MNE and the standard deviation of MNE of Chezy's C is 162.84% and 219.00% for Gumti river, 31.96% and 50.50% for Kushiyara river and 2619.66% and 2312% for Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.13, 5.14 and 5.15, respectively.

5.3.2.6 Van Rijn (1993) method

Prediction of roughness coefficients by the Van Rijn (1993) method seems to be very poor for all the three rivers. The Manning's n varies from 0.011 to 0.024 for Gumti river, 0.013 to 0.026 for Kushiyara river and 0.012 to 0.026 for the Mohananda river, respectively, at corresponding to low and high discharges. Such variation is contrary to the normal trend. The range of the corresponding Chezy's C is from 50.7 to 56.4 for Gumti river, from 52.88 to 63.63 for Kushiyara river and from 56.70 to 83.00 for the Mohananda river, respectively.

In Gumti river the value of grain roughness n′ is 0.010 for the lower flow regime and 0.012 for the upper flow regime. In Kushiyara river the grain roughness value is 0.013 for the lower flow regime and 0.015 for the upper flow regime. For the Mohananda river the grain roughness value varies from 0.010 to 0.013.

The MNE and the SD of MNE of Manning's n are -23.46% and 36.80% for the Gumti river, 0.77% and 59.50% for the Kushiyara river and -92.60% and 7.20% for the Mohananda river, respectively.

The MNE and the SD of MNE of Chezy's C is 63.07% and 117.40% for the Gumti river, 58.25%

and 120.00% for the Kushiyara river and 1732.00% and 1350.00% for the Mohananda river, respectively.

The comparison between the calculated and the predicted Manning's n values of the Gumti Kushiyara and Mohananda rivers are shown in Figs. 5.16, 5.17 and 5.18, respectively.

5.3.3 Roughness parameters predicted by different methods based on overall roughness

5.3.3.1 Simon and Richardson (1966) method

The Simons and Richardson (1966) method predicts roughness coefficients seem to be comparatively better results than other four methods based on overall roughness prediction. The method predicts higher Manning's n value than the Manning's n values obtained by the Manning formula at low discharge, and lower n value at high discharge for Gumti river. The predicted maximum and minimum Manning's n are 0.073 and 0.017 but there is a distinct discontinuity at the transition regime as shown in Fig. 5.34. For the Kushiyara river the range of the predicted Manning's n is from 0.070 to 0.045 at low and high discharges. So the method over-predicts Manning's n at high discharge but at low discharge the prediction is within the normal range. The range of Chezy's C is from 32.30 to 20.50, respectively. In Mohananda river, the predicted Manning n value increases with increasing discharges which is reverse to the normal trend. The variation of the predicted Manning's n is from 0.073 to 0.047 for ripple and from 0.014 to 0.017 for dune bed in Gumti river and the variation remains almost constant when the flow is in the transition regime.

The Simons and Richardson (1966) method is therefore not applicable both for Gumti, Kushiyara and Mohananda rivers.

The MNE and the standard deviation of MNE of Manning's n are 48.9% and 9.29% for Gumti river, 84.40% and 82.00% for the Kushiyara river and –92.89% and 5.5% for Mohananda river.

The MNE and the SD of MNE of Chezy's C is 6.3% and 92.3% for Gumti river, -28.92% and 58.00% for the Kushiyara river and 2909% and 2998% for Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.19, 5.20 and 5.21, respectively.

5.3.3.2 Garde and Ranga Raju (1977) method

The quality of the predicted values of Manning's n, Chezy's C and other hydrological parameters by the Garde and Ranga Raju (1977) method seems to be very poor. The method does not properly respond to the variation in actual flow condition of the selected alluvial rivers. The predicted Manning's roughness coefficient n for all the three rivers are nearly constant. The maximum and minimum Manning's n values are 0.020 and 0.020 corresponding to low and high discharges, respectively for the Gumti river, 0.031 and 0.017 for the Kushyara river and 0.022 and 0.021 of the Mohananda river.

The MNE and the standard deviation MNE of Manning's n is –34.07% and 27.80% for the Gumti river, -38.22% and 30.30% for the Kushiyara river and –92.56% and 5.30% for the Mohananda river.

The MNE and the SD of MNE of Chezy's C is 83.70% and 97.30% for Gumti river, 131.25% and 159.00% for Kushiyara river and 2268.47%and 194.00% for Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.22, 5.23 and 5.24, respectively.

5.3.3.3 Brownlie (1983) method

The Brownlie (1983) method could not predict reasonable values of the roughness coefficients for any of the three rivers. This method predicted maximum value of Manning's n at high discharges and minimum value of Manning n at low discharges, which seem to be unusual. The variation of predicted Manning's n and Chezy's C from low to high discharges respectively are from 0.017 to 0.026 and 47.63 to 52.27 for the Gumti river, from 0.018 to 0.026 and 54.92 to 59.10 for the

Kushiyara river and from 0.018 to 0.014 and 45.29 to 92.34 for the Mohananda river.

The MNE and the SD of MNE of Manning's n are – 34.26% and 28.6% for the Gumti river, - 6.67% and 48.6% for the Kushiyara river and – 92.34% and 5.70% for the Mohananda river.

The MNE and the SD of the MNE of Chezy's C are 74.63% and 88.80% for the Gumti river, 50.65% and 100.70% for the Kushiyara river and 1811.48% and 1552.00% for the Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.25, 5.26 and 5.27, respectively.

5.3.3.4 White et al. (1987) method

Like the Brownlie (1983) method, the White et al. (1987) method also predicts high Manning's n value at high discharge and low Manning's n value at low discharge. The range of the predicted Manning's n value from low to high discharge is 0.108 to 0.226 for the Gumti river, 0.119 to 0.212 for the Kushyara river and 0.208 to 0.271 for the Mohananda river.

The MNE and the SD of MNE of Manning's n is -82.14% and 2.30% for the Gumti river, 358.25% and 39.10% for the Kushiyara river and -15.60% and 69.10% for the Mohananda river.

The MNE and the SD of MNE of Chezy's C is -77.43% and 0.02% for the Gumti river, - 80.21% and 14.50% for the Kushiyara river and 134.51% and 201.70% for the Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.28, 5.29 and 5.30, respectively.

5.3.3.5 Karim (1995) method

The accuracy of the predicted roughness coefficients for the Gumti, the Kushiyara and the Mohananda rivers by Karim (1995) method seem to be poor. For the Gumti river the range of predicted Manning's n vary from 0.024 to 0.015 and the corresponding range of n values by the

Manning formula vary from 0.068 to 0.023, for the Kushiyara river the range of predicted values vary from 0.021 to 0.024 and by the Manning formula range is from 0.096 to 0.015, and for the Mohananda river the range of predicted values is from 0.024 to 0.013 and the corresponding range of the calculated values is from 2.316 to 0.081.

The MNE and the standard deviation of MNE of Manning's n is -31.16% and 29.50% for the Gumti river, -8.15% and 43.60% for the Kushiyara river and -94.27% and 3.10% for the Mohananda river.

The MNE and the SD of MNE of Chezy's C is -58.70% and 72.80% for the Gumti river, 143.12% and 81.40% for the Kushiyara river and 1745.76% and 1254.50% for the Mohananda river.

The comparison between the calculated and the predicted Manning's n values of the Gumti, Kushiyara and Mohananda rivers are shown in Figs. 5.31, 5.32 and 5.33, respectively.

5.4 Summary of the Prediction

Out of the eleven selected roughness predictors, six roughness predictors based on the division of roughness predict better results than the five overall roughness predictors. Only the Simons and Richardson (1966) overall roughness predictor seems to predict resonable Manning's n and Chezy's C for the river Gumti and the Kushiyara, but for the Mohananda river the prediction is very poor. The Garde and Ranga Raju (1977), Brownlie (1983), White et al. (1987) and Karim (1995) overall roughness predictors seem to fail completely to predict roughness coefficients in all the three rivers. The Brownlie (1983) method for the Gumti river and Brownlie (1983) and Karim (1995) methods for the Kushiyara river, however, predict depth in resonable range in full range of discharges but not for the Mohananda rivers (Ref. Tables 5.9, 5.10 and 5.11 and the Figs. 5.40, 5.41 and 5.42).

On the other hand, out of the six roughness predictors based on division of roughness, except for Engelund and Hansen (1967) and Van Rijn (1993) methods, the other four methods, e.g. the Einstein and Barbarossa (1952), Shen (1962), Haque and Mahmood (1983) and Shen (1990) methods, predict roughness coefficients reasonably well for the three selected rivers (Ref. Tables

5.6, 5.7 and 5.8 and the Figs. 5.34, 5.35, 5.36(a), and 5.36(b)). The Einstein and Barbarossa (1952) and Shen (1962) methods in the Gumti river predict depth reasonably well (Ref. Tables 5.9, 5.10, 5.11, 5.12 and 5.13 and the Figs. 5.40, 5.41 and 5.42). The Einstein and Barbarossa (1952) method predicts appreciably good results for all the three rivers. The predicted roughness coefficients are very close to the corresponding calculated values using Manning, Chezy and Darcy-Weisbach formulae. The Shen (1962) method predicts slightly higher values of both the roughness coefficients and depth than that of calculated values and the variation is uniform from low to high discharges for the Gumti and Kushiyara rivers. So the method of Shen (1962) is to be used carefully to predict roughness coefficients and depth for both the Gumti and Kushiyara rivers. The quality of the predicted results by the Engelund and Hansen (1967) method seems to be very poor, and so it can not be used to predict alluvial roughness coefficients for all the three rivers. Haque and Mahmood (1983) method does not predict good result in Gumti river but the method predicts satisfactory results in upper flow regime for both the Kushiyara and Mohananda rivers. Shen (1990) predicts very good results in Kushiyara river for medium to high discharges but the method predicts very poor results in Gumti and Mohananda rivers. So this method can be used successfully in the river Kushiyara for medium to high discharges (i.e. for $Q \geq 350$ m^3/s, Tables 5.12 and 5.13). Van Rijn (1993) roughness predictor based on division of roughness does not seem to predict reasonable roughness coefficients for the Gumti, Kushiyara and Mohananda rivers.

5.5 Resistance Characteristics of the Rivers

5.5.1 Gumti River

The resistance characteristics of the Gumti river are shown in Figs. 5.37 and 5.49. The maximum and minimum values of the calculated Manning's n are 0.068 and 0.023 corresponding to low and high discharges, respectively. The grain roughness n′ varies from 0.010 to 0.015 for low to high discharges. It is, therefore, obvious that bed forms are always present in the Gumti river round the year. During the moonson when the grain roughness n′ is dominant, the bed form resistance is nearly equal to or slightly higher than the grain resistance. With increasing discharge bed form changes from ripple to dune and washed-out dune. Bed forms of the Gumti river never change to anti-dunes because according to Yalin (1992) to form anti-dunes minimum Froude number should be 0.6, whereas for the Gumti river the maximum Froude number is found 0.248. At lean period,

the discharge becomes low and the resistance to the river bed becomes higher by the formation of ripples and dunes. Surveys over dunes in some major rivers of Bangladesh by the River Survey Project (FAP-24) confirmed that over 40% of the beds are always occupied by dunes at any flow stages. Julien and Klaassen (1995) showed that mean dune steepness does not always decrease at high transport stages in large rivers and that large bed forms may exist even at high transport stages.

It is seen from the variation of the Manning's n over the period from 12.02.1981 to 27.11.1996 in Gumti river that the maximum value of roughness attained is in the month of March and it decreases to the minimum value in the month of May to October (Fig. 5.49). The variation of Chezy's C is reverse of Manning's n, i.e. it attains maximum value in monsoon and minimum value during lean period. The variation of the Darcy-Weisbach friction factor f is the same as as that of Manning's n.

5.5.2 Kushiyara River

Discharge data of the Kushiyara river collected from BWDB covers the period from 25.03.1982 to 23.11.1996. The maximum and minimum values of the Manning's n, calculated by Manning formula are 0.096 and 0.015 attain during low and high discharges, respectively. The grain roughness n′ of the Kushiyara river varies from 0.011 to 0.017 at low to high discharges, respectively. The grain roughness during high discharges is about 100 % of the total roughness. The form roughness (n″) part of the total roughness in lean period is about 88.54 % and the contribution of the form roughness to the total roughness in moonson is approximately zero percent. The bed form in lower discharges is ripple or dune and washed-out dunes which with increasing discharges becomes plane bed at high discharges. The maximum Froude number in the Kushiyara river is found 0.17 which is much smaller to form anti-dunes on the river bed. The maximum value of roughness is attained in the month of February. It again decreases to the minimum value in the month of July to September (Fig. 5.50). The variation of Chezy's C and Darcy-Weisbach friction factor, f, are the same as Gumti river. The resistance characteristics of the Kushiyara river are shown in Figs. 5.38 and 5.50.

5.5.3 Mohananda River

It was not possible to assess the roughness characteristics of Mohananda river like Gumti or Kushiyara river. The available discharge, cross-sectional area, velocity, depth and width covering the period from 06.04.1982 to 24.03.1992 have been collected from BWDB. But these data don't seem reliable, because the range of Manning's n obtained by Manning formula using these BWDB's data are 2.316 and 0.081, respectively (Ref. Table: 5.8 and Figs: 5.36(a) and 5.36(b)), which are about 30 to 3 times higher than the normal ranges of Manning's n (Chow, 1959, Table 5-6). The resistance characteristics of the Mohananda river are shown in Figs. 5.39 and 5.51.

Noticing the highly inconsistent roughness coefficients by using BWDB's data, an independent measurement of discharge in the Mohananda river at the Chapai Nawabganj station was taken on 21.07.99. A wooden float of 7.5 cm diameter disk was used to measure surface velocity at a point which was at a distance equal to $1/4^{th}$ of the river width from the left water edge. The float was released from an upstream point about 100 m above the measurement axis to attain the constant velocity before reaching that cross-section. The process was repeated four times at four different locations to have a representative surface velocity. The travel times to pass 30.5 m (100 ft) distance were recorded and the average of these four travel times was calculated. To get mean velocity the surface velocity was multiplied by 0.8 (Ref. LGED, 1997). Mean velocity was obtained as 0.94 m/s. The cross-section was divided into twelve pockets by eleven verticals. The water depth at each vertical was measured by lowering a 10 kg weight from a rope. This operation was performed by boat. The discharge was calculated by multiplying the total cross-sectional area by the mean velocity. The measured discharge was 2650 m^3/s and the corresponding value of Manning's n obtained is 0.039. For the same discharge (2650 m^3/s) the Manning's n is 0.130 from the Q Vs. n curve (Fig. 5.36(a)) of BWDB's data. The values of the roughness coefficients for the discharge of 2650 m^3/s, obtained from the independent measurement, from BWDB's data and predicted by the Einstein and Barbarossa (1952), Shen (1962) and Haque and Mahmood (1983) methods are given Table 5.2.

Table 5.2. Roughness coefficients of the river Mohananda by different methods when the discharge is 2650 m^3/s.

Method		Roughness coefficients		
Measured	Predicted	n	C	f
Independent (measurement)	-	0.039	33.40	0.070
From BWDB's data	-	0.130	14.50	0.674
-	Einstein-Barbarossa	0.017	66.67	-
-	Shen (1962)	0.033	54.50	0.058
-	Haque-Mahmood	0.019	71.84	0.015

CHAPTER 6

CONCLUSIONS AND RECOMMANDATIONS

6.1 Conclusions

The present study leads to the following conclusions:

6.1.1 River Gumti

1. Among the eleven selected alluvial roughness predictors, the Einstein and Barbarossa (1952) method seems to be the best to predict the roughness coefficients satisfactorily. It also seems to predict grain roughness satisfactorily.
2. The Shen (1962) method seems to be the second best method which can also be used to predict the roughness coefficients and depth but the predicted Manning's n and Chezy's C are about 45 % higher and 19 % lower, respectively, than the corresponding calculated values. The method also seems to be applicable to predict roughness coefficients of the Gumti river.
3. The Simon and Richardson (1966) is another method that can be used to predict roughness coefficients of the Gumti river. The Manning's n obtained by using Simons and Richardson (1966) method is approximately 48 % higher than the calculated values for all discharges.
4. Most of the predictors give better results for higher discharges (i.e. $Q \geq 150$ m^3/s) when the velocity is also high. The associated bed form is either dunes or washed-out dunes.
5. The resistance characteristics of the river Gumti show a cyclic variation over the year like other alluvial rivers. The resistance to flow is maximum during the dry season when the discharge is low and resistance is minimum during monsoon when the discharge is high.
6. The Manning's n for the river Gumti increases with decreasing discharge, attains its maximum value during the month of March, then decreases gradually with increasing discharge and attains its minimum value during months of May and September. The Darcy-Weisbach friction factor f also shows a similar trend over the year. The variation of Chezy's C is reverse to that of Manning's n and Darcy-Weisbach friction factor f. The Chezy's C increases with increasing discharge and decreases with decreasing discharge.

7. The grain roughness increases with increasing discharges. At low discharges the grain roughness is the minor part of the total roughness. At high discharges the grain roughness is close to the total roughness.

8. The form roughness decreases with increasing discharge. It is more responsive to discharge than the grain roughness. At high discharges, the form roughness is close to zero, i.e. its influence is negligible. The form roughness increases more gradually with the decrease in discharges. At low discharges the influence of the form roughness relative to the total roughness is prominent and becomes the major constituent of the total roughness.

6.1.2 River Kushiyara

1. The Einstein and Barbarossa (1952) method predicts roughness coefficients satisfactorily for the Kushiyara river also. The Haque and Mahmood (1983) and the Shen (1990) methods seem to predict satisfactory results and can be used for moderate to high discharges (i.e. $Q \geq 350\ m^3/s$,). These two methods predict lower values of Manning's n than those obtained by the Manning formula if discharge $Q < 350\ m^3/s$ (Ref. Table 5.7. and 5.13).

2. The Simons and Richardson (1966) method predicts roughness coefficients satisfactorily for low discharges and gives higher values for high discharges.

3. Shen (1962) predicts Manning's n about 100 % higher than the calculated Manning's n for all discharges.

4. Except for the above mentioned methods no other methods predict roughness coefficients close to the calculated values for any range of discharges.

5. The resistance characteristics of the river Kushiyara is same as Gumti river, showing a cyclic variation over the year like other alluvial rivers. The resistance to flow is maximum during the dry season when the discharges is low and resistance is minimum during monsoon when the discharge is high.

6. The Manning's n for the river Kushiyara also increases with decreasing discharge, attains its maximum value during February, then decreases gradually with increasing discharge and attains its minimum value during August and September. The Darcy-Weisbach friction factor f also shows a similar variation over the year. The variation of Chezy's C is reverse

to that of Manning's n and Darcy-Weisbach friction factor f. The Chezy's C increases with increasing discharge and vice versa.

7. The grain roughness increases with increasing discharges. At low discharges the grain roughness is the minor part of the total roughness. At high discharges the grain roughness alone is either equal to or close to the total roughness.
8. The form roughness decreases with increasing discharge. It is more responsive to discharge than the grain roughness. At high discharges, the form roughness is either zero or very close to zero, i.e. its influence is negligible. The form roughness increases more gradually with the decrease in discharges. At low discharges the influence of the form roughness relative to the total roughness is prominent and becomes the major constituent of the total roughness.

6.1.3 River Mohananda

The hydrological data collected by BWDB for the Mohananda river at the station Chapai Nawabganj do not seem reliable (Art. 5.5.3). All the analyses are, however, done using the data collected by BWDB. So, the roughness coefficients calculated by the Manning, the Chezy and the Darcy-Weisbach formulae and predicted by the alluvial roughness predictors may not be reliable. This is why, no conclusion could be drawn on the calculated values of roughness coefficients as well as on the applicability of the alluvial roughness predictors for the Mohananda river.

6.2 Recommendations

1. The present study indicates that no one method was found satisfactory to predict the roughness coefficients of the three rivers. Only the Einstein and Barbarossa (1952) method predicts roughness coefficients satisfactorily for full range of discharges for the Gumti and the Kushiyara rivers. With the exception of Einstein and Barbarossa (1952) method no orther methods were satisfactory for the Gumti and Kushiyara rivers for full range of discharge. The Haque and Mahmood (1983) and Shen et al. (1990) methods predict satisfactory results from moderate to high discharges for the Kushiyara river. Moreover, prediction by most of the methods at low discharges was less satisfactory than that for high discharges. Therefore,

a comprehensive study may be undertaken to develop roughness predictor(s) to study the roughness characteristics of the alluvial rivers of Bangladesh.

2. For the Mohananda river the calculated values of the roughness coefficients have indicated that the discharge data collected by BWDB at Chapai Nawabganj station do not seem reliable. BWDB can take necessary measures to identify the problem as to whether it is due to human or instrumental errors.

3. Hydrological data for the period 15 years for the river Gumti, 14 years for the river Kushiyara and 10 years for the river Mohananda, as were available with BWDB, have been used in this study. More data covering at least a period of 20 years or more are to be used for a better analysis.

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APPENDIX – A: TABLES

Table 5.3 MNE and standard deviation of MNE of the predicted roughness and hydraulic parameters using different roughness predictors for the Gumti river.

Methods	Predicted parameters of the Gumti river									
	n		C		f		U		R	
	MNE	SD of MNE	MNE	SD of MNE	MNE	SD of MNE	MNE	SD of MNE	MNE	SD of MNE
Einst.-Bar	1.9	32.3	5.2	27.3					7.2	52.7
Shen(62)	45.0	45.7	-19.4	43.5	81.2	88.0			81.2	88.0
Enge-Han	-60.3	2.8	197.7	154	-80.4	162				
Haque-Mah	-16.6	53.0	84.4	134.0	-1.9	143.0				
Shen (90)	-37.2	46.0	162.4	219.0	-38.9	110.0				
Van Rijn	-23.3	36.8	63.1	117.4	-26.2	50.3				
Simons	48.9	92.4	6.3	92.3	913.9	1432.5				
Garde-R.	-34.1	27.8	83.7	97.3	-48.8	49.1				
Brownlie	-34.3	28.6	74.6	88.8					-23.5	51.6
White	-82.1	2.3	-77.4	0.1	3082.3	2527.0				
Karim	-31.2	29.5	58.7	72.8	-38.7	52.7	28.9	39.4	-25.5	20.2

Table 5.4 MNE and standard deviation of MNE of the predicted roughness and hydraulic parameters using different roughness predictors for the Kushiyara river.

Methods	Predicted parameters of the Kushiyara river									
	n		C		f		U		R	
	MNE	Std of MNE	MNE	Std of MNE	MNE	Std of MNE	MNE	Std of MNE	MNE	Std of MNE
Einst-Bar	**38.1**	**38**	**-16.2**	**23**	**66.9**	**64**				
Shen(62)	130.8	47.7	-38.7	13.5	197.1	97.6			197.6	98.0
Enge-Han	-43.3	29.3	152.6	173.3	-59.5	32.3				
Mahmood	-5.6	40.3	24.9	55.0	5.3	113.4				
Shen(90)	-12.7	35.7	31.9	50.5	-11.4	88.0				
Van Rijn	0.8	59.5	58.3	120.0	15.8	84.1				
Simons	87.4	82.0	-28.9	58.0	1275.0	1037.0				
Raju	-38.2	30.3	131.3	159.0	-53.9	37.0				
Brownlie	-6.7	48.6	50.6	100.7	6.6	84.2			-7.1	39.0
White	658.3	391.0	-80.2	14.5	7189.9	5999.0				
Karim	-8.2	43.6	143.1	81.4	2.1	74.4	19.6	47.0	-82.8	12.1

Table 5.5 MNE and standard deviation of MNE of the predicted roughness and hydraulic parameters using different roughness predictors for the Mohananda river

Methods	Predicted parameters of the Mohananda river									
	n		C		f		U		R	
	MNE	SD of MNE	MNE	SD of MNE	MNE	SD of MNE	MNE	SD of MNE	MNE	SD of MNE
Einst-Bar.	-99.9	0.1	556.7	185.7					-69.4	8.5
Shen(62)	-83.5	7.4	472.4	277.0	-95.2	3.0			-75.3	15.4
Enge-Han	-94.7	5.2	3295.0	2922.0	-94.4	1.3				
Mahmood	-90.5	5.4	1577.0	1634.0	-98.8	1.3				
Shen(90)	-94.4	3.2	2619.7	2312.0	-99.6	0.5				
Van Rijn	-92.6	7.2	1732.0	1350.0	-98.2	2.3				
Simons	-92.9	5.5	2909.0	2998.0	-97.3	3.8				
Garde-R.	-92.6	5.3	2268.5	1940.0	-99.2	1.1				
Brownlie	-92.3	5.7	1811.5	1552.0	-97.8	17.8	291.8	204.2	-66.3	18.9
White	-15.6	69.1	134.5	201.7	19.4	185.2				
Karim	-94.3	3.1	1745.7	1254.5	-99.4	0.7				

Table 5.6 Variation of calculated and predicted Manning's n with discharge of the Gumti river

Discharge (m3/s)	Method of calculations				
	By Manning formula	Calculations using different roughness predictors			
		Einst-Bar.(52)	Shen(62)	Haque-M.(83)	Shen (90)
1.70	0.068	0.052	0.076	0.035	0.010
7.00	0.048	0.047	0.070	0.039	0.073
15.00	0.046	0.048	0.063	0.029	0.062
19.00	0.038	0.038	0.059	0.028	0.061
27.00	0.038	0.038	0.058	0.023	0.058
30.00	0.036	0.039	0.055	0.020	0.056
40.00	0.044	0.039	0.056	0.016	0.052
50.00	0.039	0.039	0.053	0.016	0.052
64.00	0.033	0.033	0.048	0.015	0.051
75.00	0.035	0.033	0.048	0.015	0.050
100.00	0.034	0.030	0.045	0.050	0.048
111.00	0.033	0.030	0.043	0.050	0.047
117.00	0.029	0.028	0.040	0.050	0.047
145.00	0.029	0.028	0.039	0.050	0.014
154.00	0.030	0.028	0.038	0.047	0.015
250.00	0.029	0.026	0.033	0.044	0.016
364.00	0.023	0.019	0.024	0.040	0.017

Table 5.7 Variation of calculated and predicted Manning's n with discharge of the Kushiyara river

Discharge (m3/s)	Method of calculations					
	By Manning formula	Calculations using different roughness predictors				
		Einst-B.(52)	Shen (62)	Haque-M.(83)	Shen (90)	Simons-R.(66)
43.00	0.096	0.078	0.080	0.035	0.035	0.070
83.00	0.076	0.056	0.095	0.042	0.043	0.057
108.00	0.078	0.055	0.094	0.038	0.040	0.054
132.00	0.072	0.055	0.093	0.038	0.040	0.054
145.00	0.065	0.054	0.091	0.038	0.040	0.054
196.00	0.049	0.054	0.085	0.037	0.040	0.054
245.00	0.045	0.054	0.083	0.038	0.040	0.054
313.00	0.042	0.050	0.080	0.035	0.037	0.053
442.00	0.035	0.044	0.074	0.032	0.034	0.051
530.00	0.034	0.044	0.072	0.029	0.030	0.050
730.00	0.023	0.038	0.061	0.029	0.031	0.050
945.00	0.023	0.038	0.058	0.026	0.026	0.049
1250.00	0.023	0.039	0.054	0.021	0.020	0.047
1670.00	0.020	0.031	0.046	0.019	0.015	0.046
2000.00	0.020	0.031	0.043	0.018	0.014	0.045
2200.00	0.018	0.028	0.041	0.019	0.015	0.046
3080.00	0.015	0.022	0.030	0.018	0.012	0.045

Table 5.8 Variation of calculated and predicted Manning's n with discharge of the Mohananda river

Discharge (m3/s)	Method of calculations				
	By ManningD formula	Calculations using different roughness predictors			
		Einst-Bar.(52)	Shen(62)	Haque-M.(83)	Shen (90)
12.00	2.316	0.166	0.690	0.043	0.027
26.00	1.526	0.104	0.070	0.040	0.025
35.00	0.955	0.072	0.070	0.042	0.025
45.00	0.585	0.051	0.069	0.043	0.025
52.00	0.578	0.050	0.069	0.043	0.025
59.00	0.410	0.044	0.067	0.045	0.026
100.00	0.493	0.043	0.066	0.036	0.022
160.00	0.362	0.039	0.063	0.036	0.022
300.00	0.263	0.038	0.055	0.032	0.019
440.00	0.220	0.039	0.050	0.028	0.017
570.00	0.191	0.031	0.046	0.026	0.016
818.00	0.171	0.030	0.042	0.021	0.013
1010.00	0.175	0.030	0.042	0.021	0.012
1290.00	0.166	0.026	0.039	0.019	0.011
1620.00	0.112	0.021	0.029	0.019	0.011
2130.00	0.098	0.019	0.024	0.019	0.010
3660.00	0.081	0.015	0.016	0.017	0.008

Table 5.9 Variation of observed and predicted water depth with discharge of the Gumti river

Discharge (m3/s)	Method of calculations				
	Observed depth(m)	Predicted water depth (m) using different predictors			
		Einst-Bar(52)	Shen (62)	Brownlie (83)	Karim (95)
1.70	0.560	0.380	0.660	0.240	0.200
7.00	0.950	1.640	2.400	0.680	0.600
15.00	1.430	1.530	2.300	0.860	0.800
19.00	1.630	1.690	2.440	0.970	1.000
27.00	1.720	1.720	3.170	1.180	1.200
30.00	1.890	2.080	3.560	1.360	1.400
40.00	2.450	2.000	3.480	1.590	1.600
50.00	2.410	2.400	3.860	1.690	1.600
64.00	3.230	2.670	4.080	2.140	2.000
75.00	3.280	2.670	4.080	2.160	2.200
100.00	3.400	2.820	5.110	2.690	2.600
111.00	3.450	2.930	5.180	2.770	2.600
117.00	4.000	2.750	4.700	2.760	2.600
145.00	4.330	3.010	4.910	3.030	2.800
154.00	4.490	3.330	5.610	3.630	3.200
250.00	4.770	3.640	5.960	4.340	3.600
364.00	5.390	3.940	5.960	5.740	4.200

Table 5.10 Variation of observed and predicted water depth with discharge of the Kushiyara river

Discharge (m3/s)	Method of calculations				
	Observed depth(m)	Predicted water depth (m) using different predictors			
		Einst-Bar(52)	Shen (62)	Brownlie (83)	Karim (95)
43.00	1.810	1.330	1.940	1.020	1.000
83.00	3.610	2.300	5.020	1.590	1.600
108.00	3.230	3.430	7.440	1.810	1.800
132.00	4.270	3.000	6.200	1.980	2.000
145.00	4.310	3.280	7.160	2.140	2.200
196.00	4.970	4.140	8.670	2.620	2.600
245.00	5.080	4.930	9.790	2.860	3.000
313.00	4.730	7.260	12.600	3.170	3.200
442.00	5.140	7.390	16.080	3.930	4.000
530.00	5.630	8.360	17.420	4.380	4.400
730.00	5.490	11.470	23.030	5.430	5.600
945.00	6.140	12.850	24.100	6.150	6.200
1250.00	7.090	15.380	25.570	7.260	7.200
1670.00	7.390	15.520	27.470	8.680	8.400
2000.00	8.480	15.850	27.610	9.460	9.000
2200.00	8.130	15.620	27.600	9.910	9.400
3080.00	8.490	17.050	27.540	10.320	9.800

Table 5.11 Variation of observed and predicted water depth with discharge of the Mohananda river

Discharge (m3/s)	Method of calculations				
	Observed depth(m)	Predicted water depth (m) using different predictors			
		Einst-Bar(52)	Shen(62)	Brownlie (83)	Karim (95)
12.00	2.190	0.210	0.070	0.250	0.200
26.00	2.240	0.250	0.310	0.470	0.400
35.00	2.560	0.270	0.380	0.560	0.600
45.00	3.160	0.280	0.440	0.690	0.600
52.00	3.210	0.290	0.450	0.710	0.600
59.00	3.790	0.260	0.370	0.720	0.600
100.00	4.050	0.360	0.690	1.000	1.000
160.00	4.060	0.500	1.020	1.240	1.200
300.00	4.850	0.910	1.570	1.850	1.600
440.00	5.380	1.360	2.000	2.320	1.800
570.00	5.770	1.270	2.370	2.750	1.800
818.00	6.730	1.730	2.800	3.500	2.200
1010.00	6.990	1.730	2.800	3.600	2.400
1290.00	7.910	1.690	3.030	4.240	2.800
1620.00	7.500	2.100	3.430	3.530	3.200
2130.00	7.890	2.290	3.400	4.040	3.800
3660.00	9.370	2.610	2.960	5.450	5.000

Table 5.12 Variation of calculated and predicted Manning's n with different range of discharges for the Gumti river

Range of discharge, Q (m^3/s)	Values of Manning's n by different methods		
	By Manning formula	Einstein-Barbarosssa (1952)	Shen (1962)
$Q \leq 2$	0.068	0.052	0.076
2 - 10	0.041	0.046	0.070
10 - 20	0.034	0.042	0.062
20 - 30	0.032	0.037	0.057
30 - 40	0.033	0.036	0.054
40 - 50	0.040	0.037	0.054
50 - 75	0.034	0.034	0.049
75 - 100	0.031	0.030	0.043
100-125	0.031	0.030	0.040
125-175	0.029	0.028	0.037
175-250	0.032	0.027	0.037
$Q \geq 250$	0.025	0.023	0.031

Table 5.13 Variation of calculated and predicted Manning's n with different range of discharges for the Kushiyara river

Range of discharge ,Q (m^3/s)	Values of Manning's n by different methods			
	By Manning formula	Haque and Mahmood(1983)	Shen et al.(1990)	Einstein and Barbar.(1952)
$Q \leq 50$	0.096	0.043	0.035	0.078
50 - 100	0.081	0.042	0.043	0.058
100 - 150	0.068	0.038	0.040	0.055
150 - 200	0.052	0.036	0.038	0.054
200 - 350	0.050	0.035	0.037	0.054
350 - 700	0.033	0.028	0.031	0.044
700 -1200	0.025	0.024	0.025	0.039
1200-1600	0.022	0.021	0.020	0.034
1600-2000	0.019	0.020	0.018	0.031
2000-2500	0.017	0.019	0.016	0.025
$Q \geq 2500$	0.016	0.018	0.014	0.024

APPENDIX – B: FIGURES

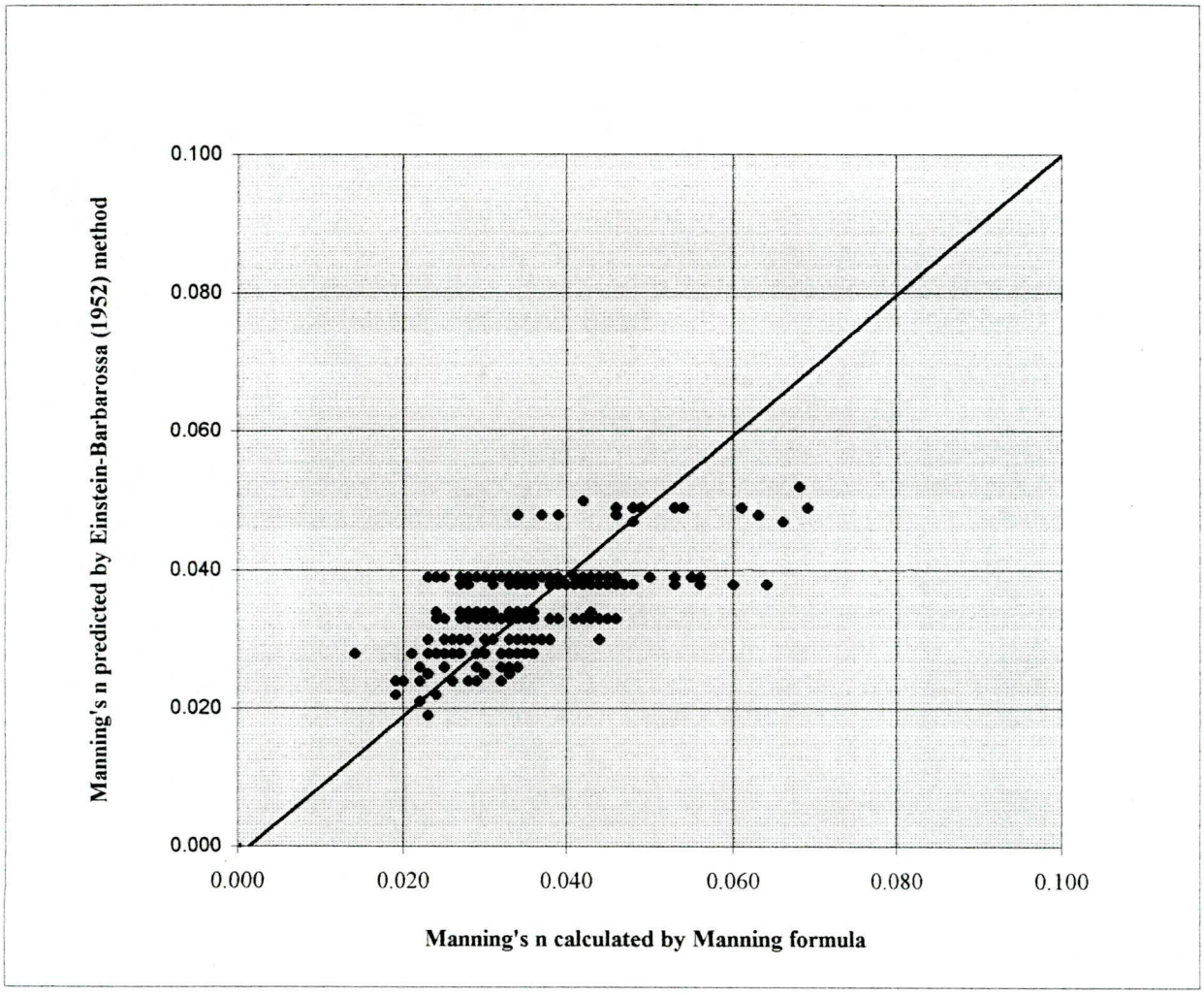

Fig.5.1	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Einstein-Barbarossa (1952) method of Gumti river**

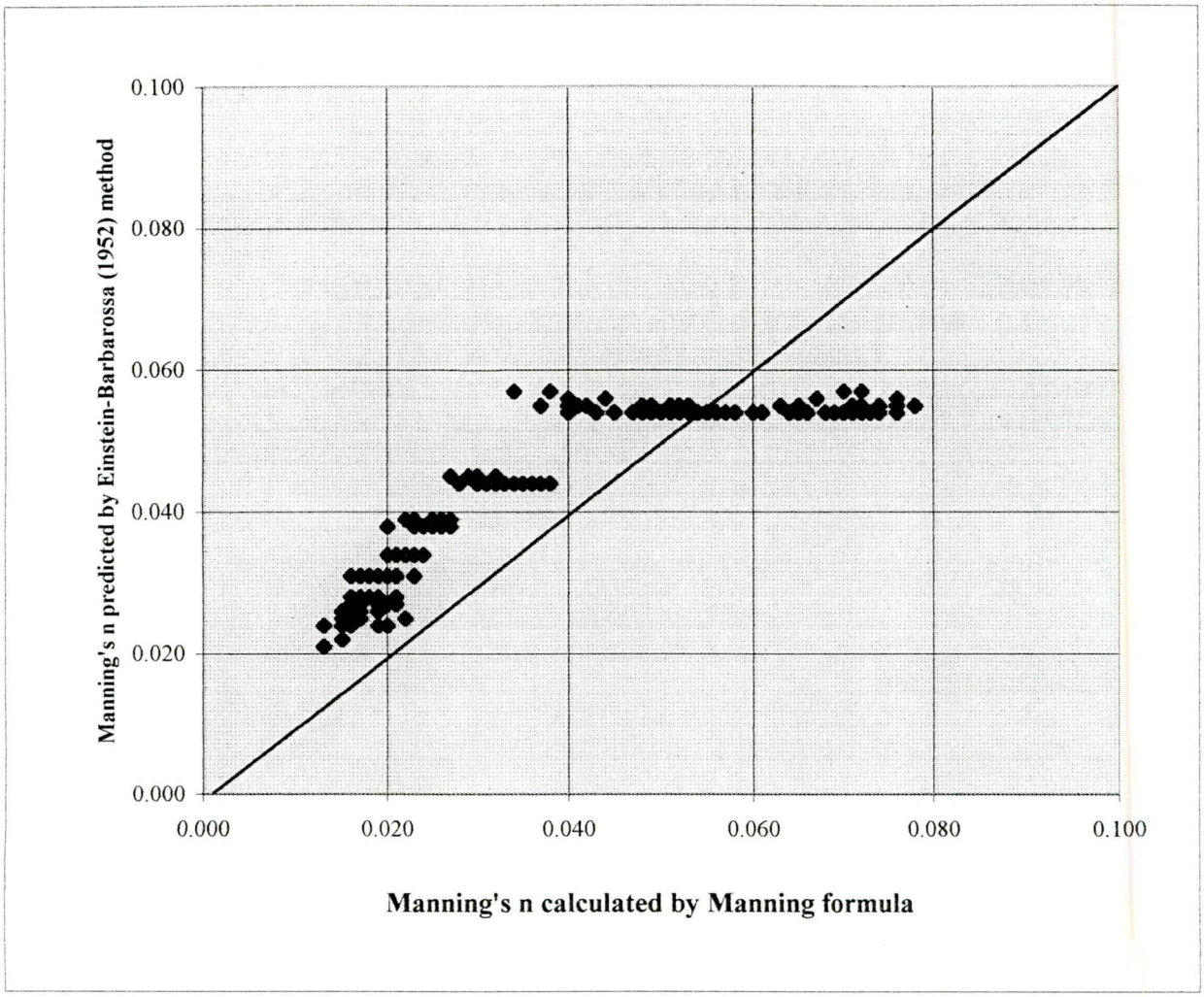

Fig.5.2	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Einstein and Barbarossa (1952) method of the Kushiyara river**

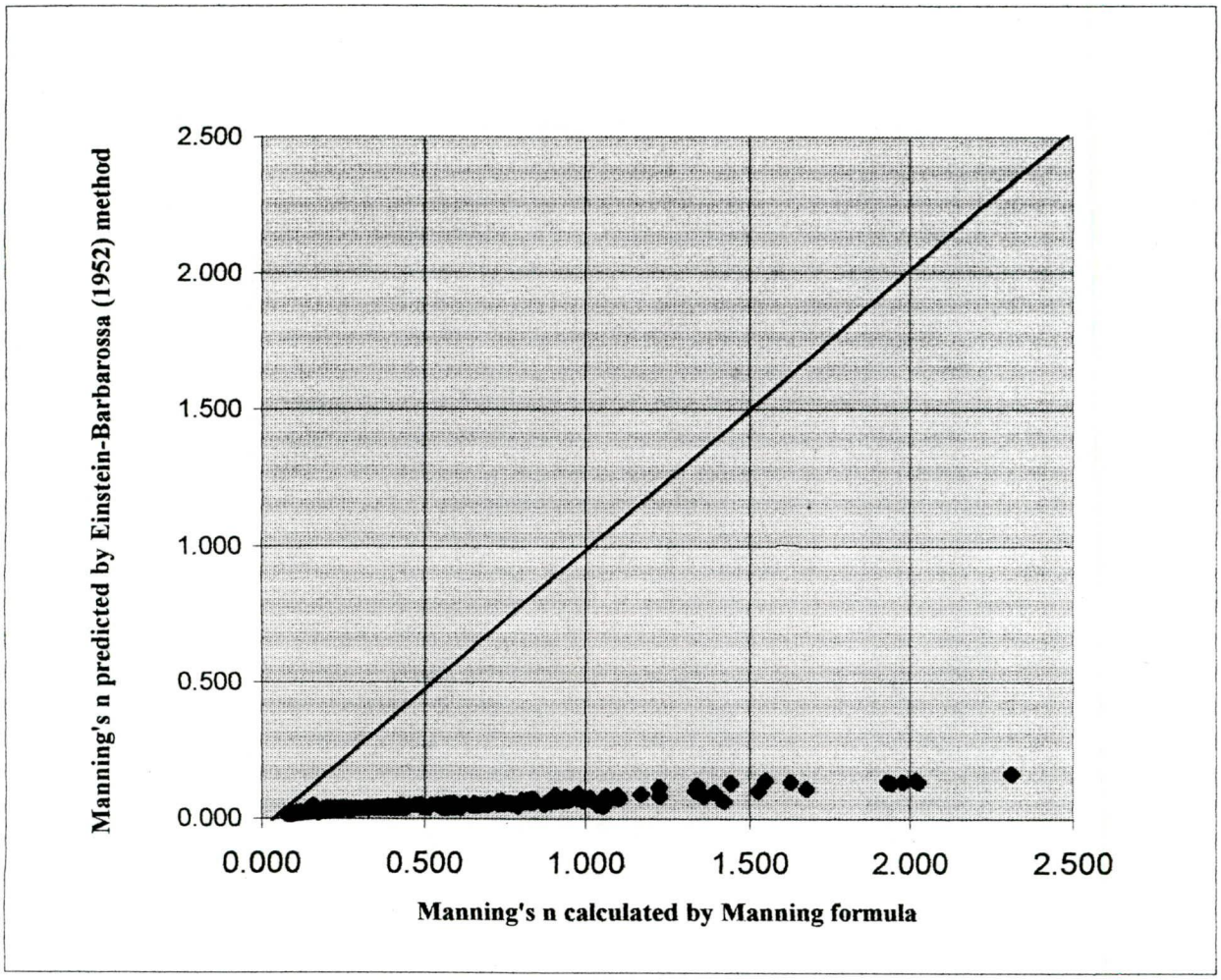

Fig.5.3	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Einstein-Barbarossa (1952) method of the Mohananda river**

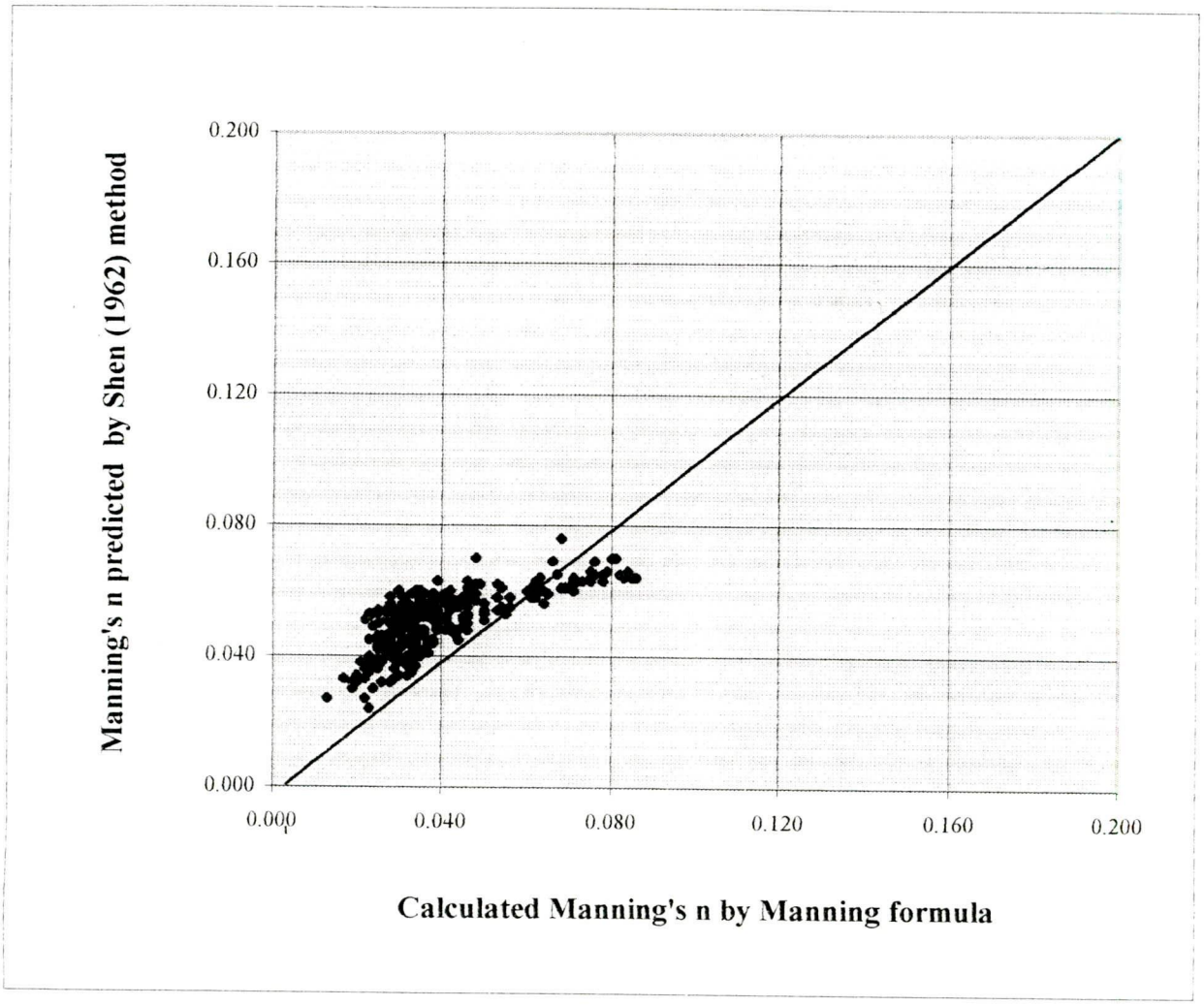

Fig.5.4	**Comparrison between the values of Manning's n calculated by Manning formula and those predicted by Shen (1962) method of the Gumti river**

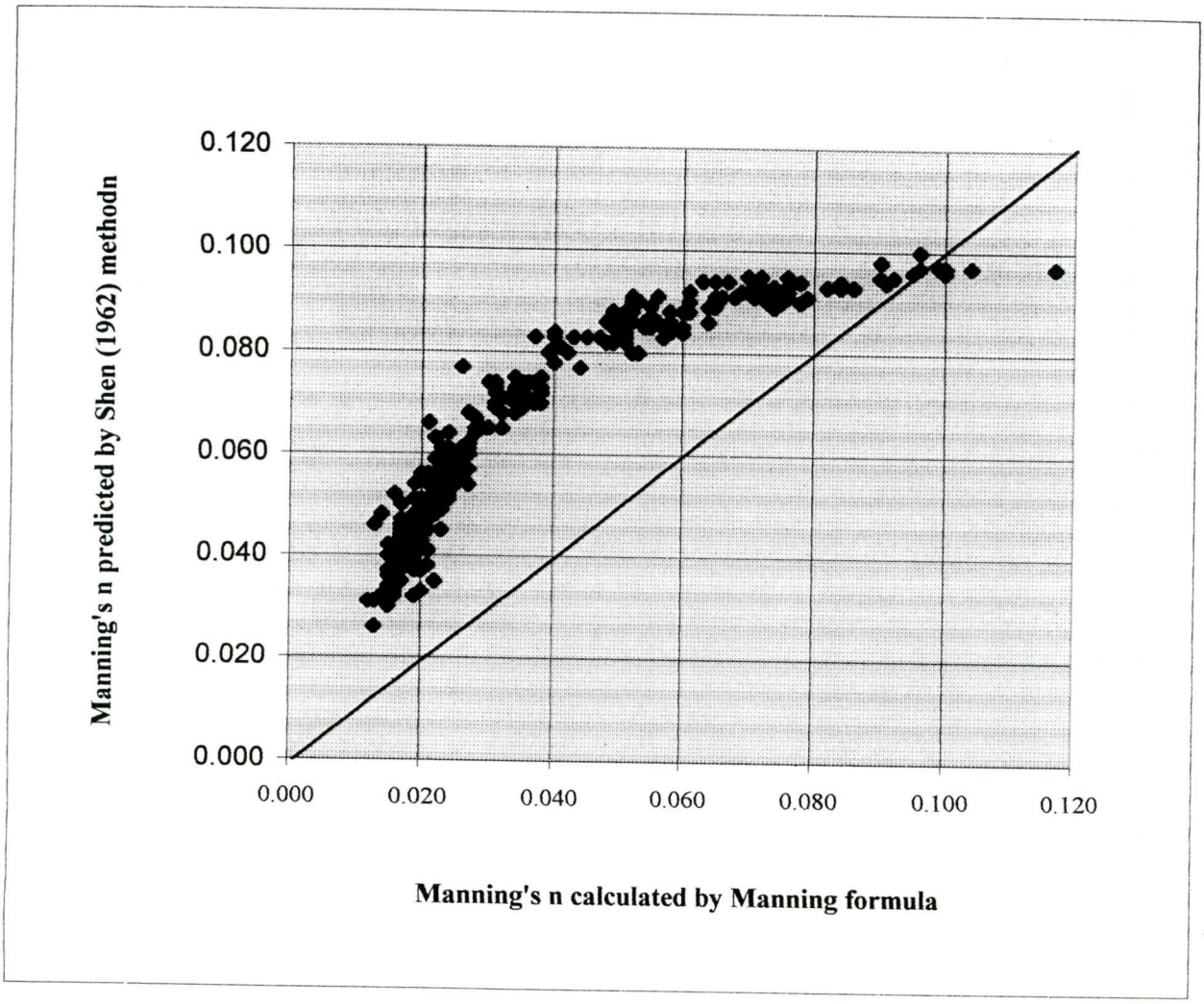

Fig.5.5	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Shen (1962) method of the Kushiyara river**

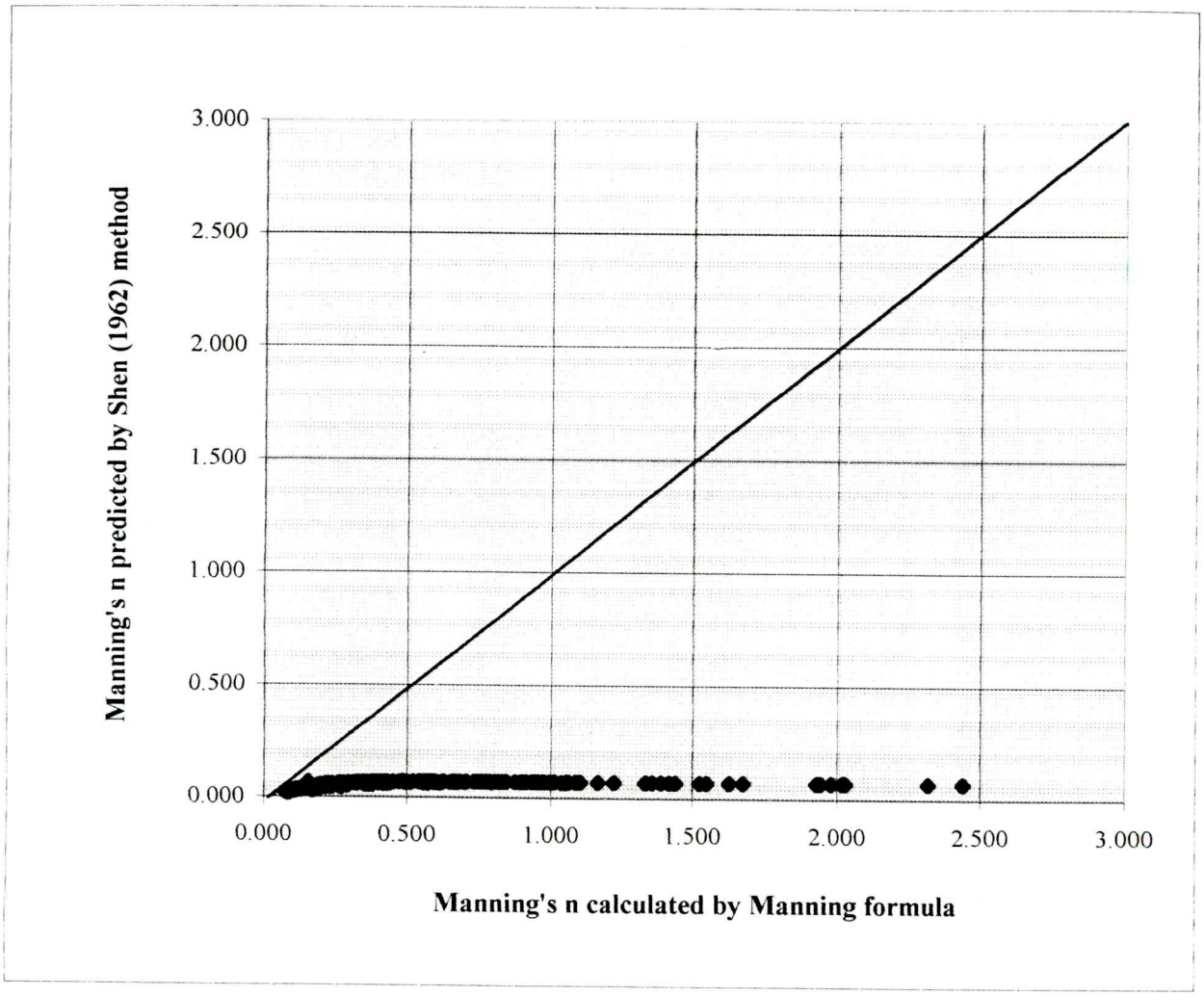

Fig.5.6	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Shen (1962) method of the Mohananda river**

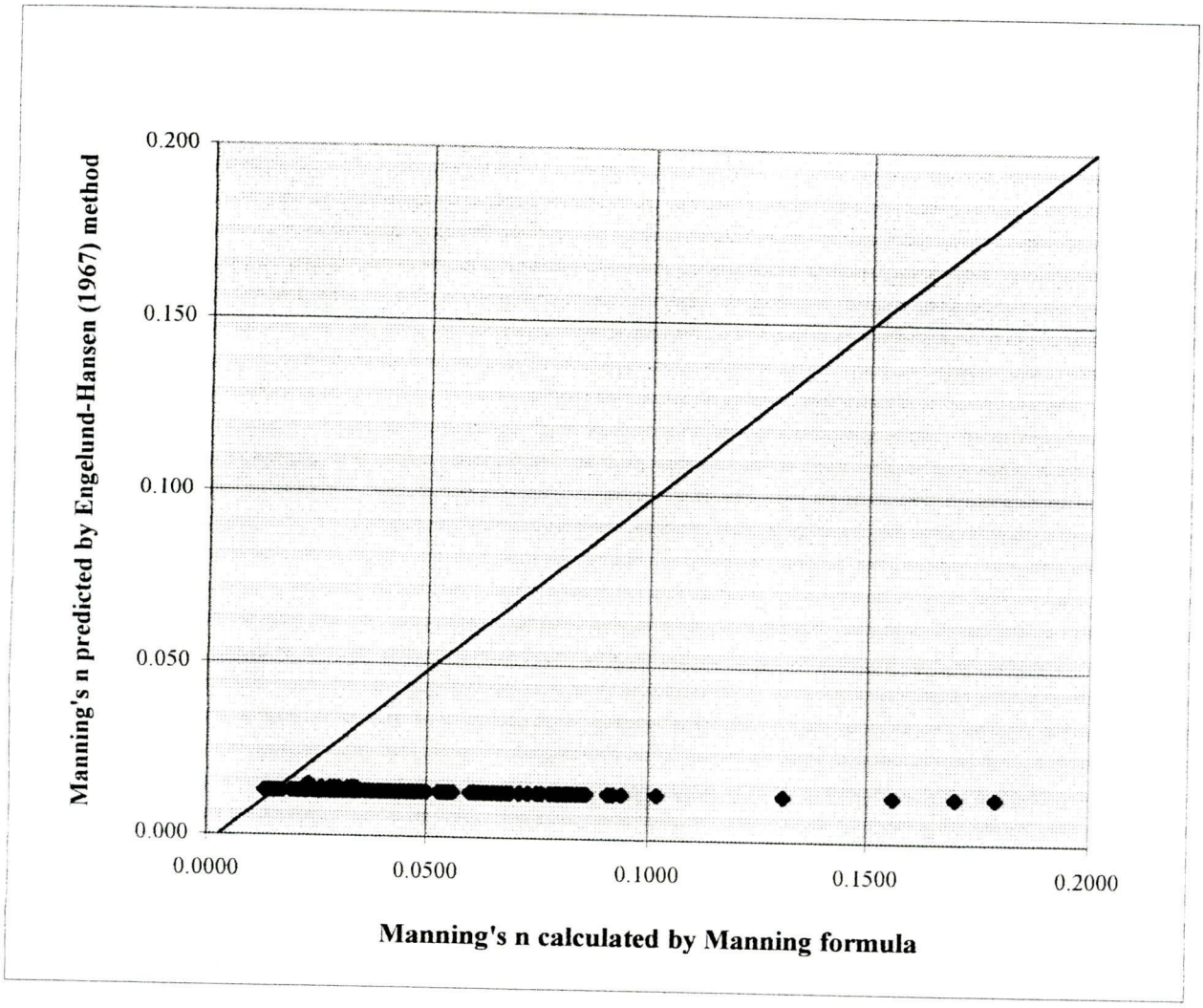

Fig.5.7	**comparison between the values of Manning's n calculated by Manning formula and those predicted by Engelund and Hansen (1967) method of the Gumti river**

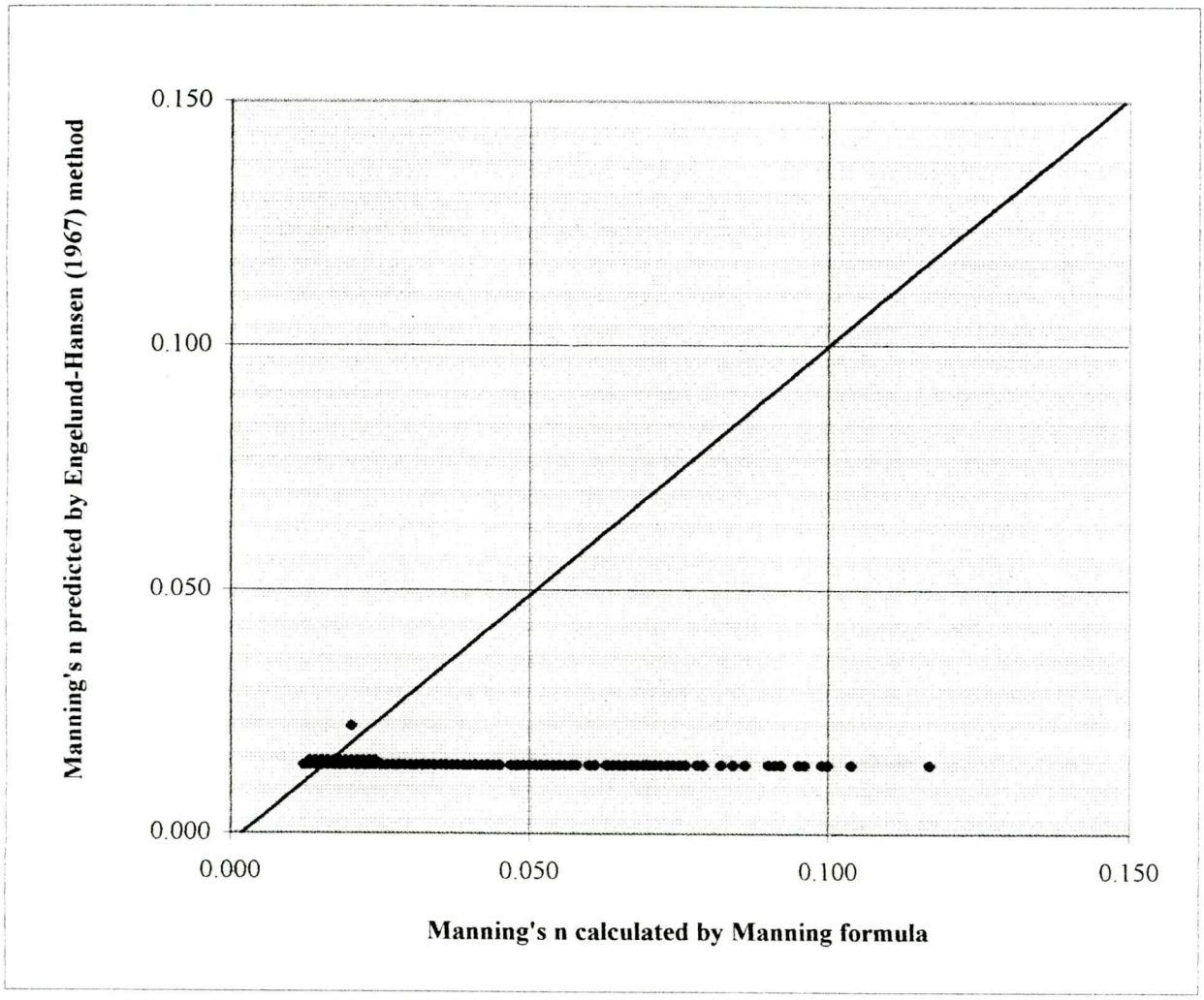

Fig.5.8	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Engelund-Hansen (1967) method of theKushiyara river**

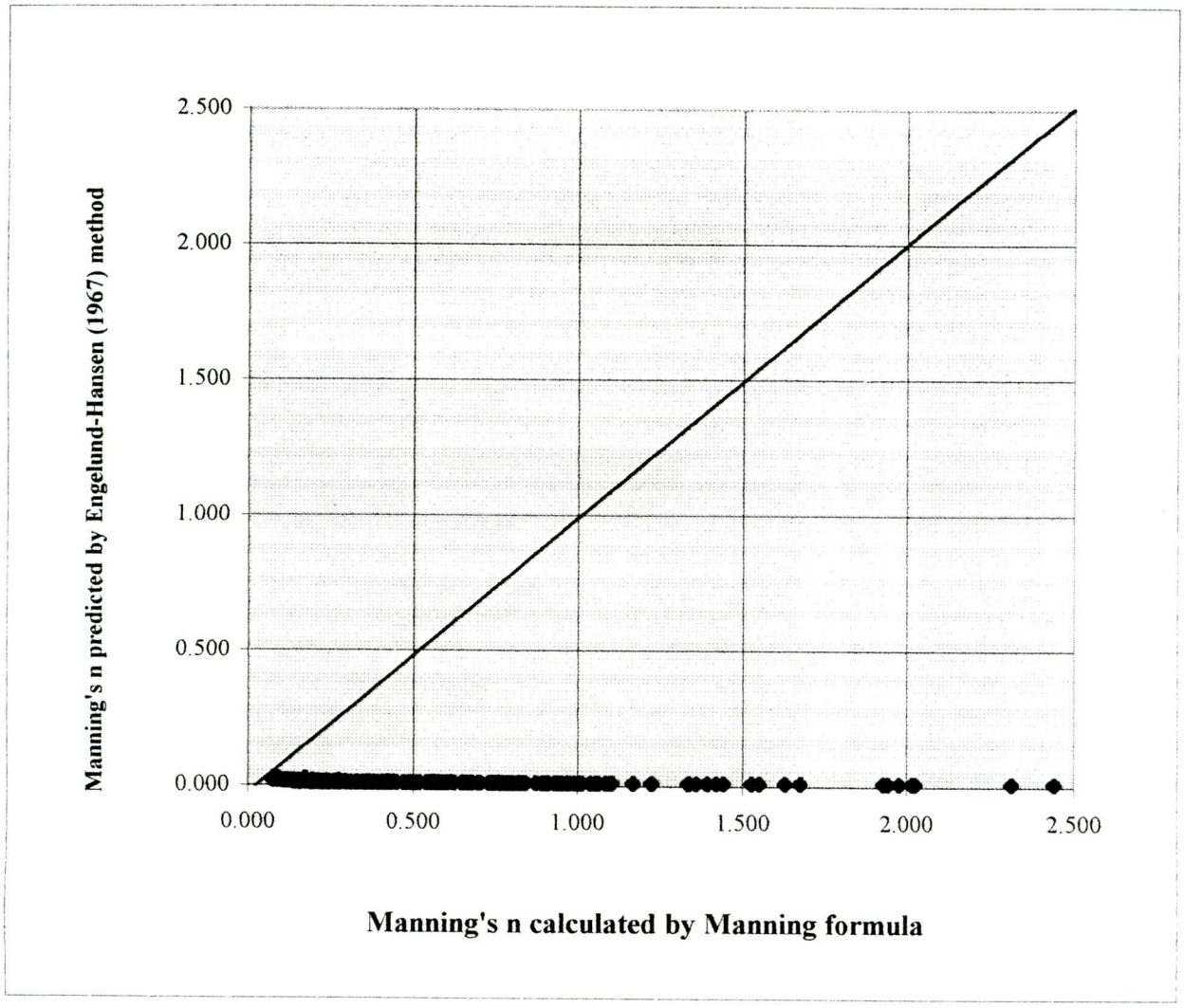

Fig.5.9	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Engelund and Hansen (1967) method of the Mohananda river**

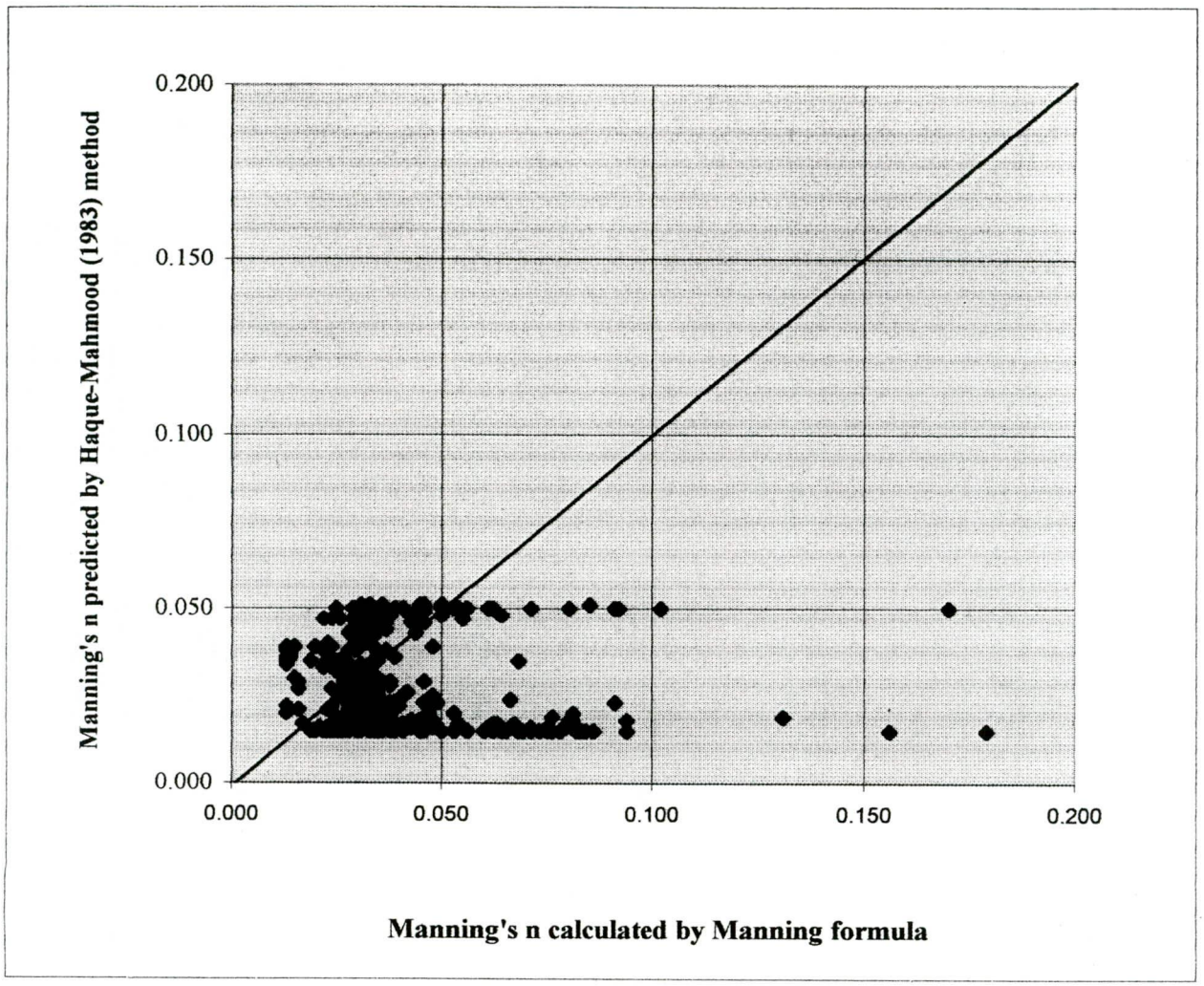

Fig.5.10	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Haqeu-Mahmood (1983) method of the Gumti river**

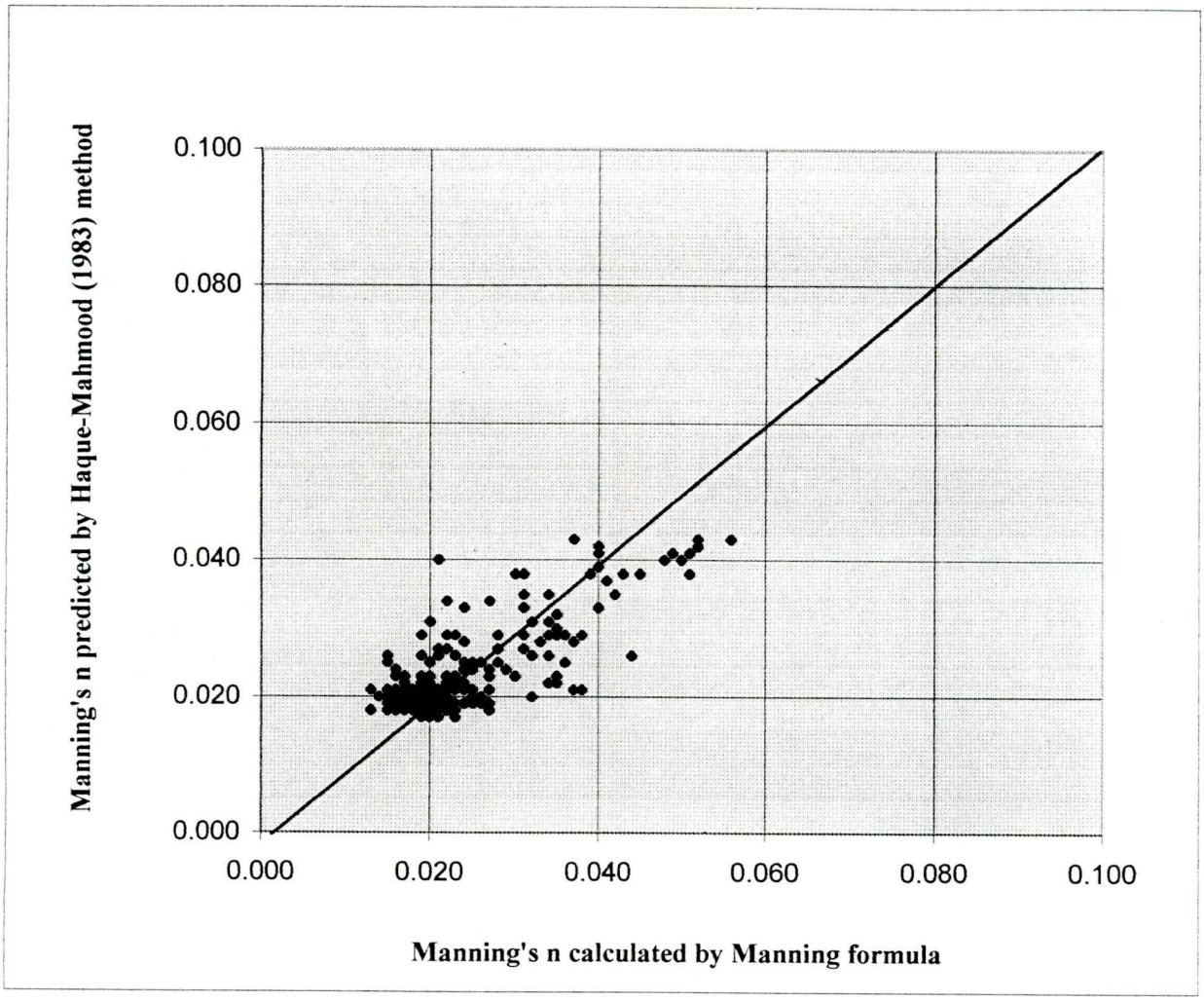

Fig.5.11	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Haque- Mahmood (1983) method of the Kushiyara river**

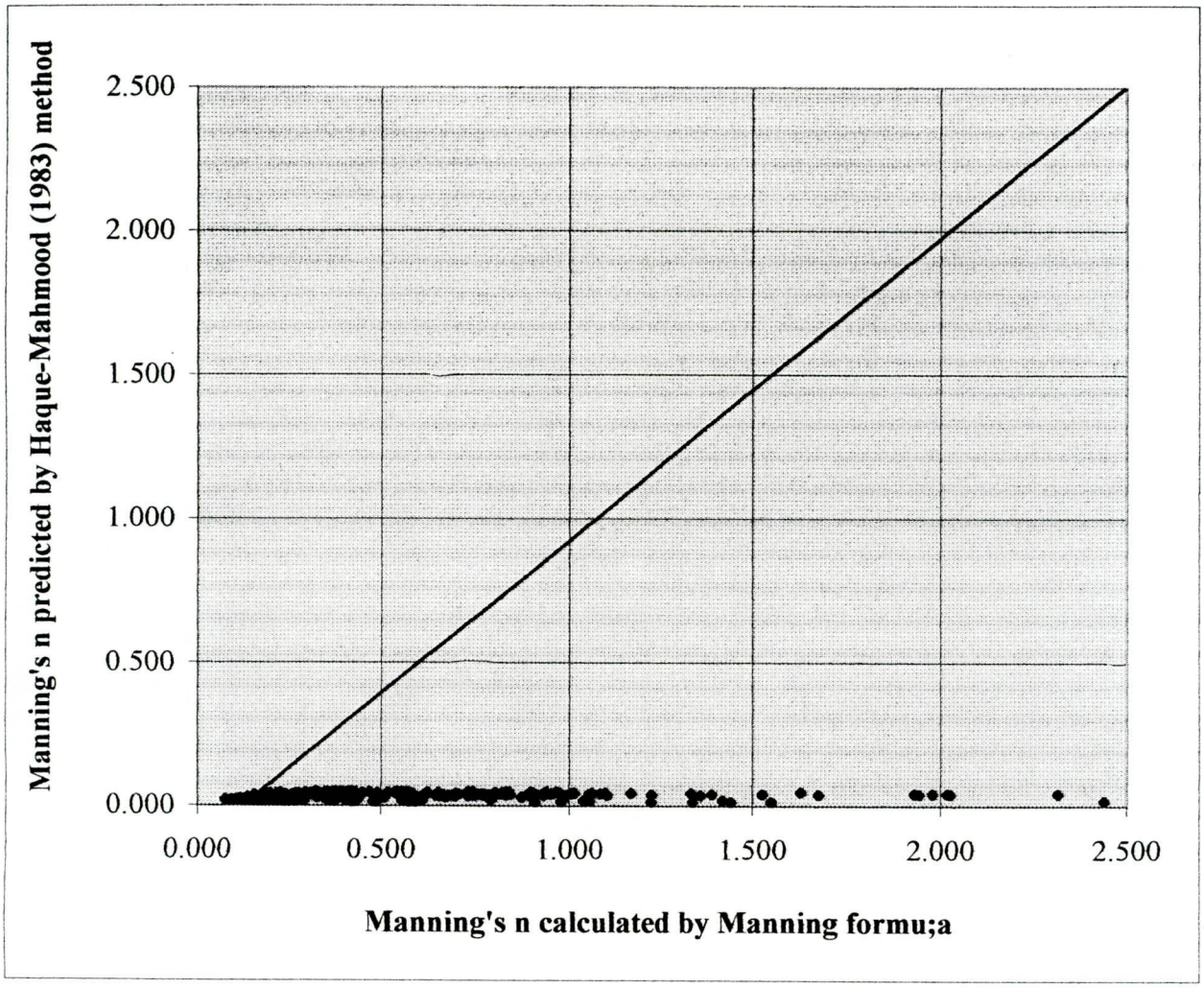

Fig.5.12	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Haque-Mahmood (1983) method of the Mohananda river**

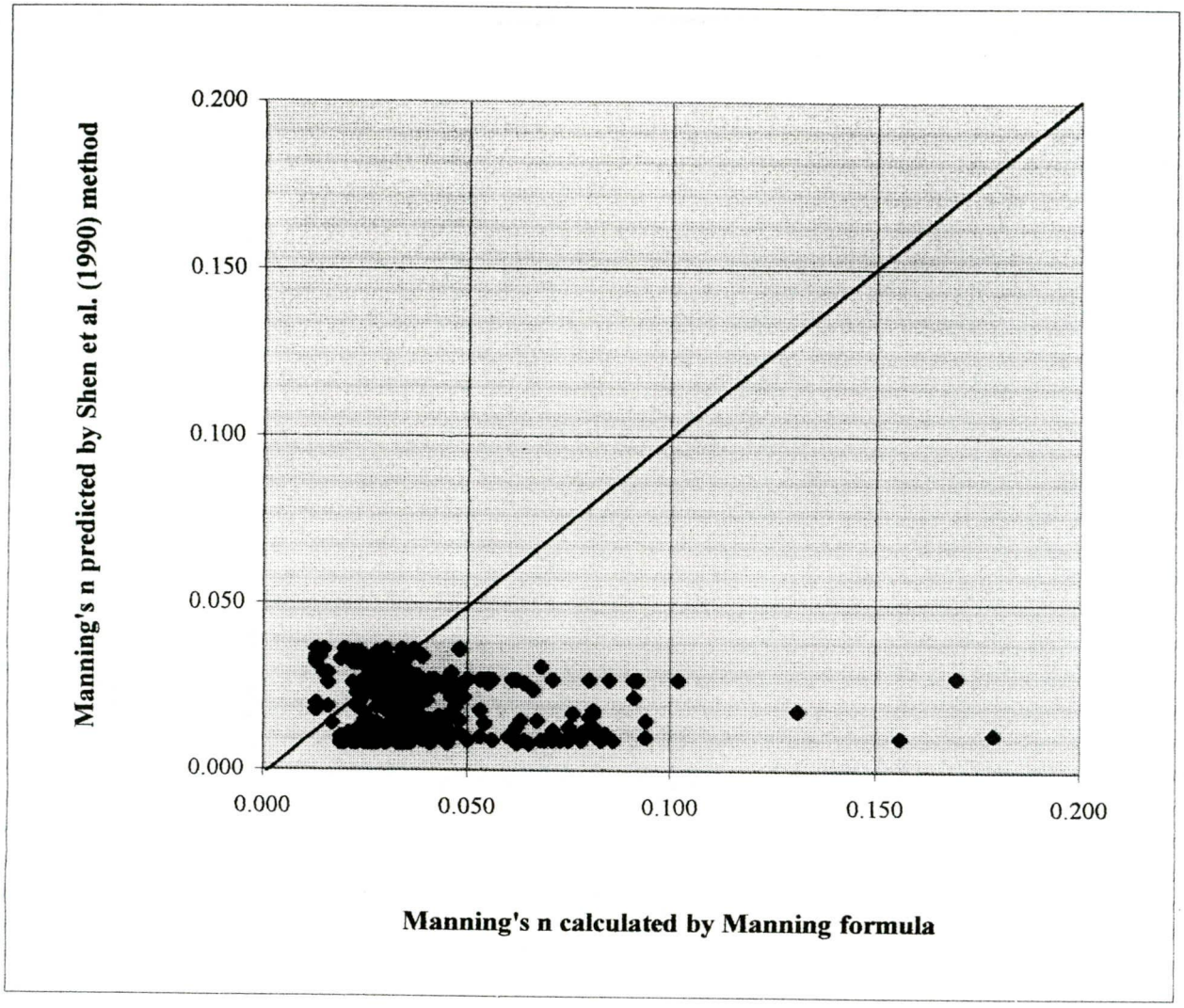

Fig.5.13	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Shen et al. (1990) method of the Gumti river**

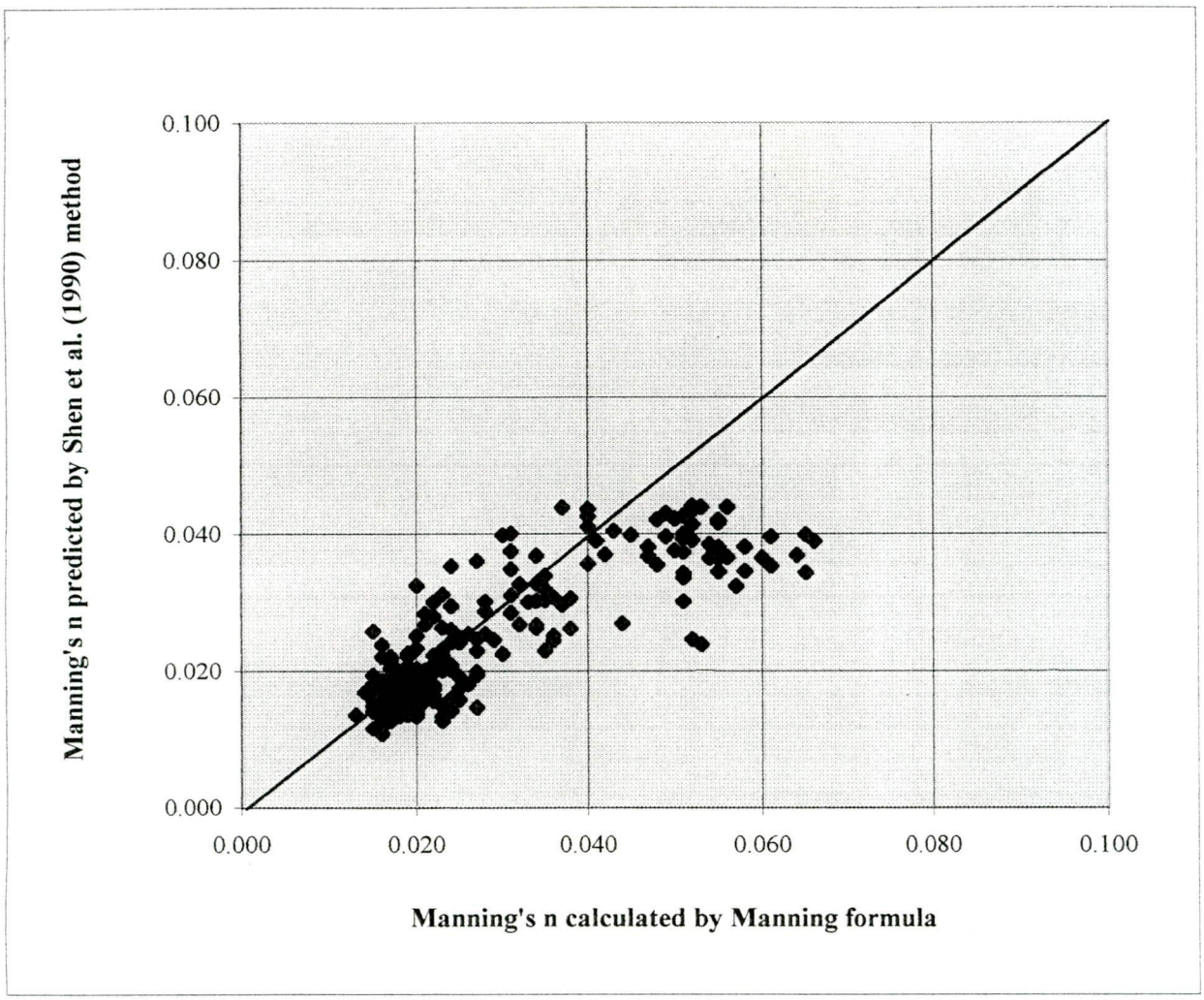

Fig.5.14	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Shen et al. (1990) method of the Kushiyara river**

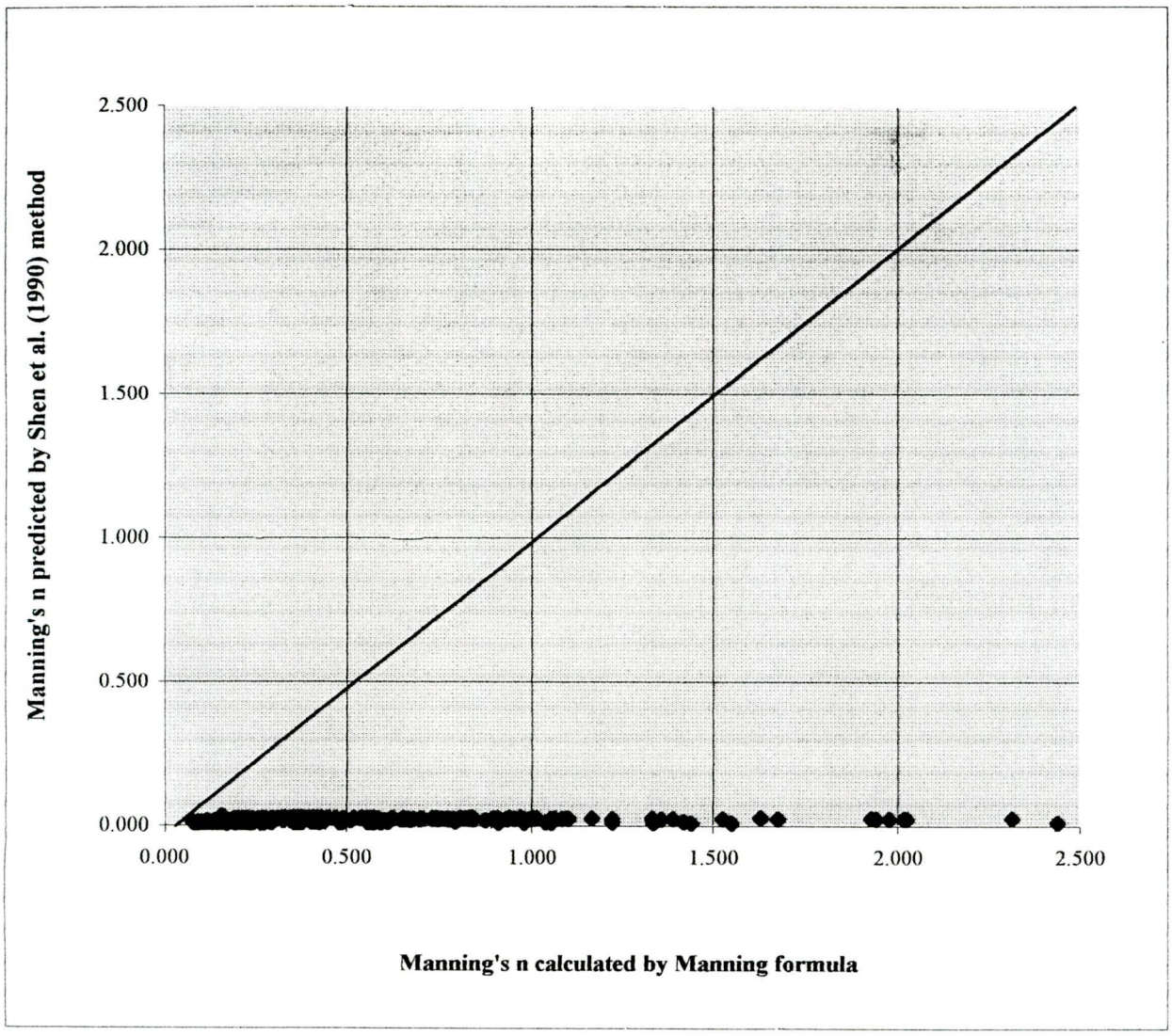

Fig.5.15	**Comparison between the values of Manning;s n calculated by Manning formula and those predicted by Shen et al. (1990) method of the Mohananda river**

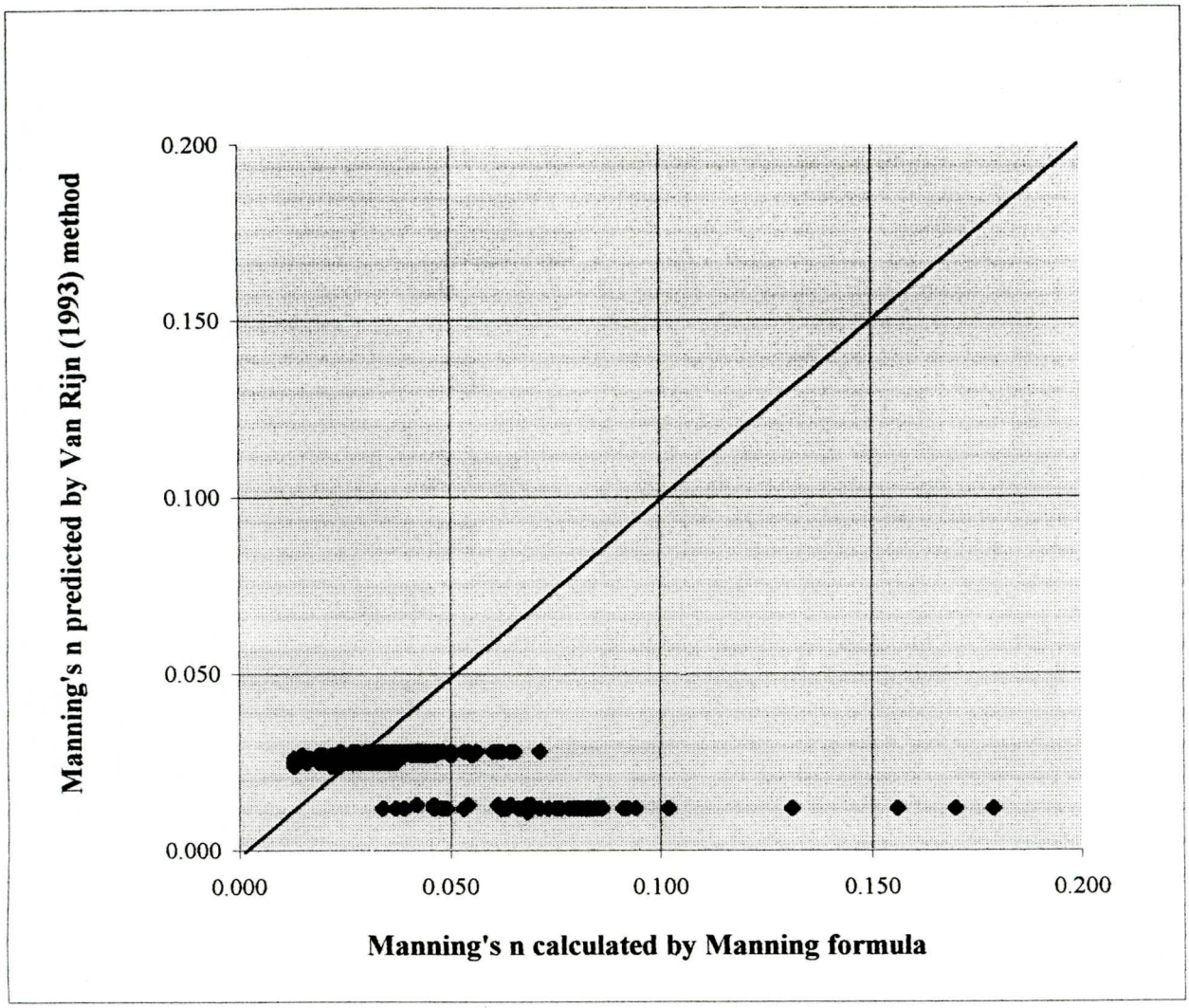

Fig.5.16	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Van Rijn (1993) method of the Gumti river**

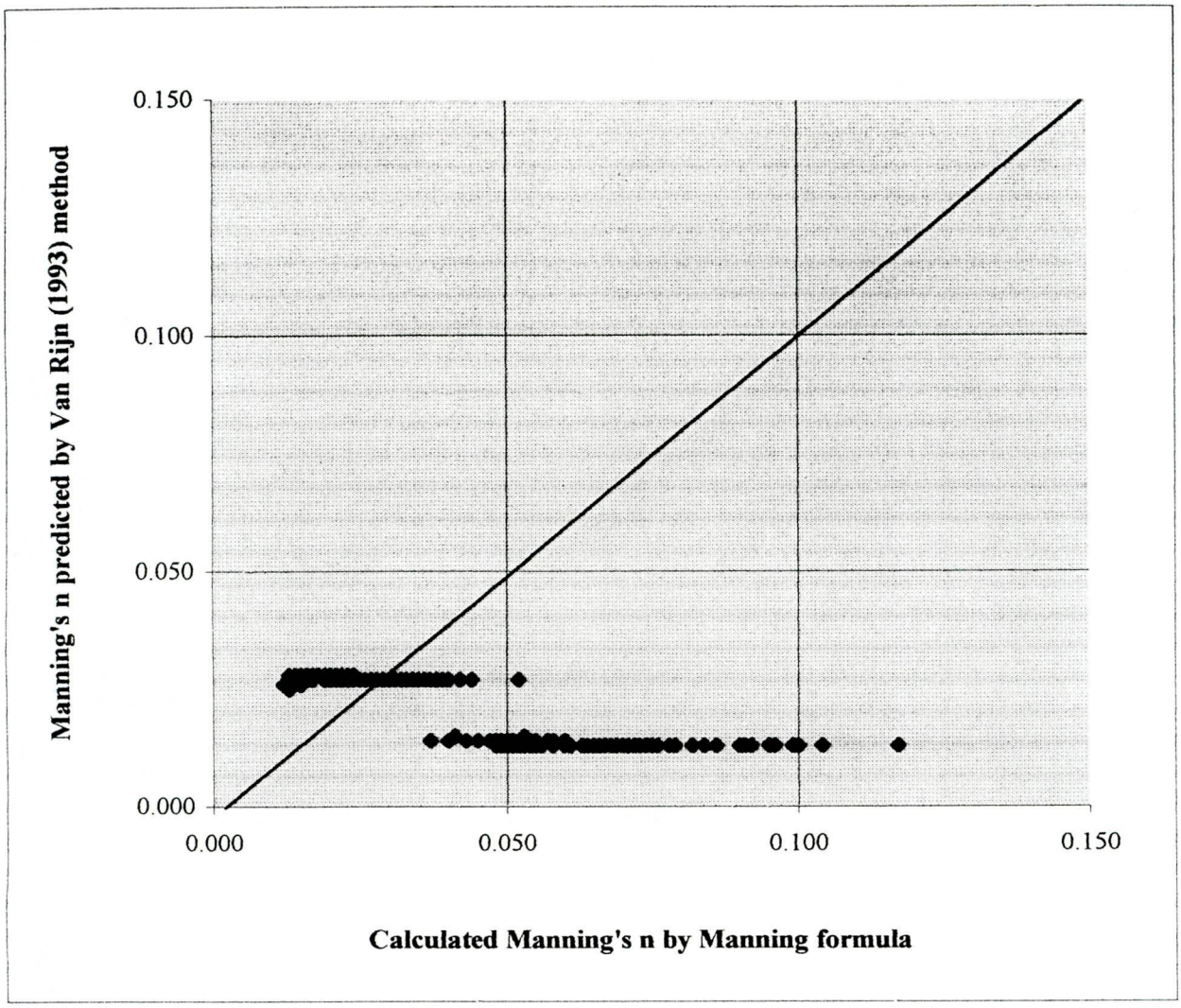

Fig.5.17	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Shen et al. (1990) method of the Kushiyara river**

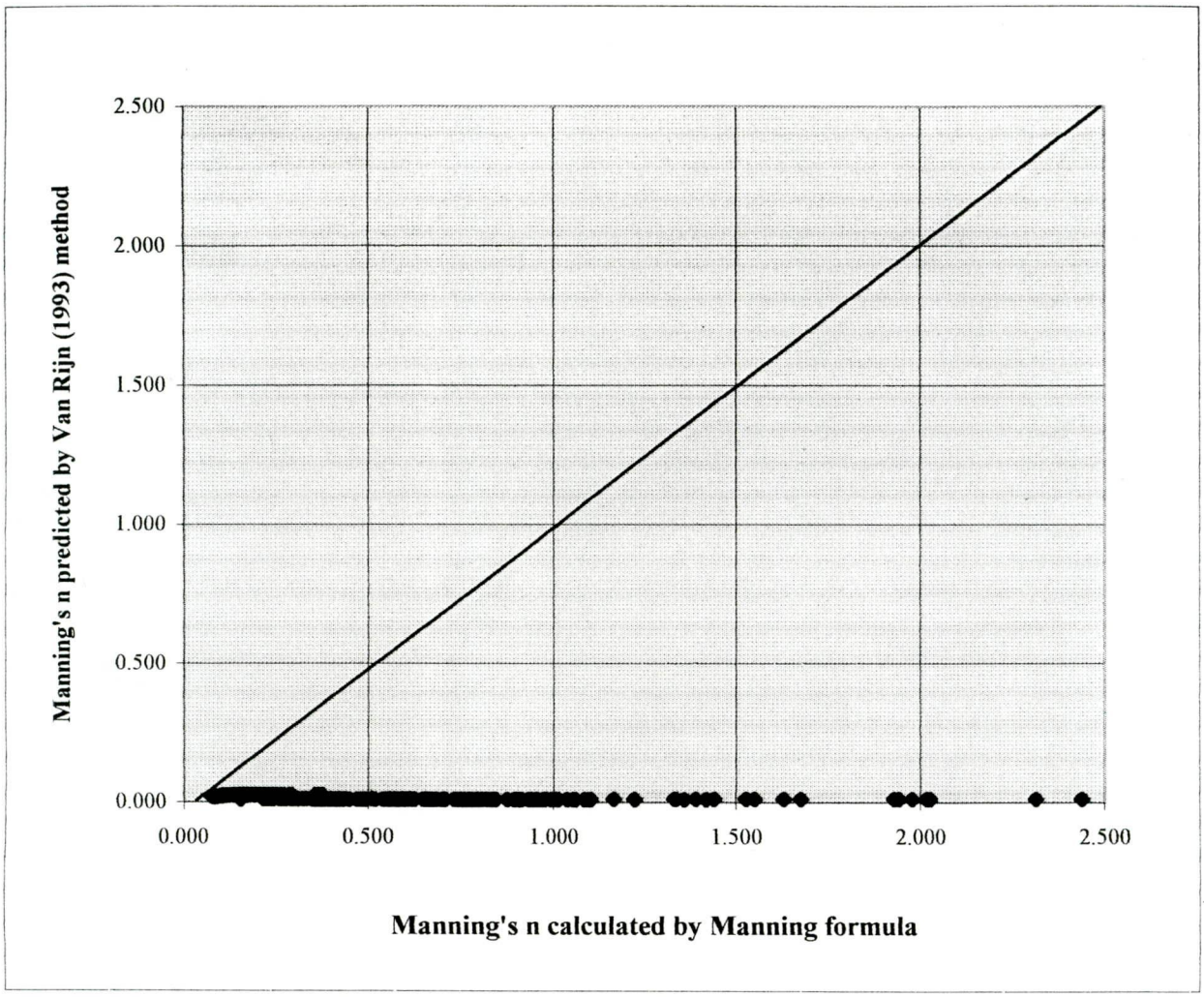

Fig.5.18	**Comparison between the valuesof Manning's n calculated by Manning formula and those predicted by Van Rijn (1993) method of the Mohananda river**

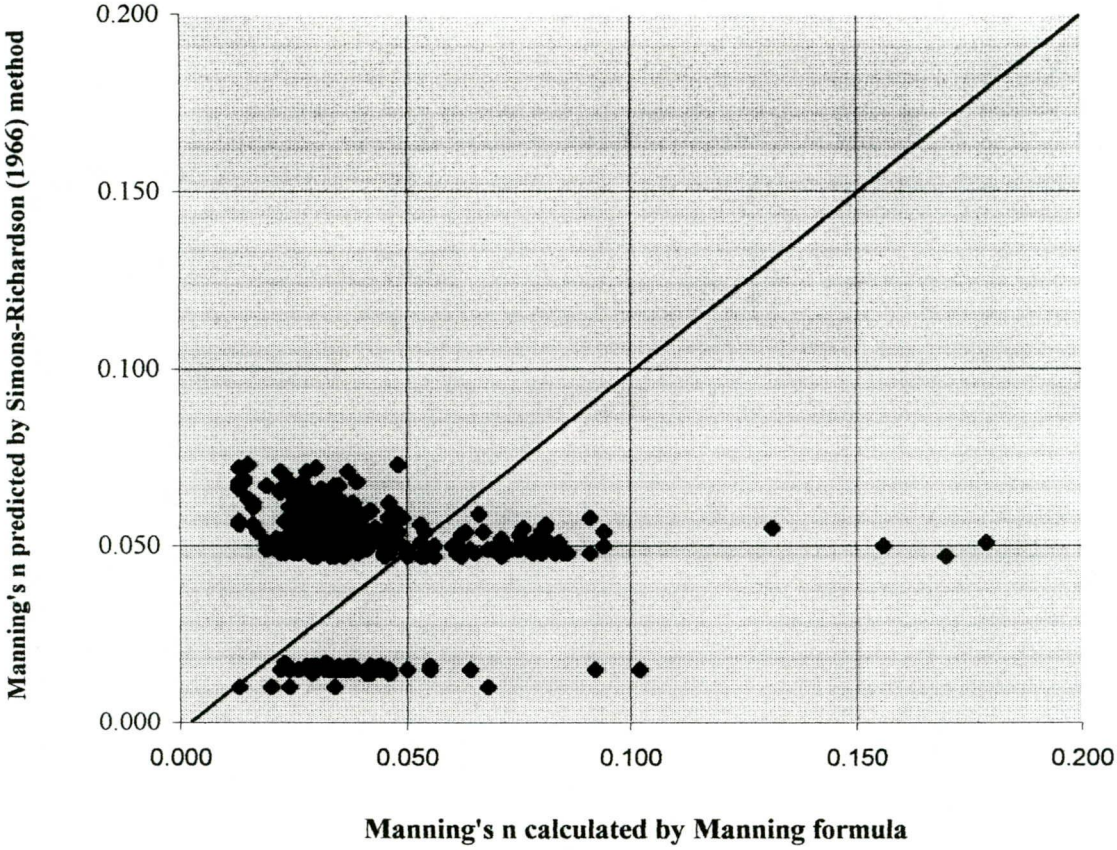

Fig.5.19	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Simons-Richardson (1966) method of the Gumti river**

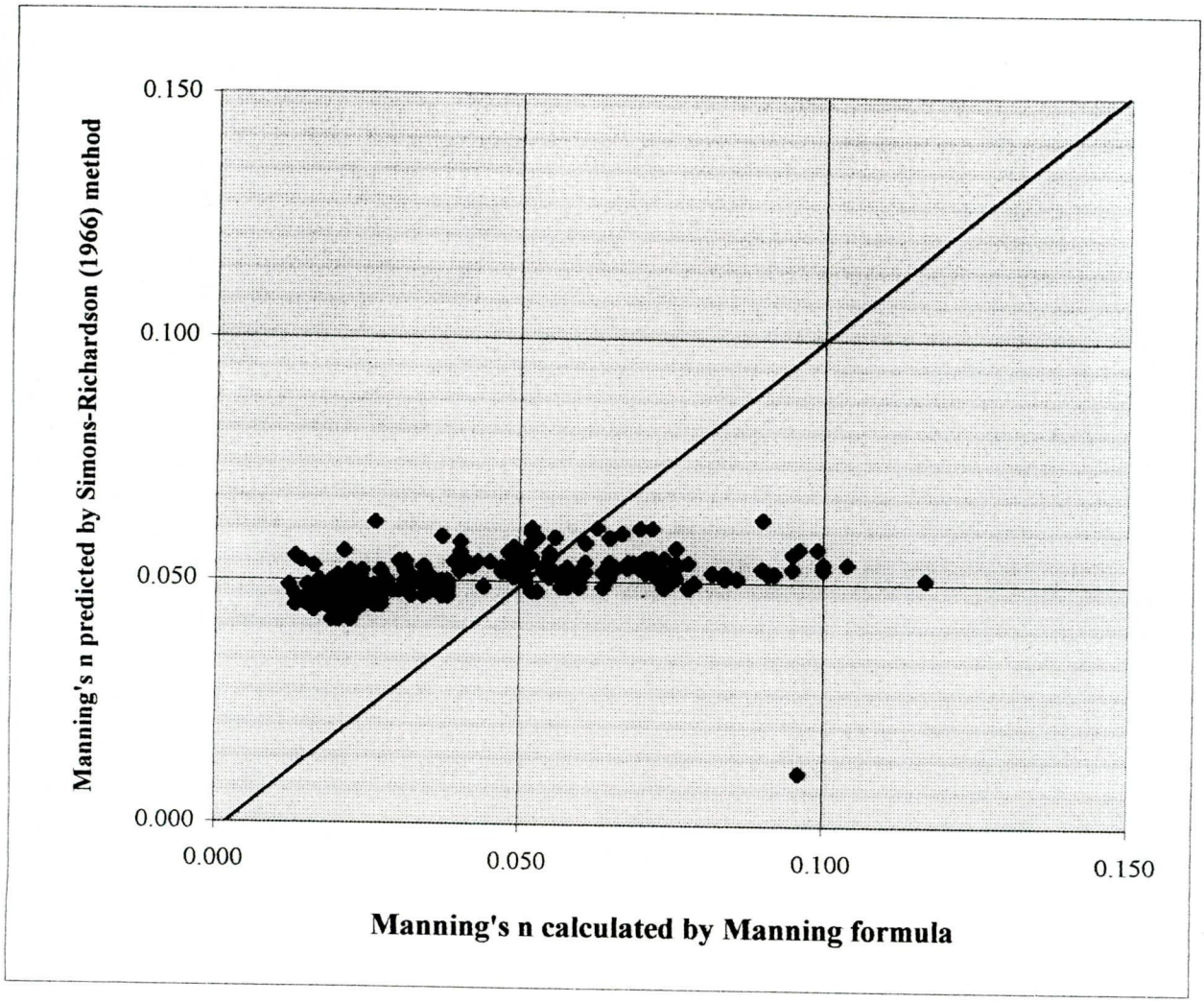

Fig.5.20	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Simons-Richardson (1966) method of the Kushiyara river**

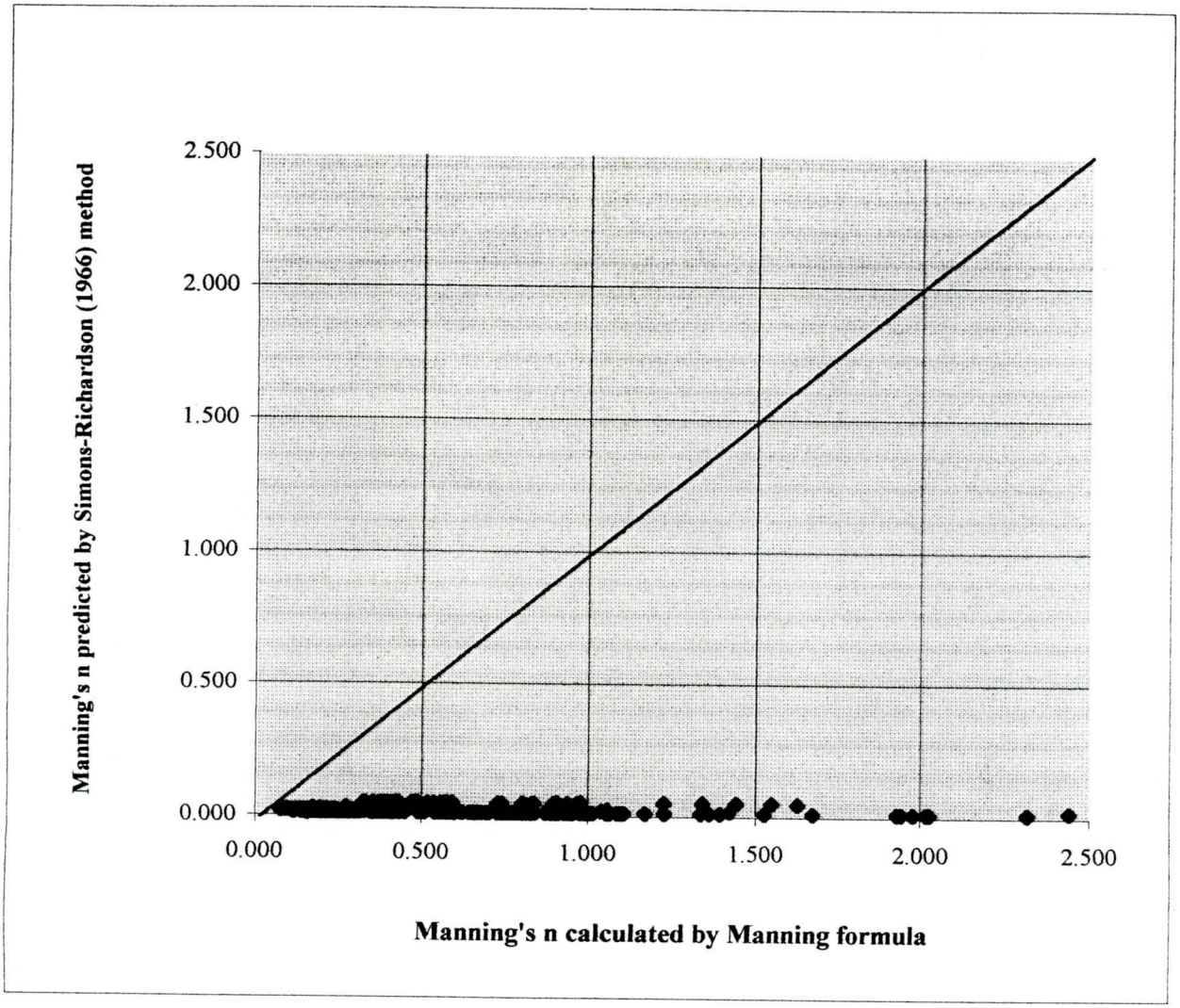

Fig.5.21	**Comparison between the values of Manning's *n* calculated by Manning formula and those predicted by Simons-Richardson (1966) method of the Mohananda river**

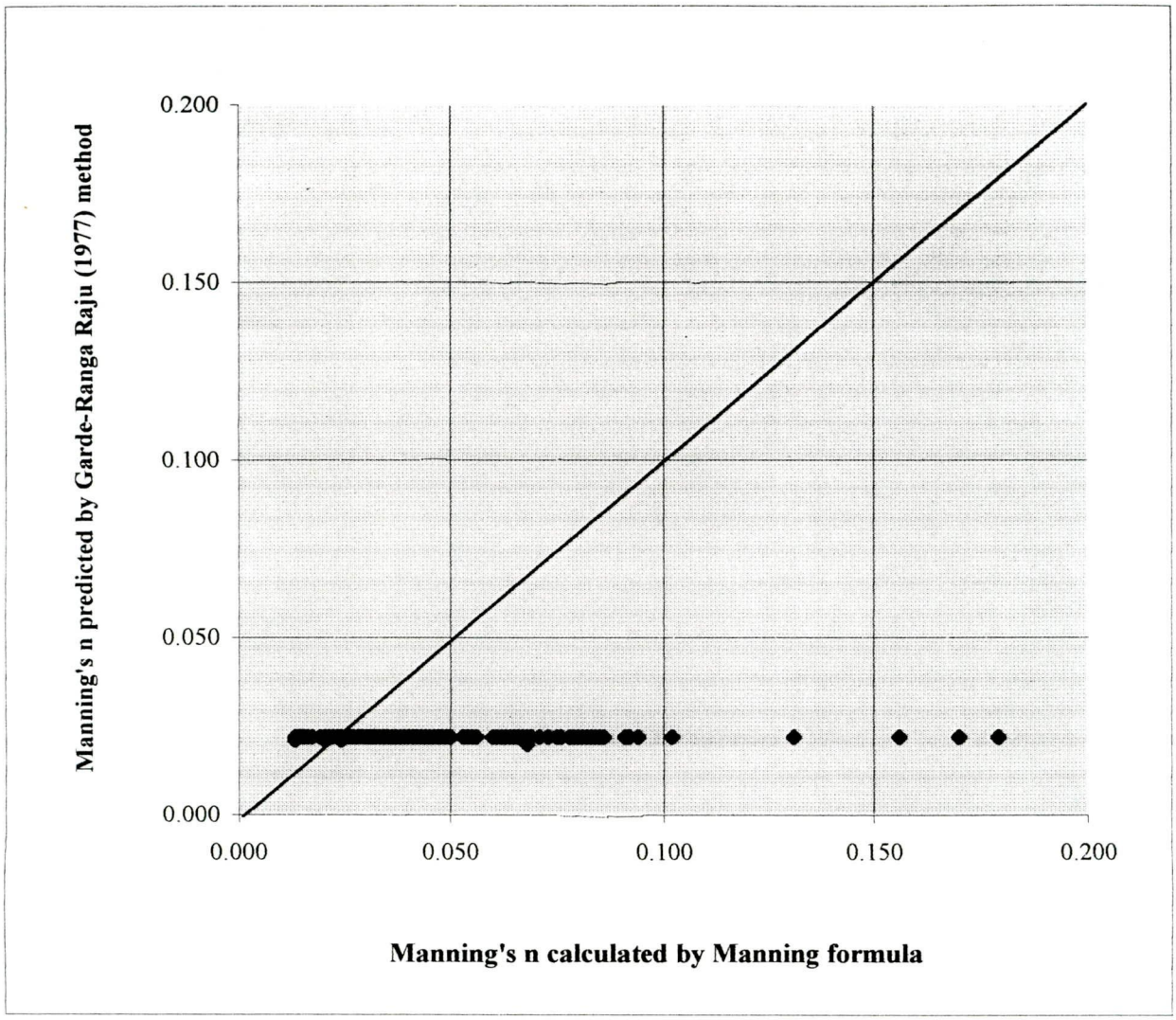

Fig.5.22	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Garde-Ranga Raju (1977) method of the Gumti river**

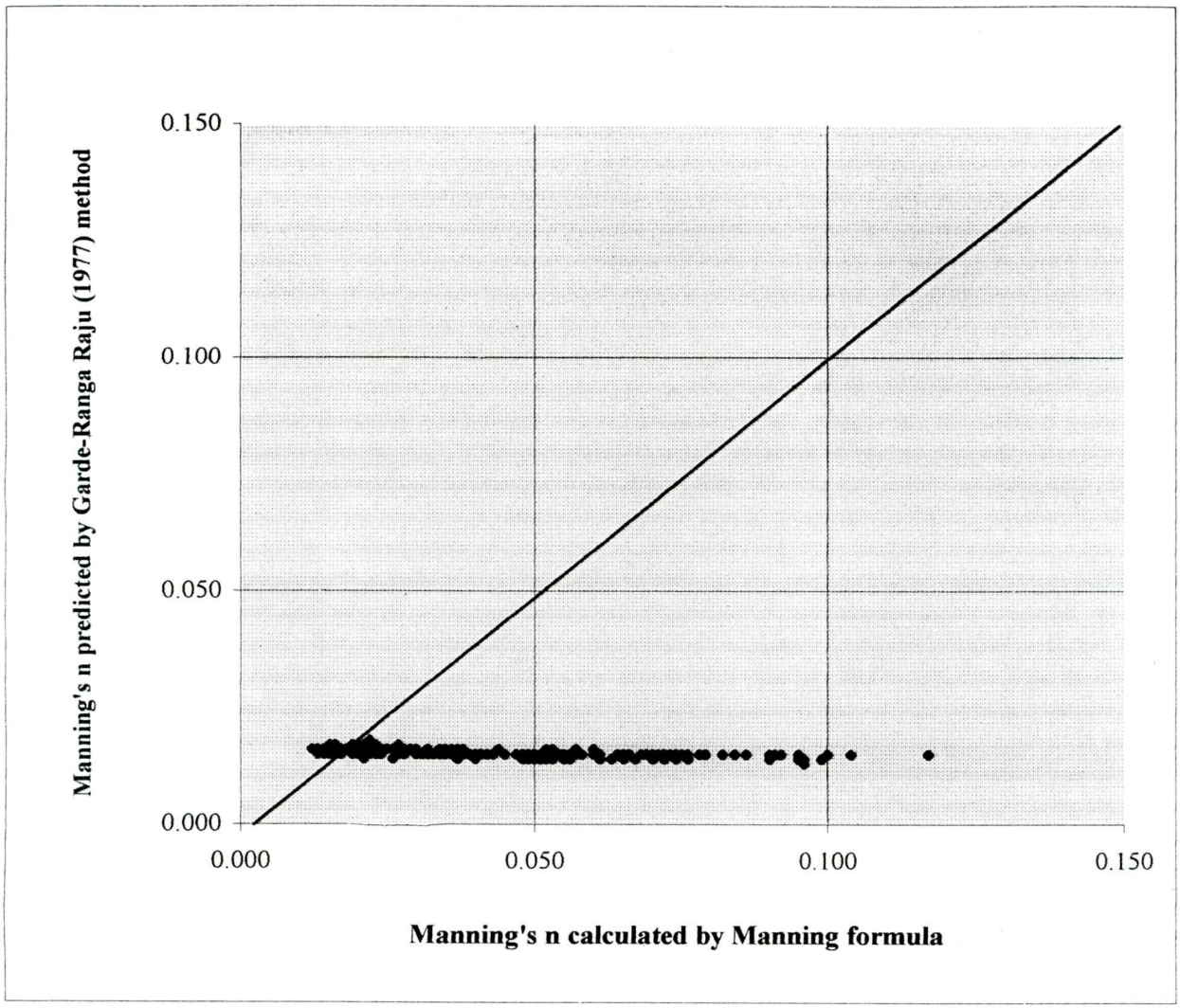

Fig.5.23	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Garde-Ranga Raju (1977) method of the Kushiyara river**

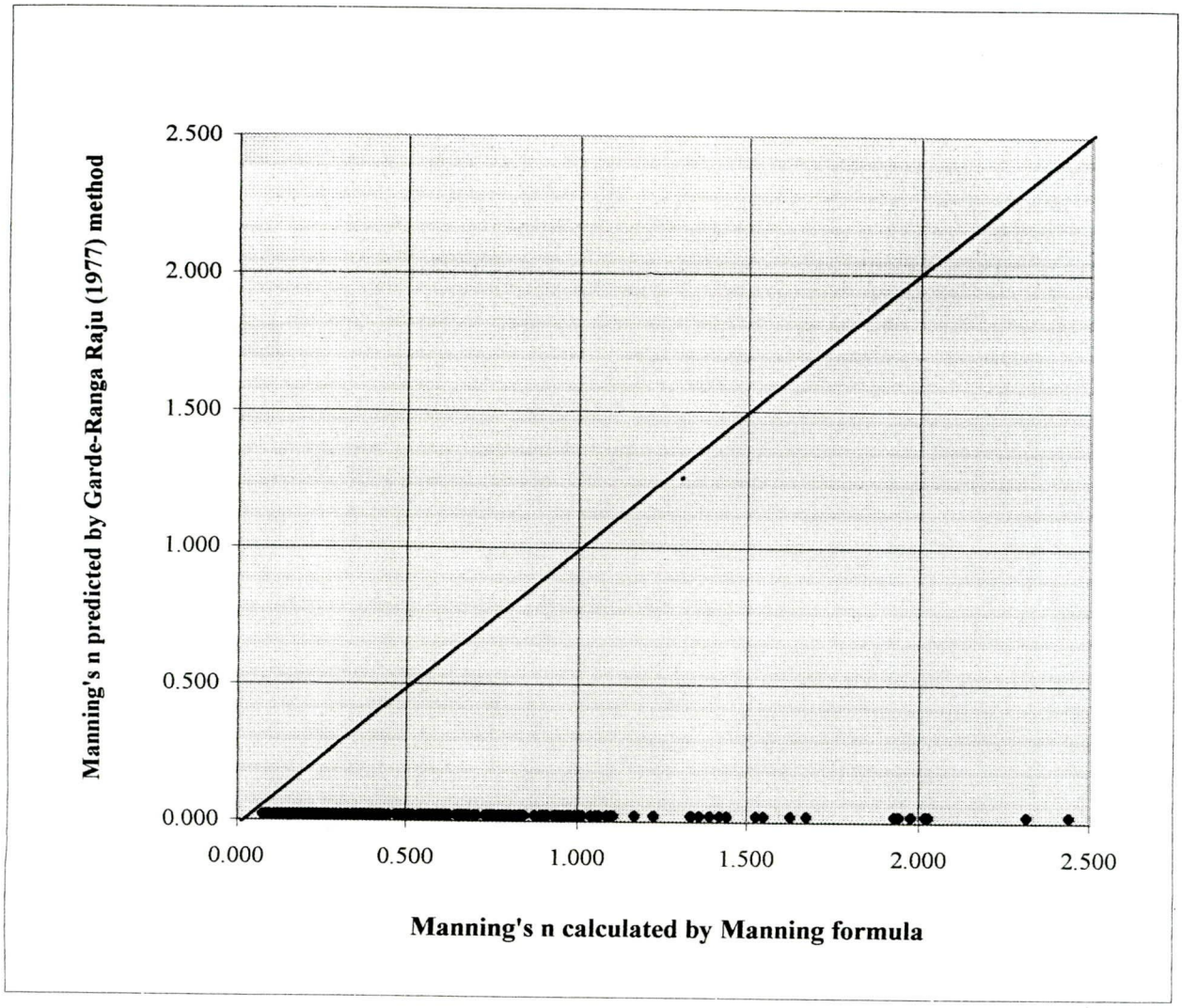

Fig.5.24	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Garde-Ranga Raju (1977) method of the Mohananda river**

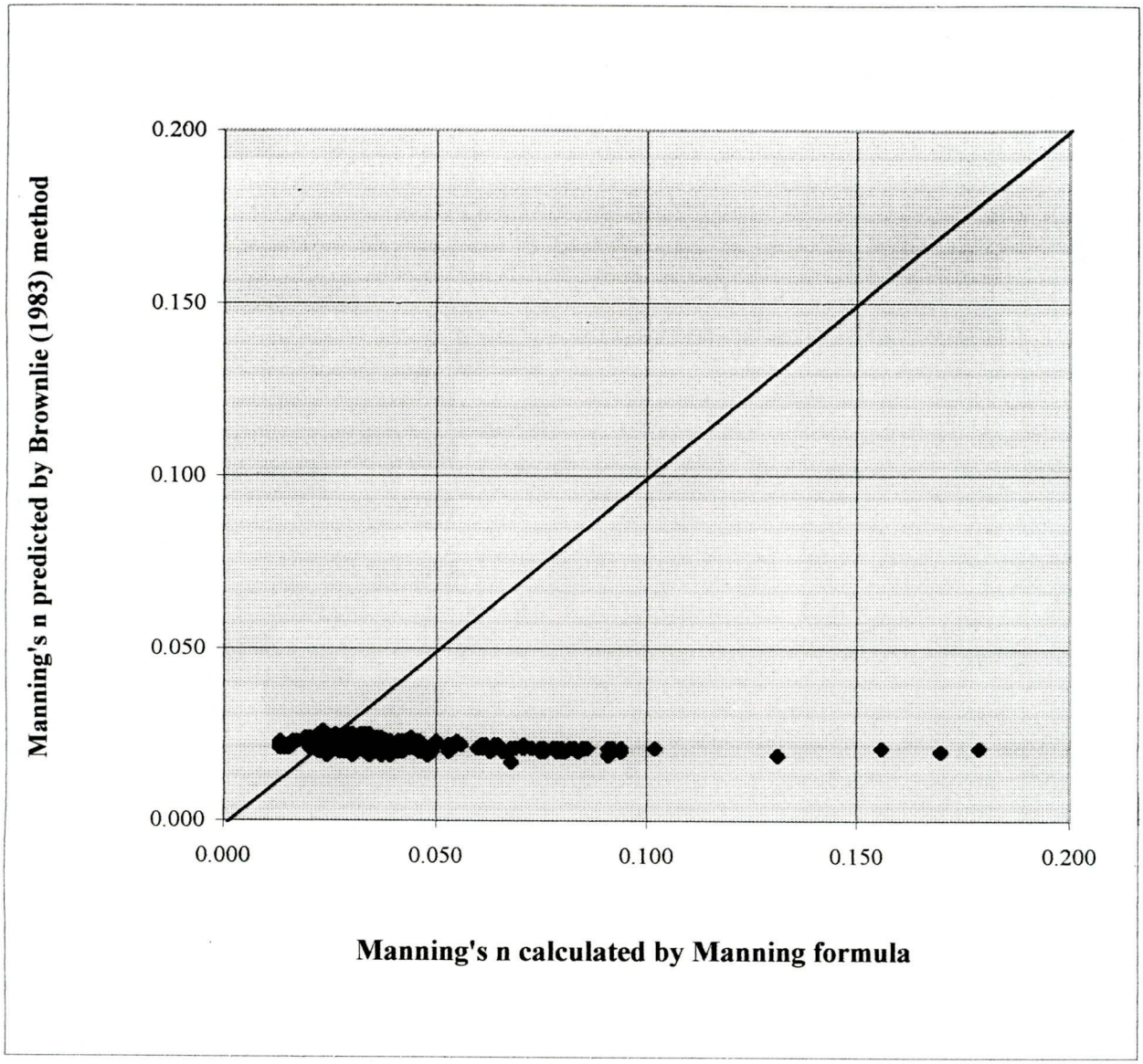

Fig.5.25	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Brownlie (1983) method of the Gumti river**

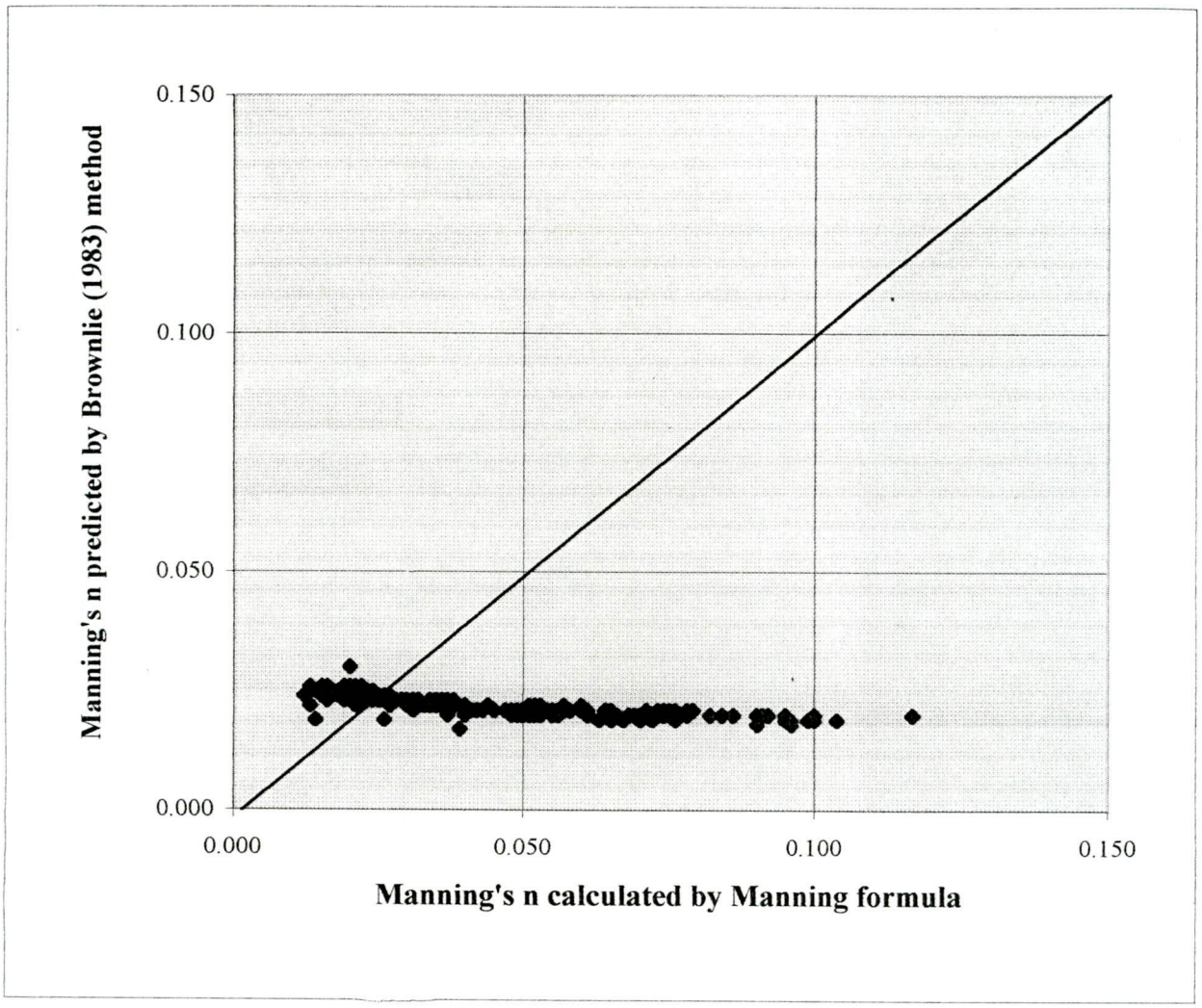

Fig.5.26	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Brownlie (1983) method of the Kushiyara river**

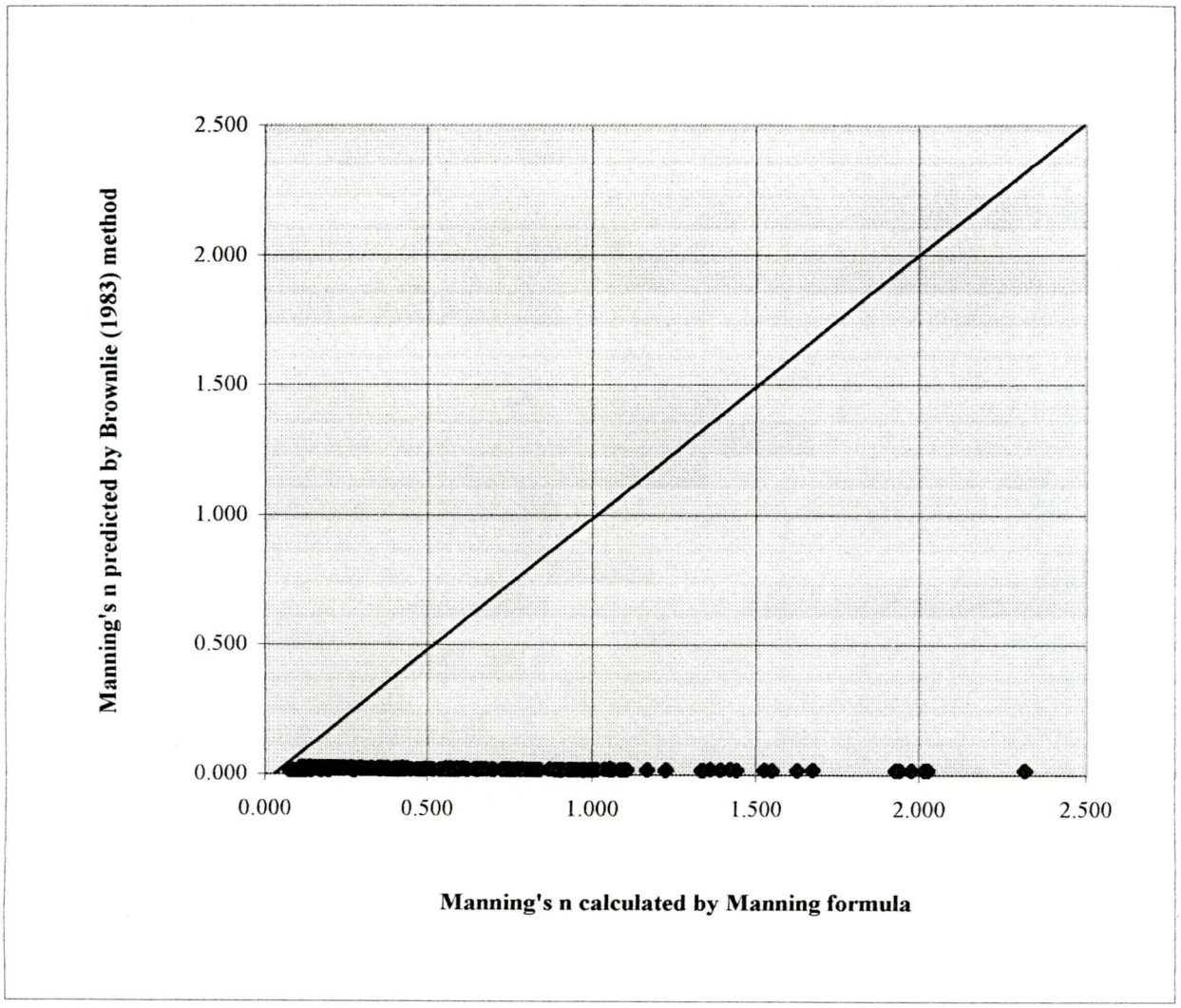

Fig.5.27 **Comparison between the values of Manning's n calculated by Manning formula and those predicted by Brownlie (1983) method of the Mohananda river**

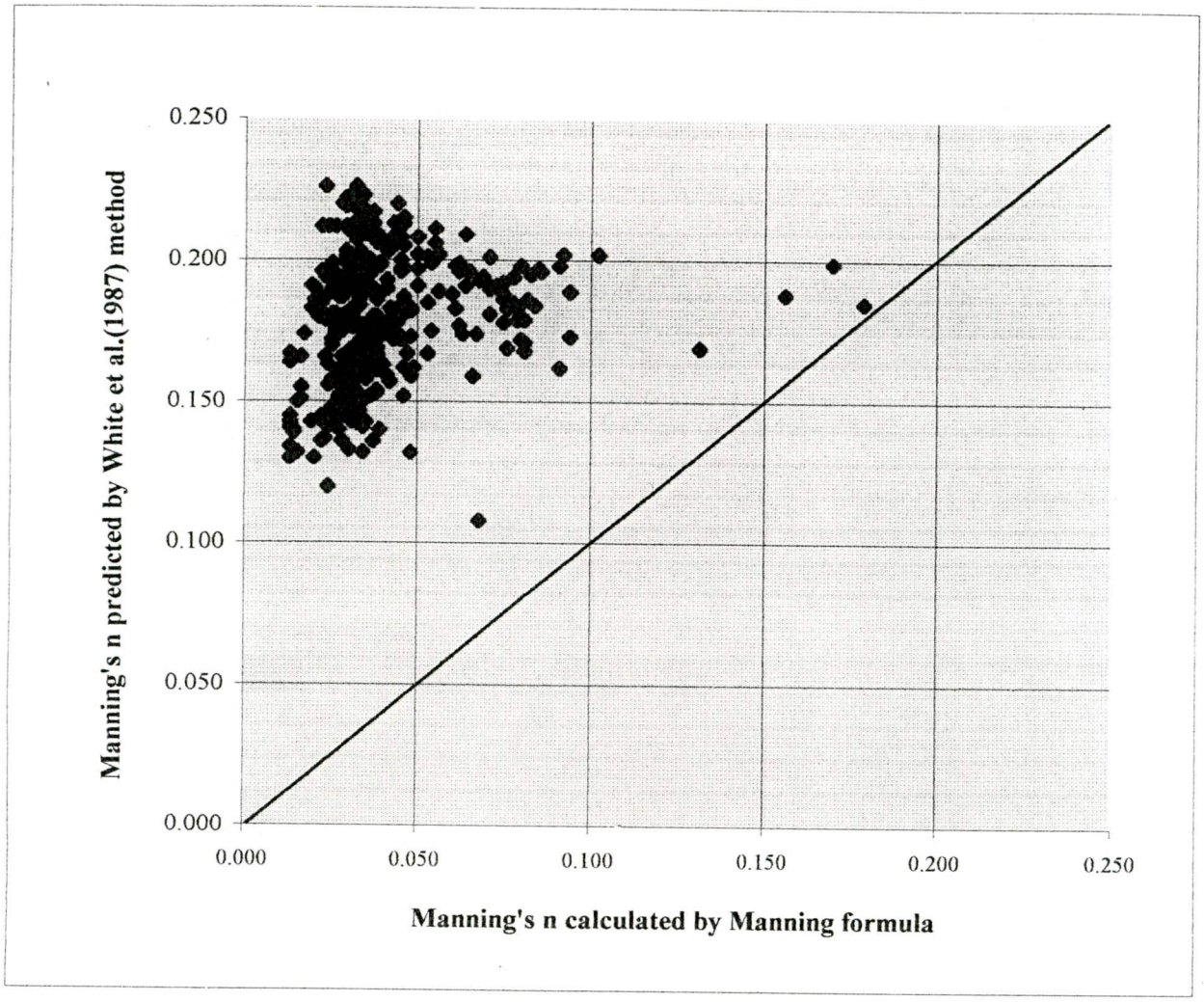

Fig.5.28	Comparison between the values of Manninig's n calculated by Manning formula and those predicted by White et al. (1987) method of the Gumti river

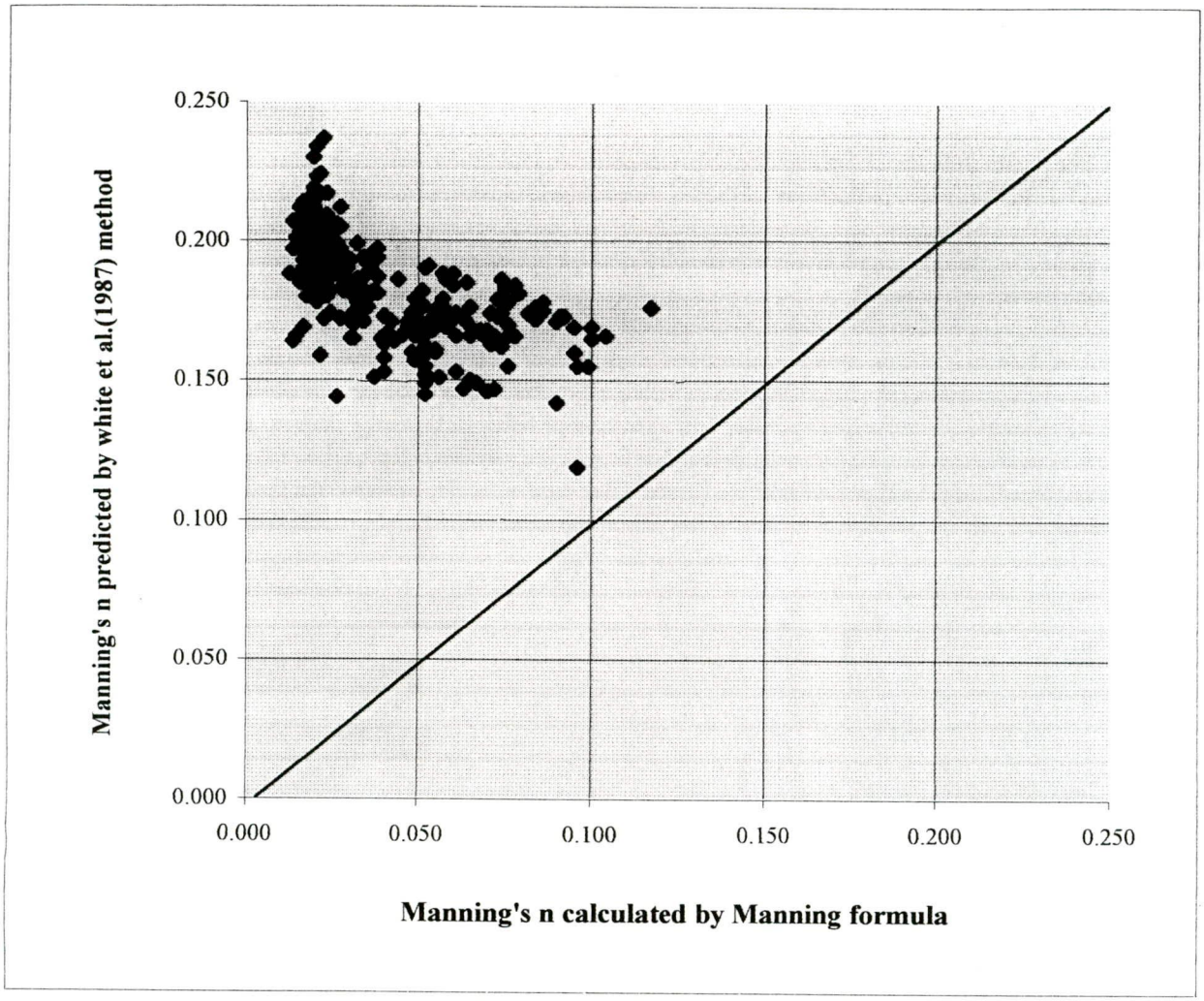

Fig.5.29	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by White et al. (1987) method of the Kushiyara river**

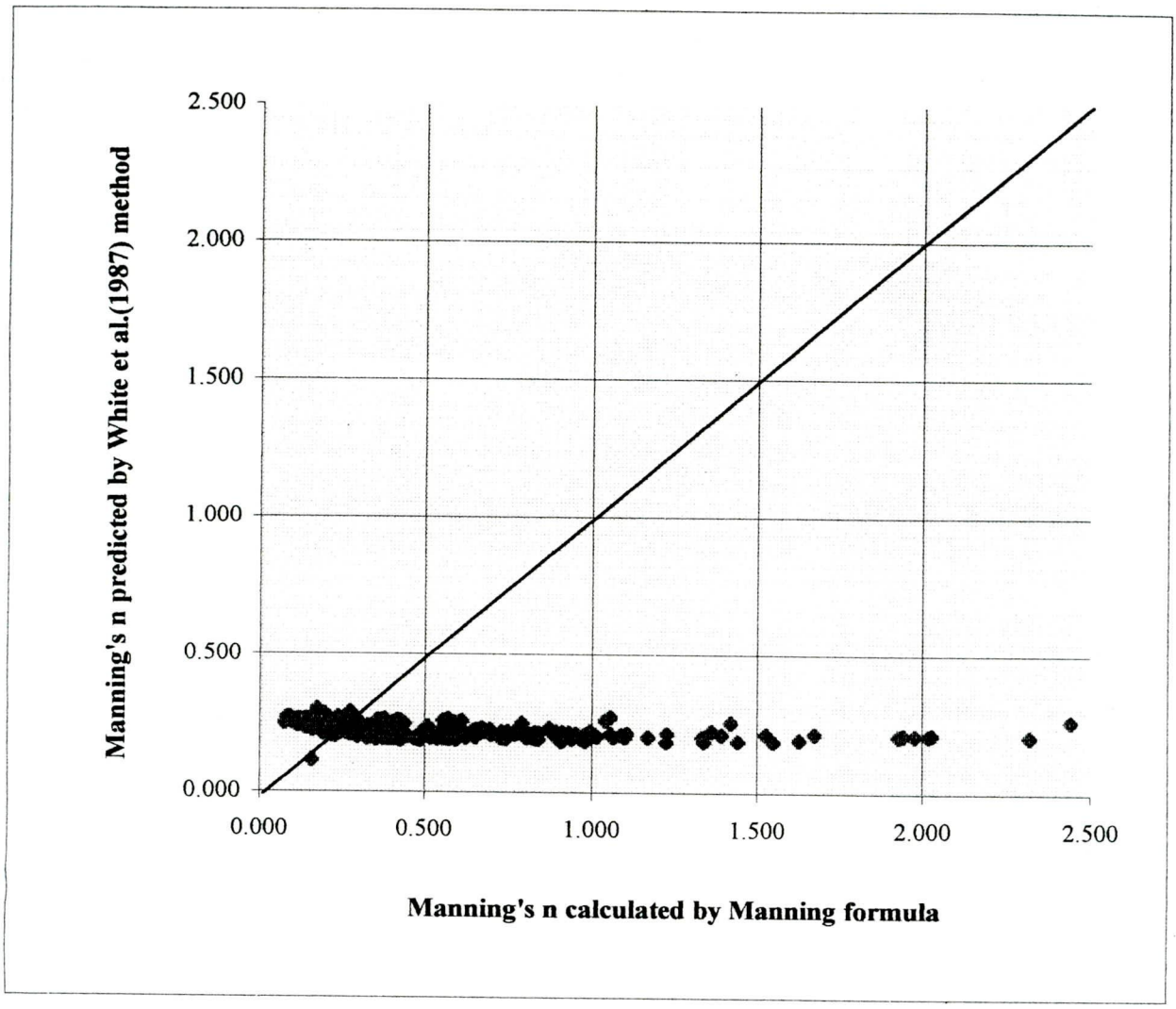

Fig.5.30	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by White et al. (1987) method of the Mohananda river**

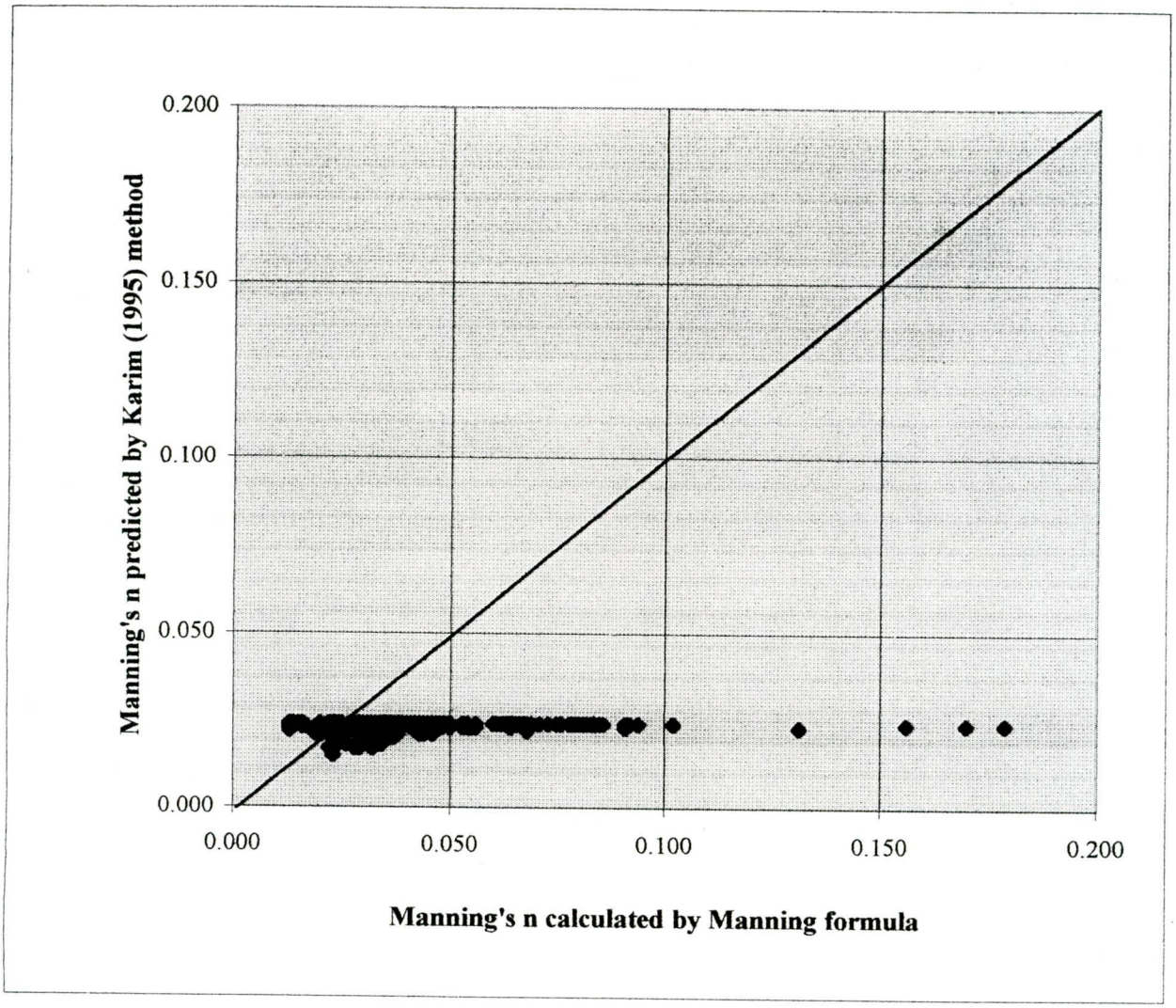

Fig.5.31	**Coparison between the values of Manning's n calculated by Manning formula and those predicted by Karim (1995) method of the Gumti river**

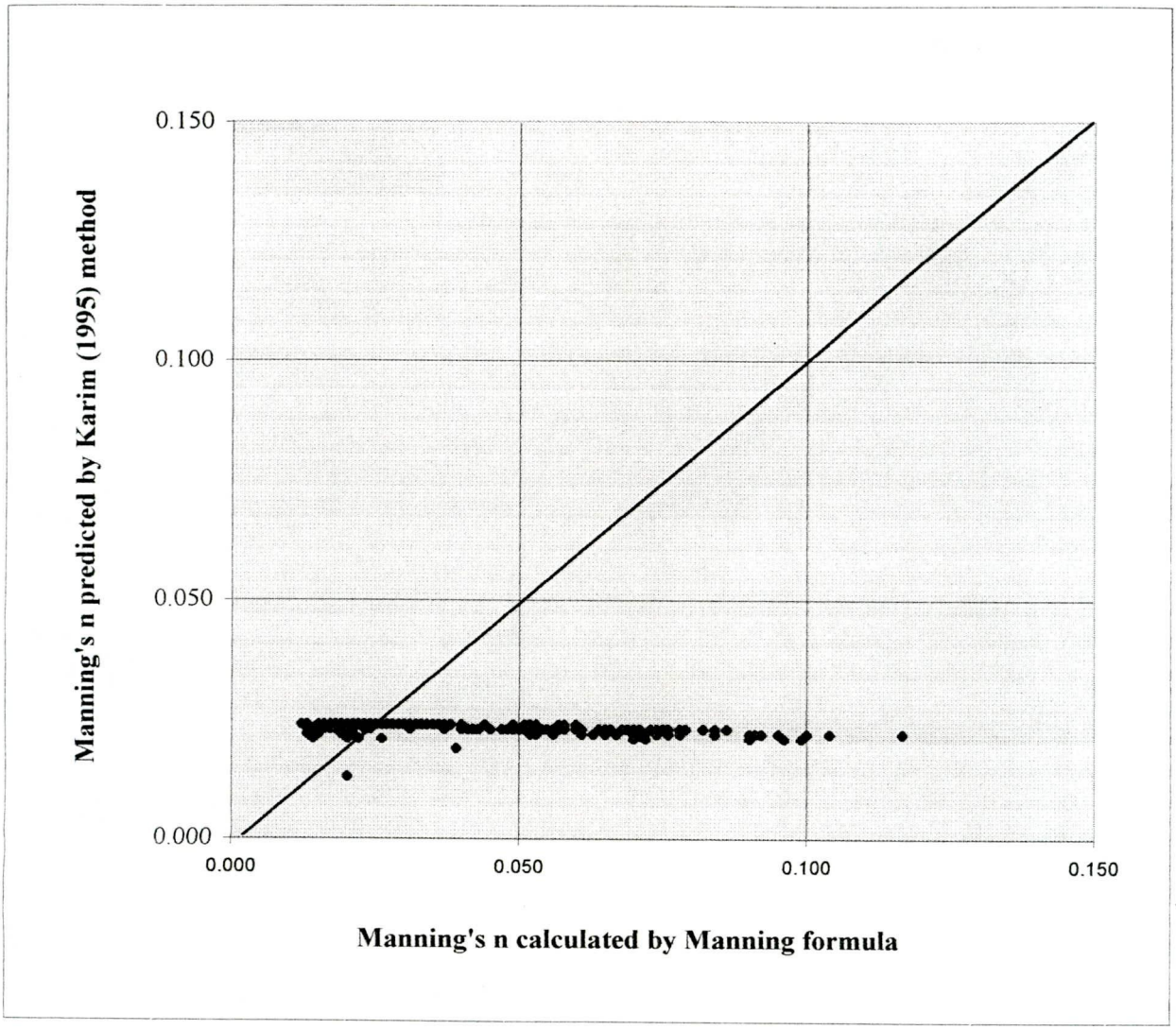

Fig.5.32	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Karim (1995) method of the Kushiyara river**

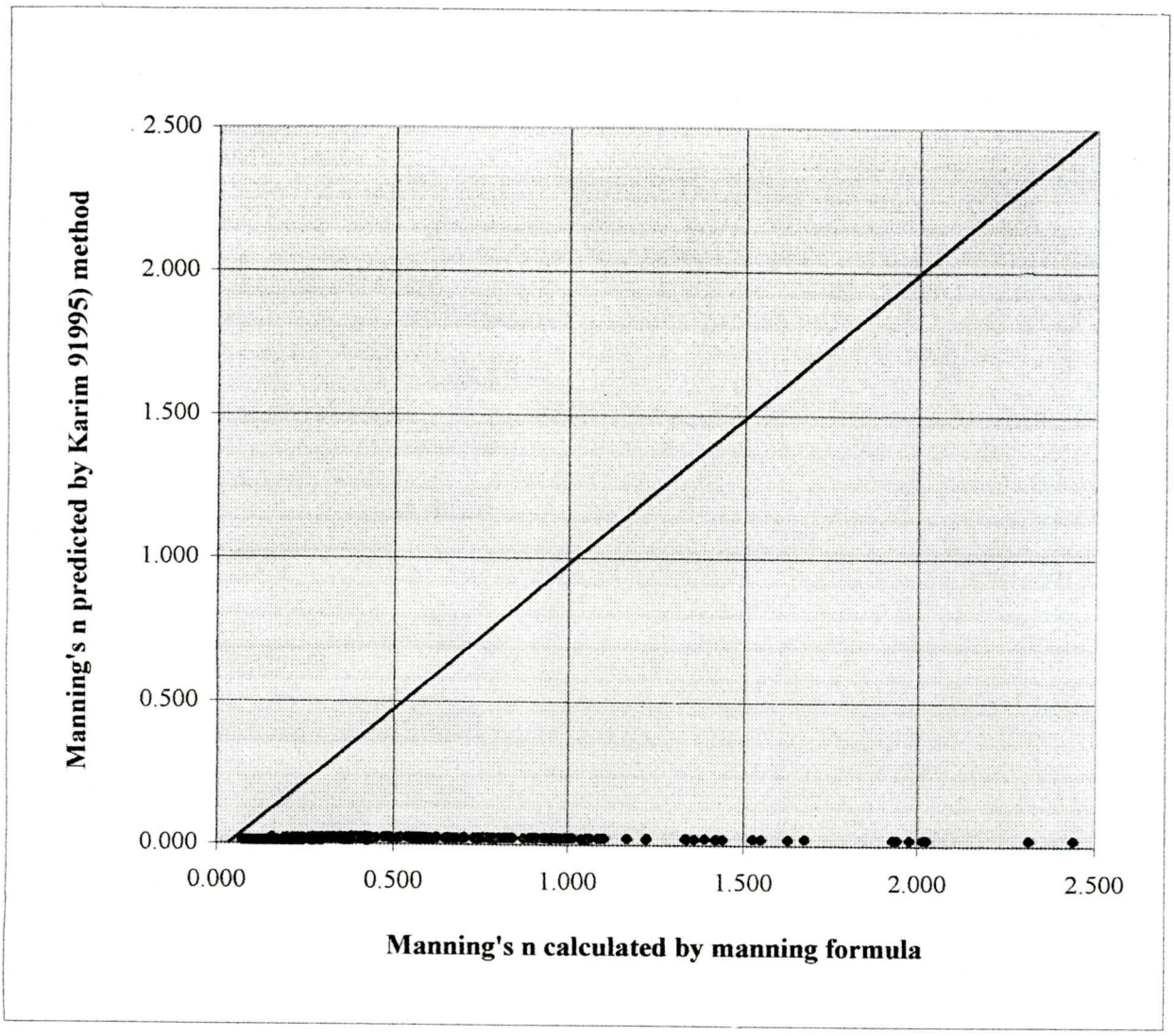

Fig.5.33	**Comparison between the values of Manning's n calculated by Manning formula and those predicted by Karim (1995) method of the Mohananda river**

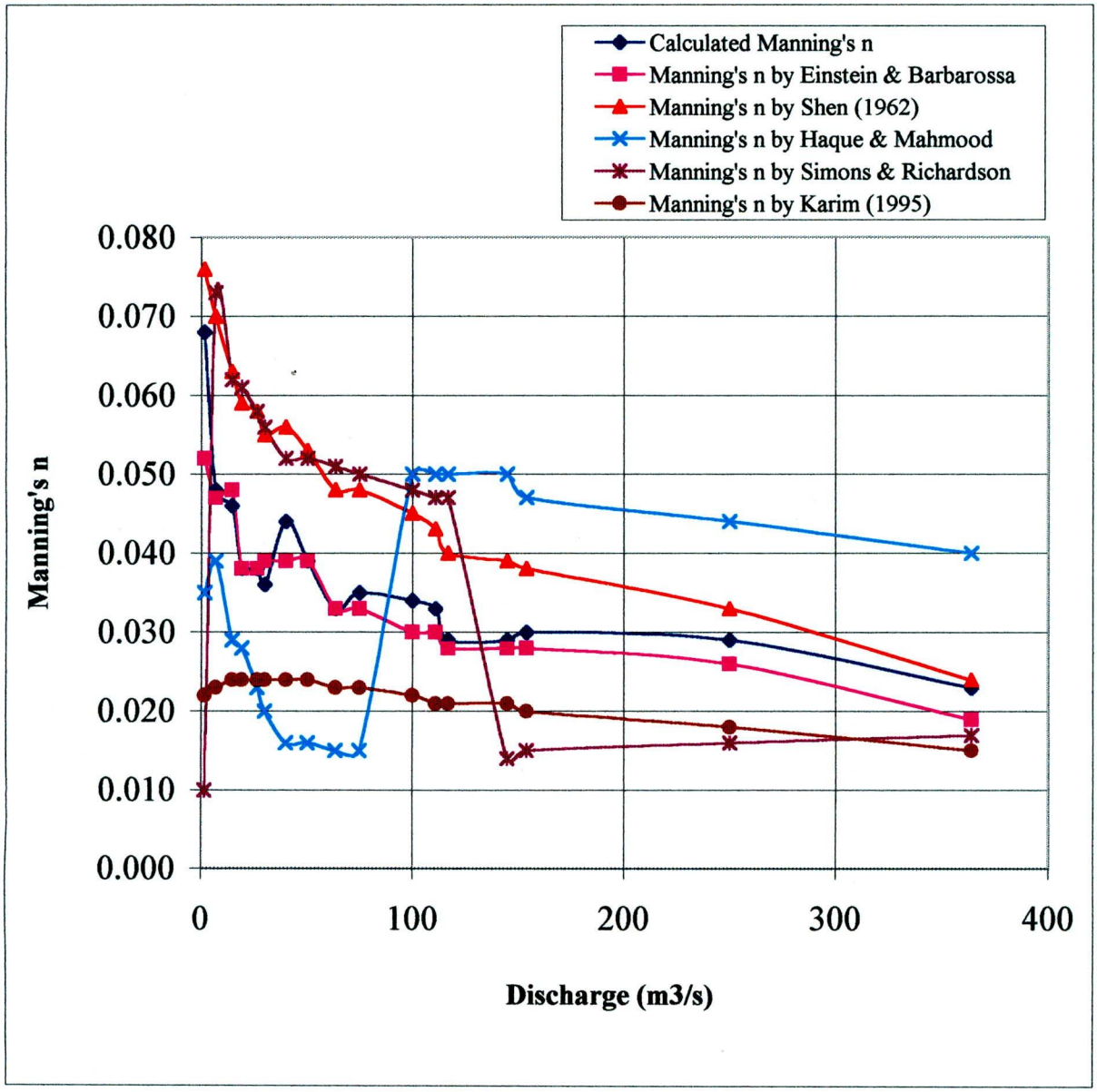

Fig.5.34	Comparison of the calculated Manning's n with the predicted Manning's n using different roughness predictors for the Gumti river

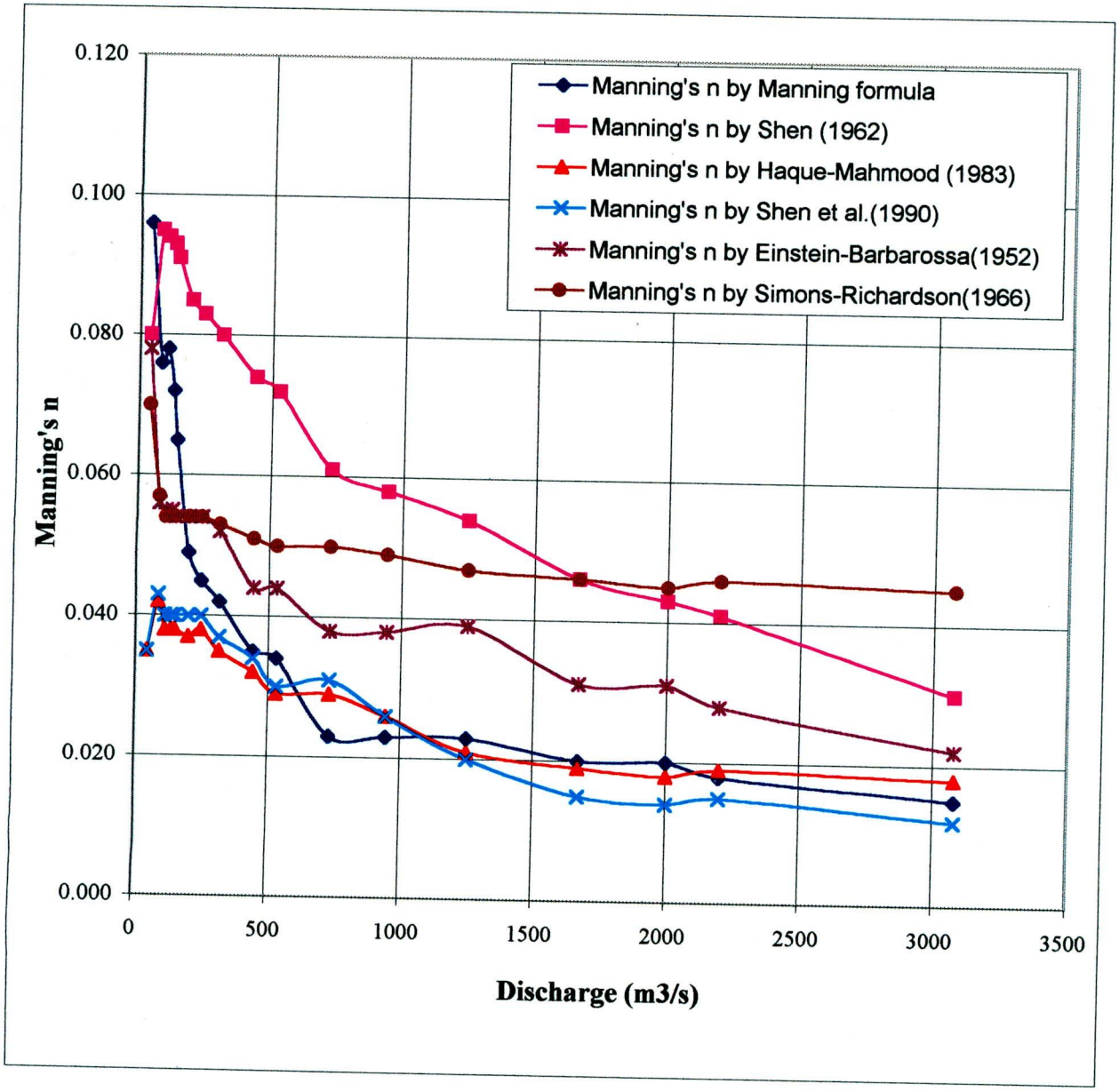

Fig.5.35	**Comparison of the calculated Manning's n with the predicted Manning's n using different roughness predictors for the Kushiyara river**

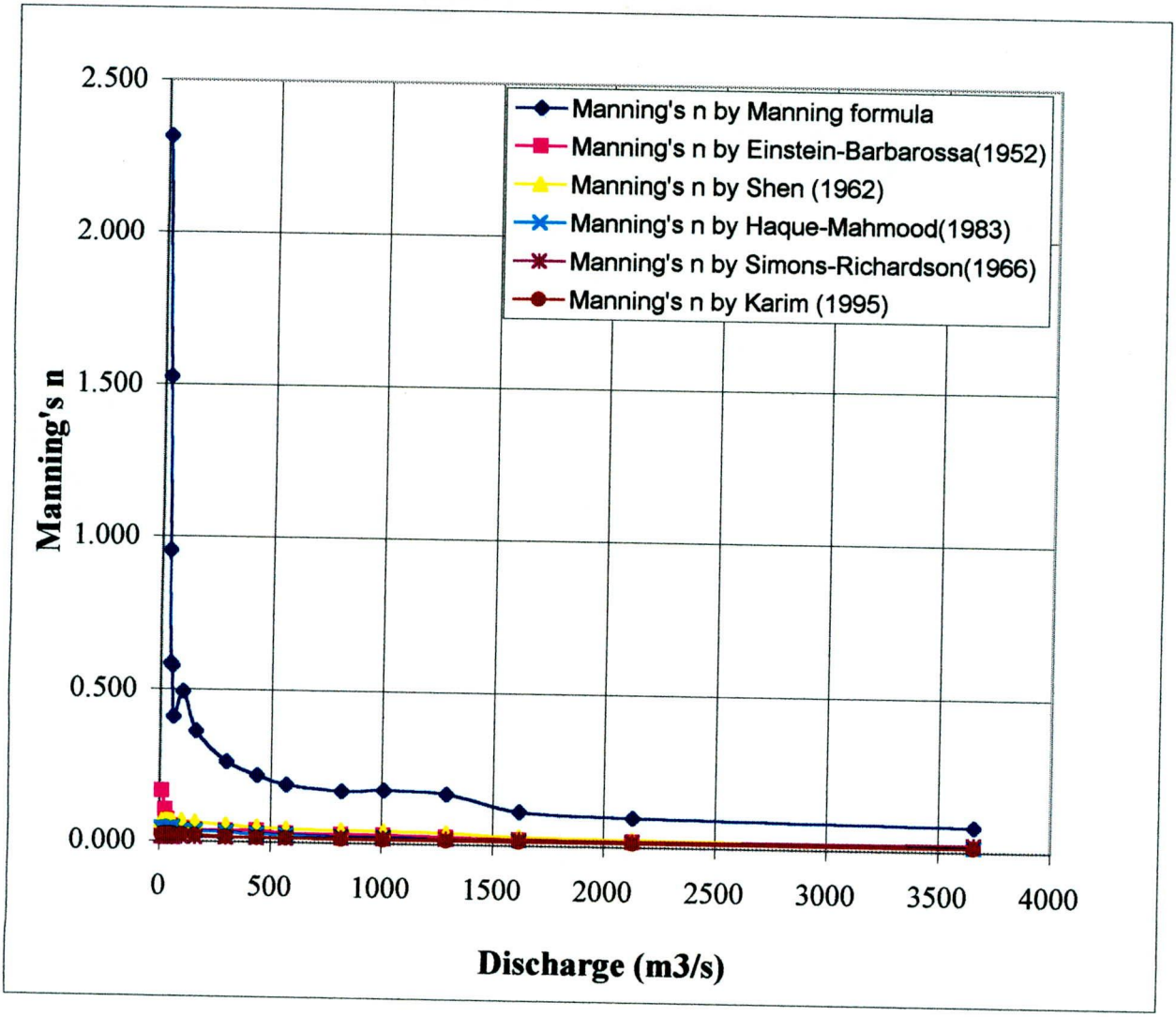

Fig.5.36(a)	**Comparison of the calculated Manning's n with the Manning's n predicted by different roughness predictors for the Mohananda river**

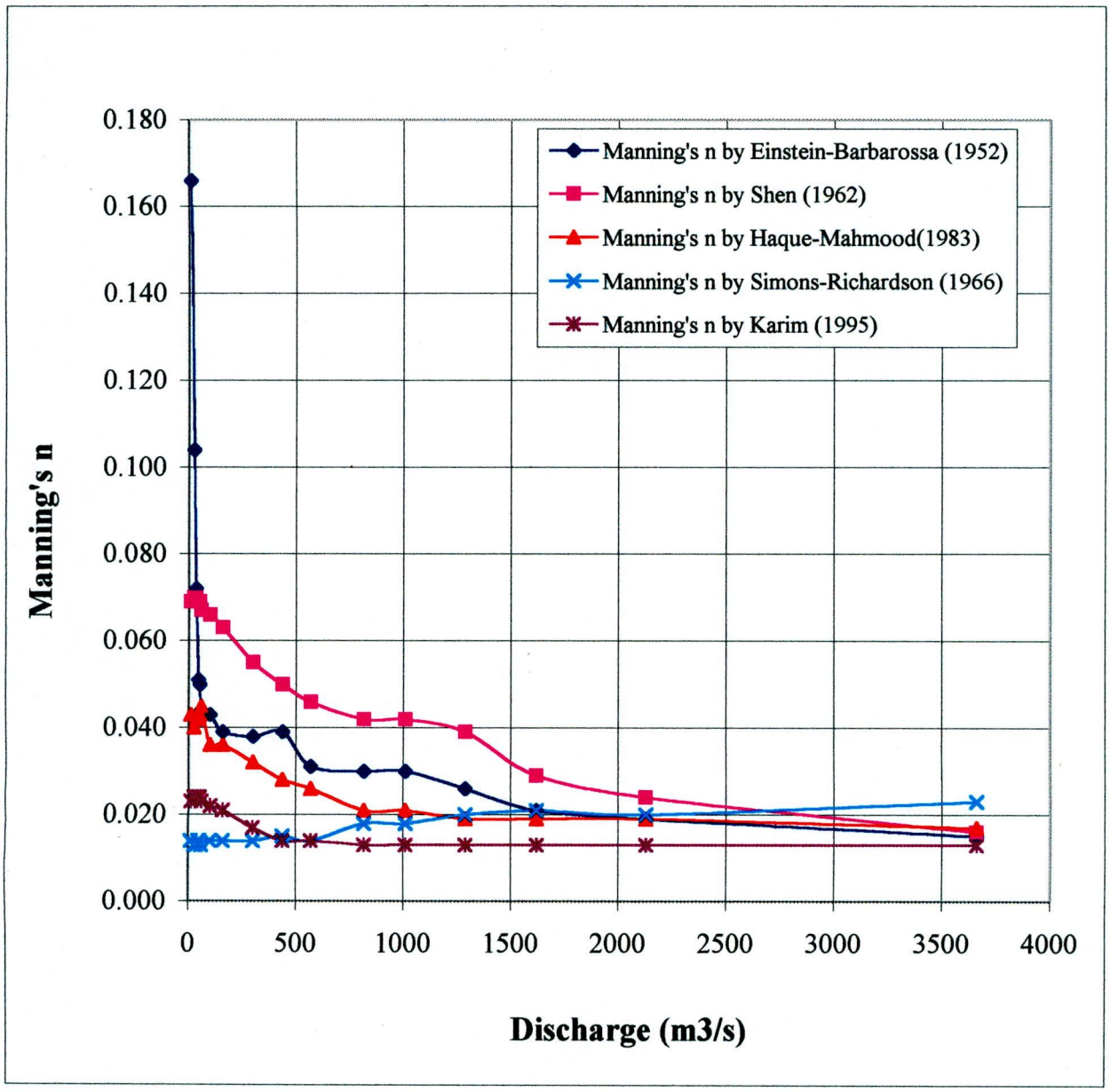

Fig.5.36(b)	**Comparison between the predicted Manning's n using different roughness predictors of the Mohananda river**

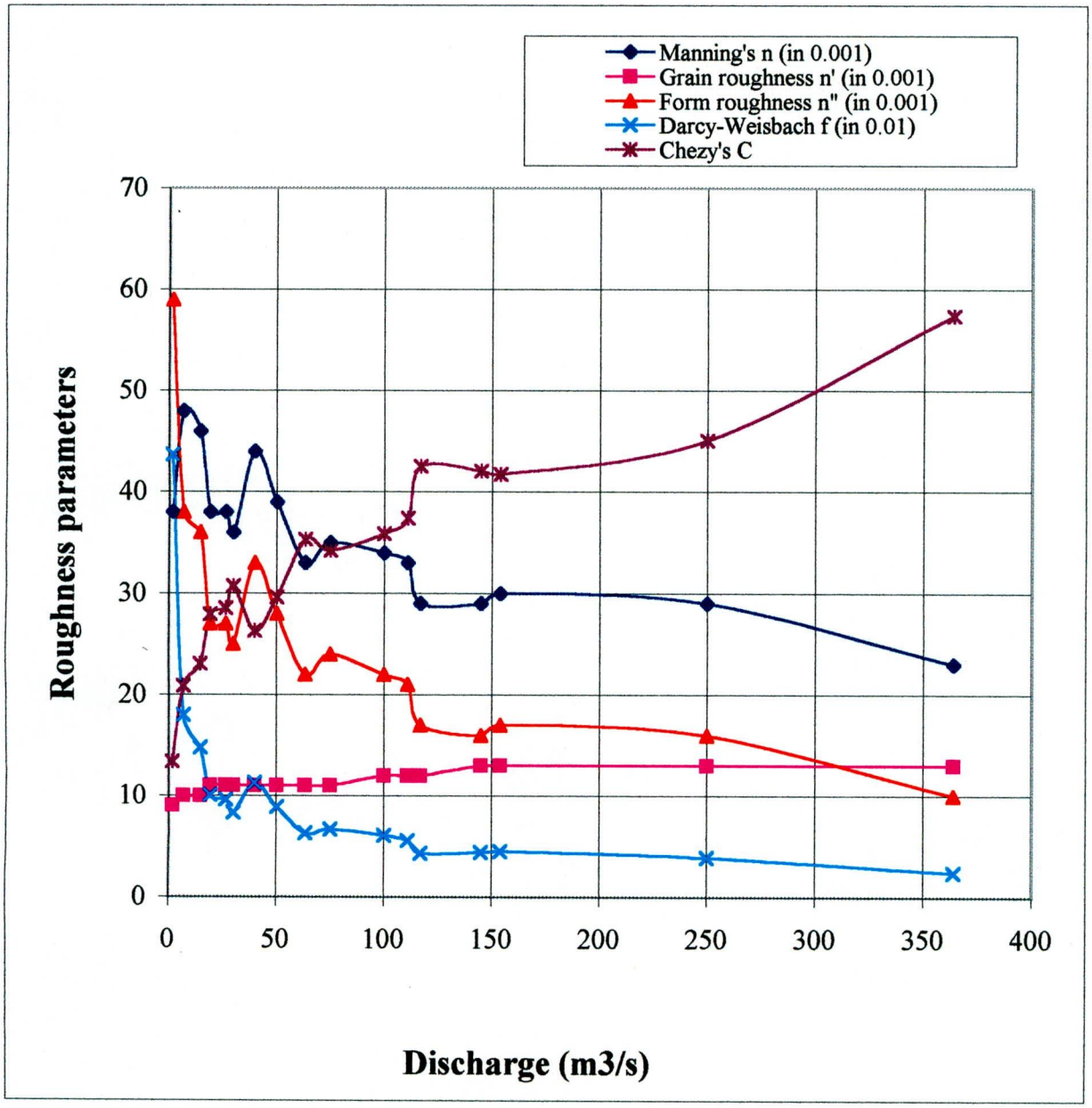

Fig.5.37	Variation of different roughness parameters with discharge of the river Gumti at station Jibanpur

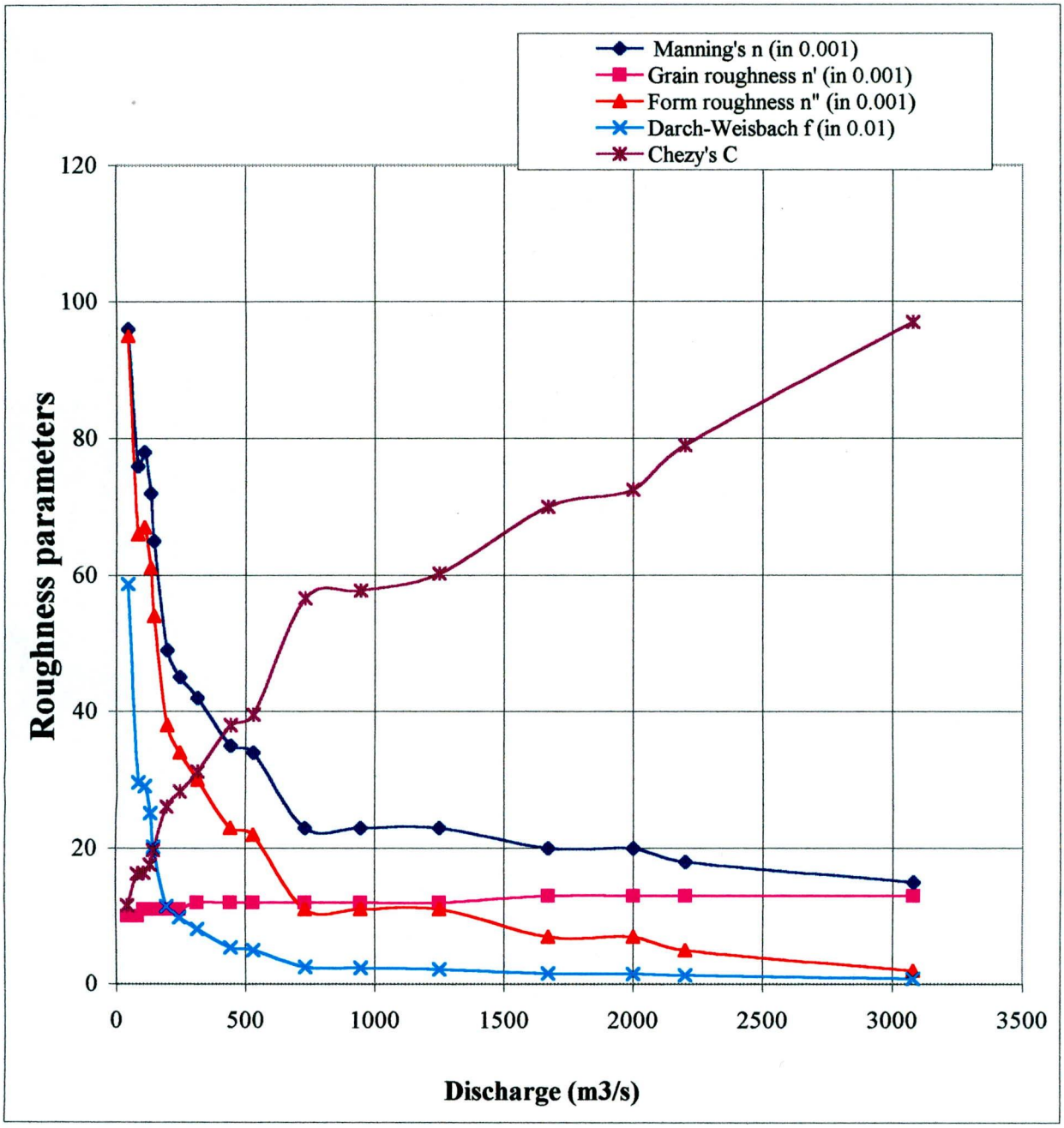

Fig.5.38	Variation of different roughness parameters with discharge for the river Kushiyara at Sherpur station

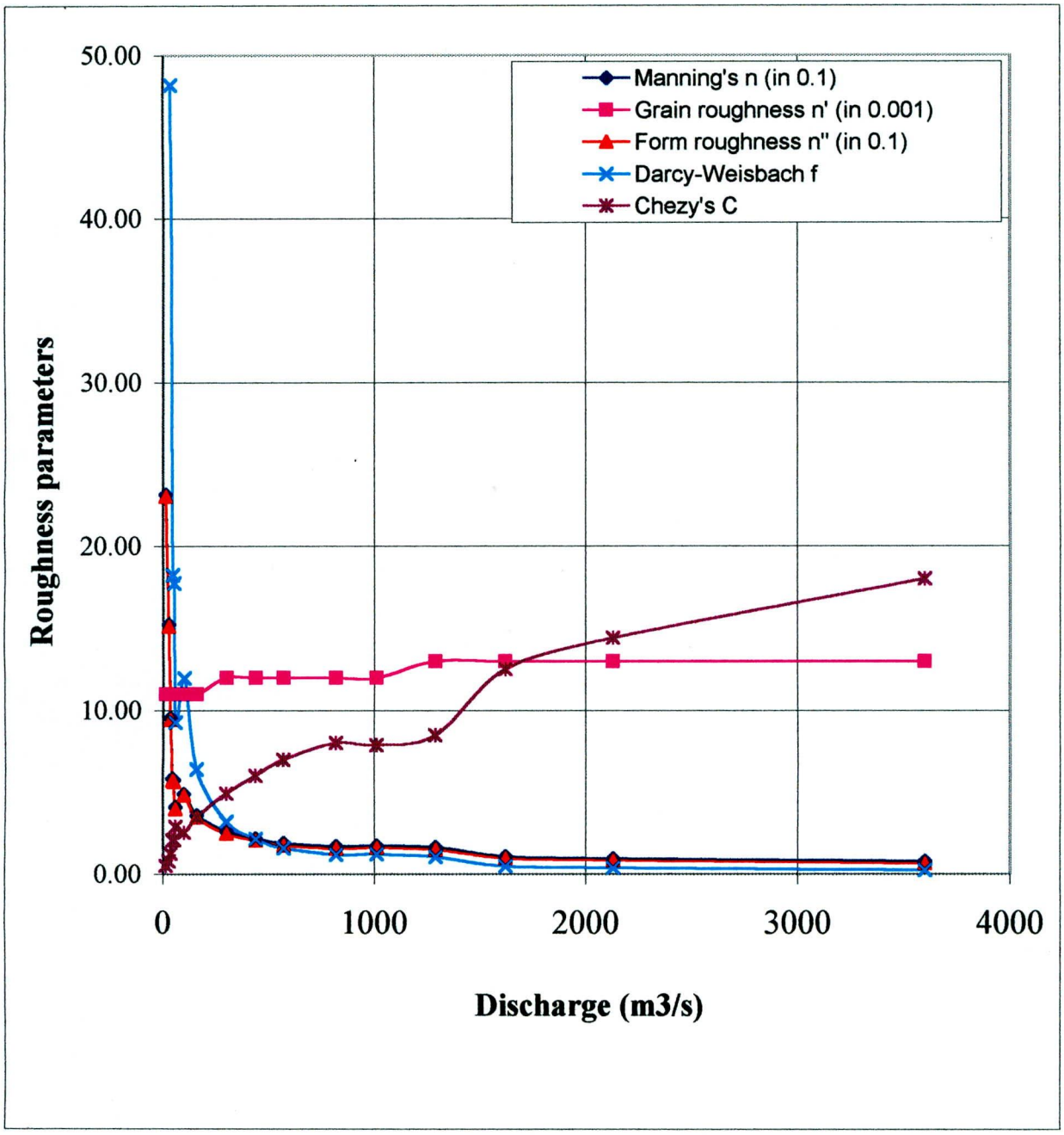

Fig.5.39	**Variation of different roughness parameters with discharge for thethe river Mohananda at Chapai Nawabganj station**

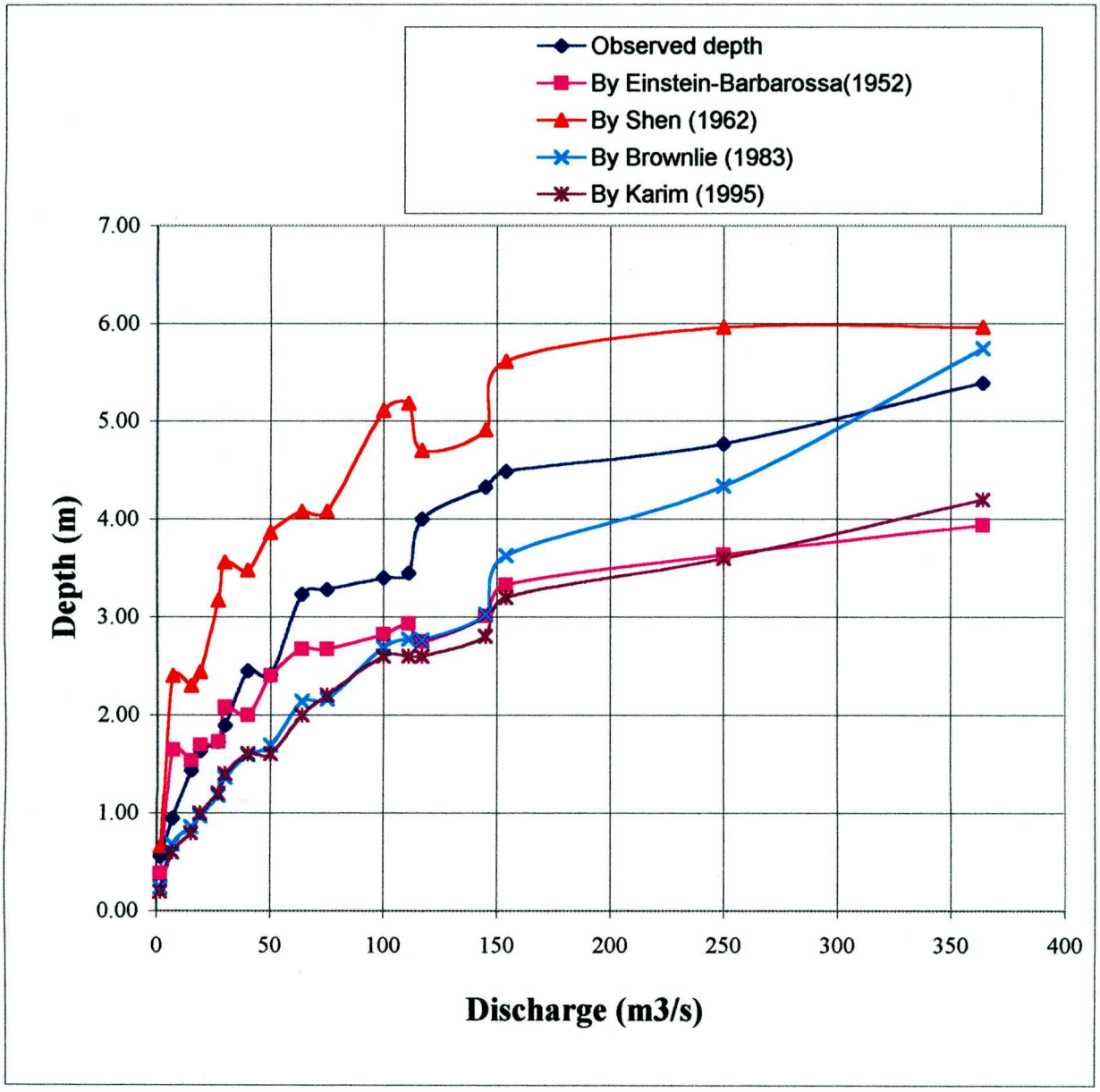

Fig.5.40	**Comparison of the observed depth with the predicted depth using different predictors for the Gumti river**

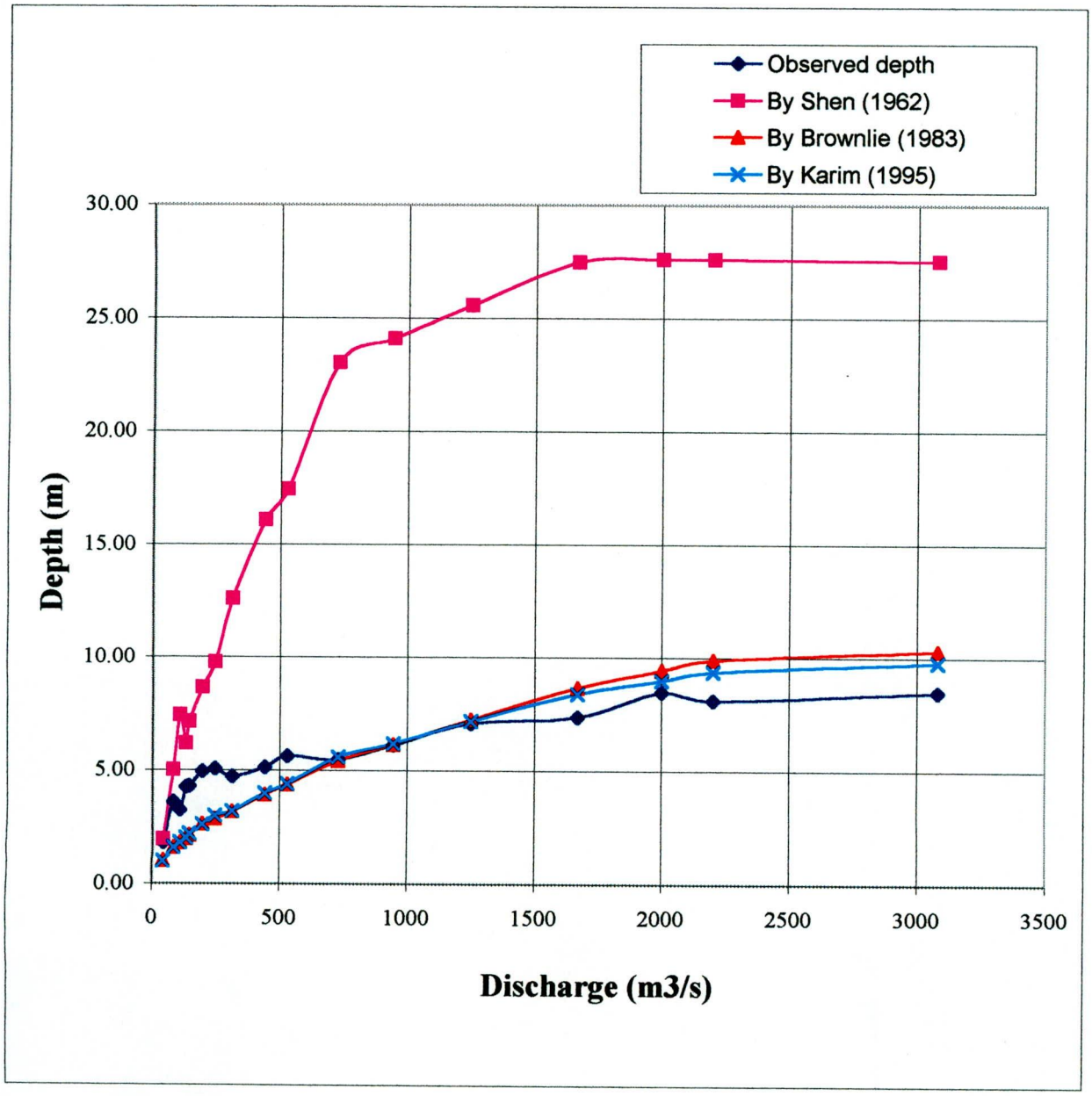

Fig.5.41	Comparison of the observed depth with the predicted depth using different predictors for the Kushiyara river

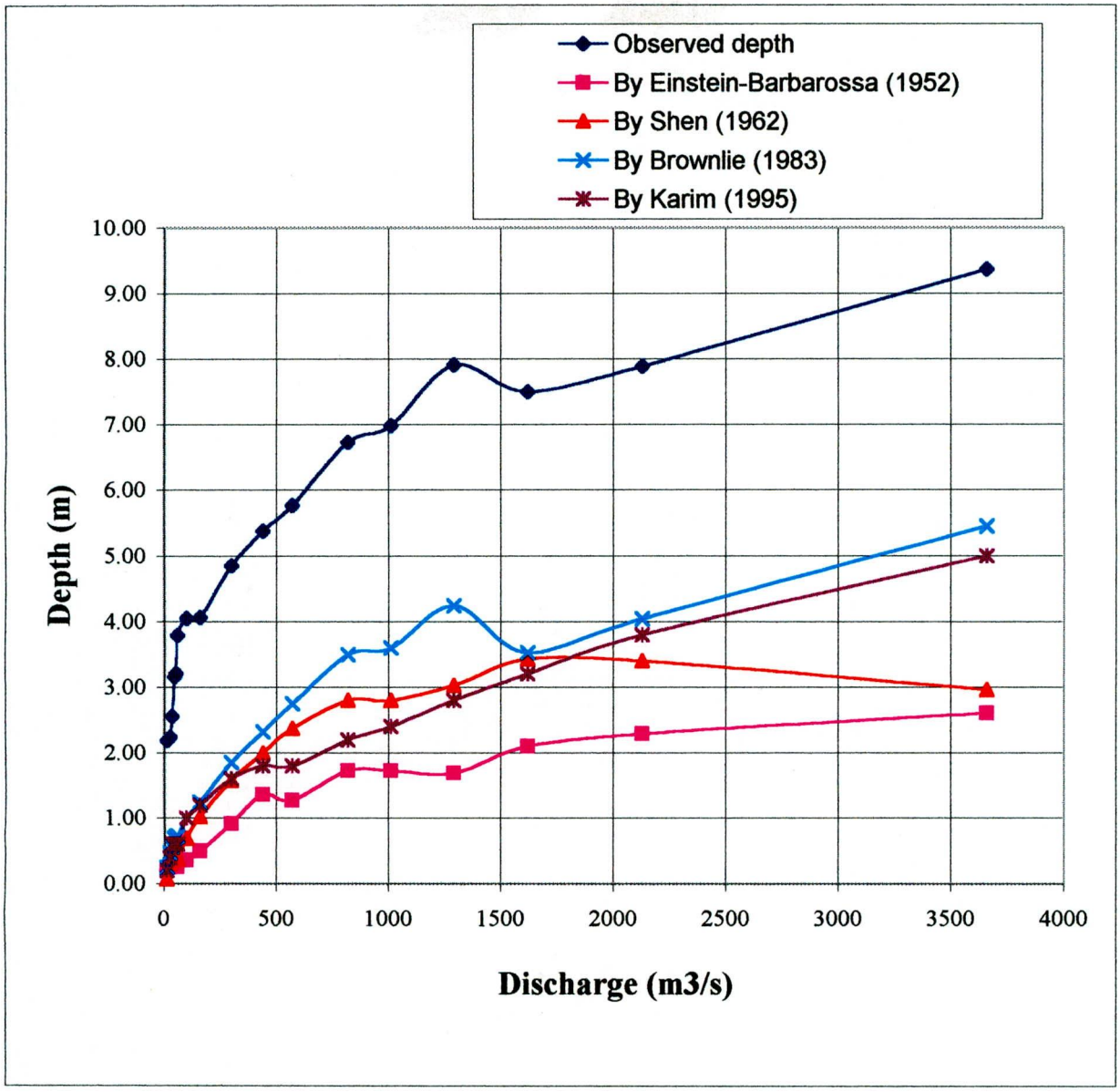

Fig.5.42	**Comparison of the observed depth with the predicted depth using different predictors for the Mohananda river**

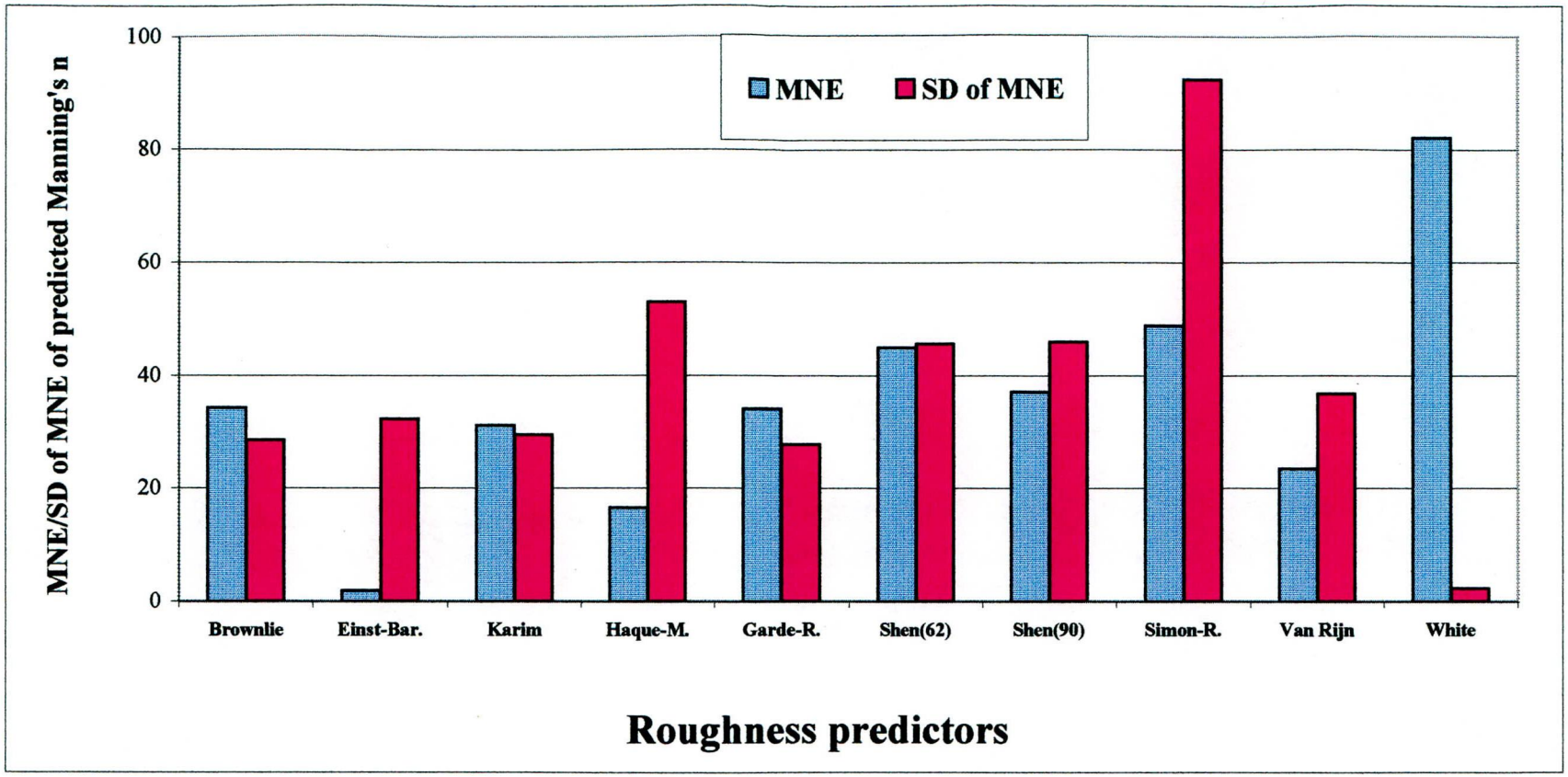

Fig.5.43	**The MNE and standard deviation of MNE of the predicted Manning's n using different roughness predictors for the Gumti river**

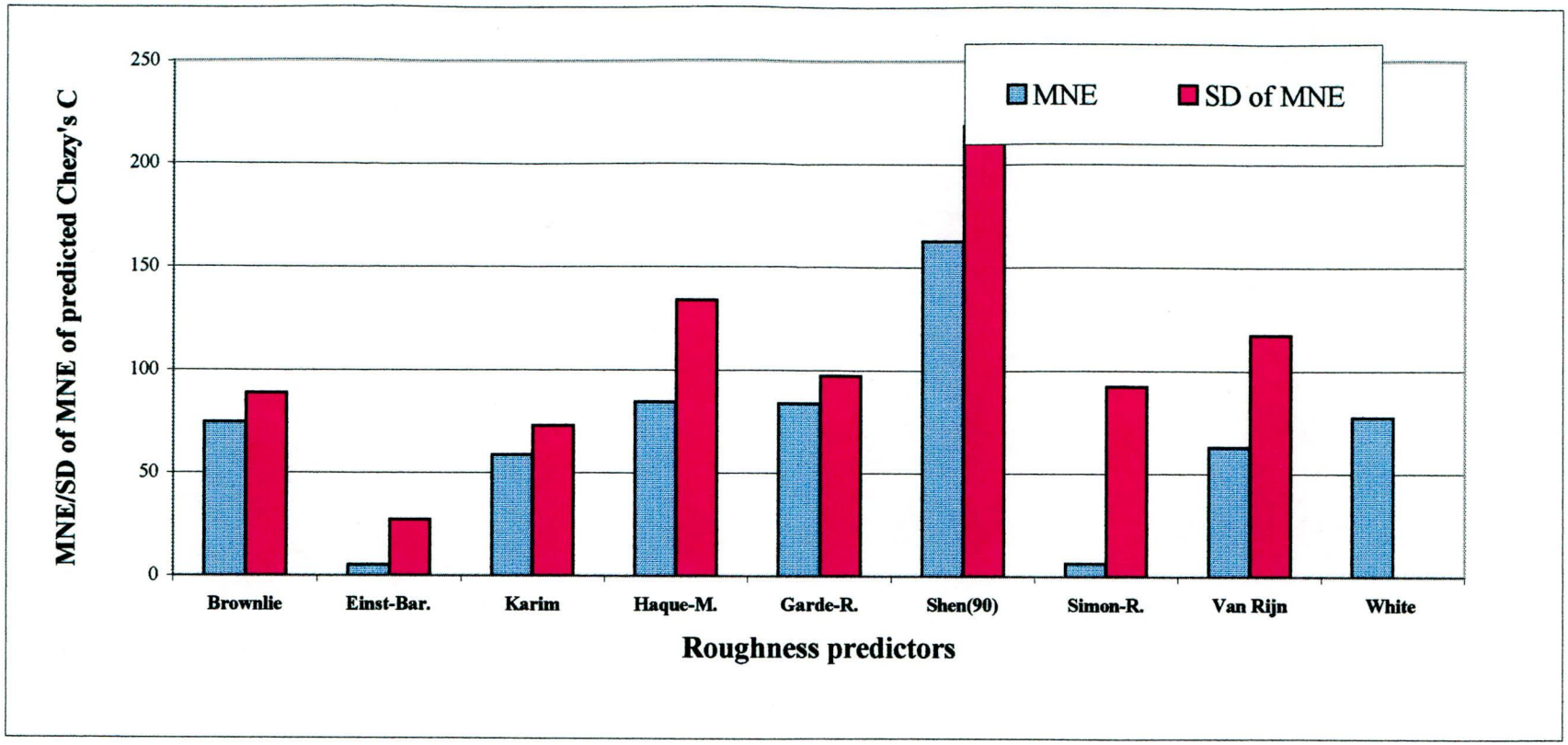

Fig.5.44	The MNE and standard deviation of MNE of the predicted Chezy's C using different roughness predictors for the Gumti river

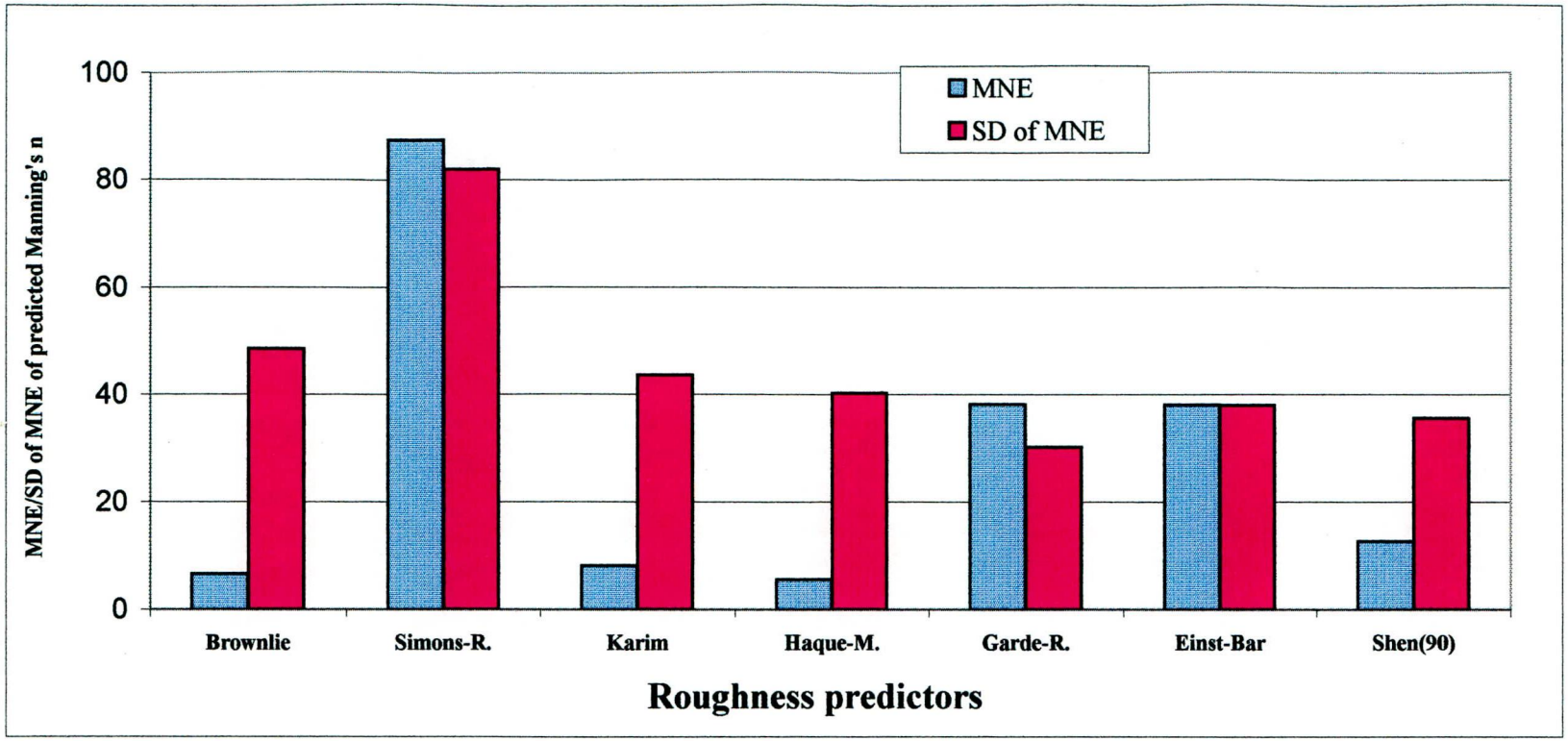

Fig.5.45	**The MNE and standard deviation of MNE of the predicted Manning's n using different roughness predictors for the Kushiyara river**

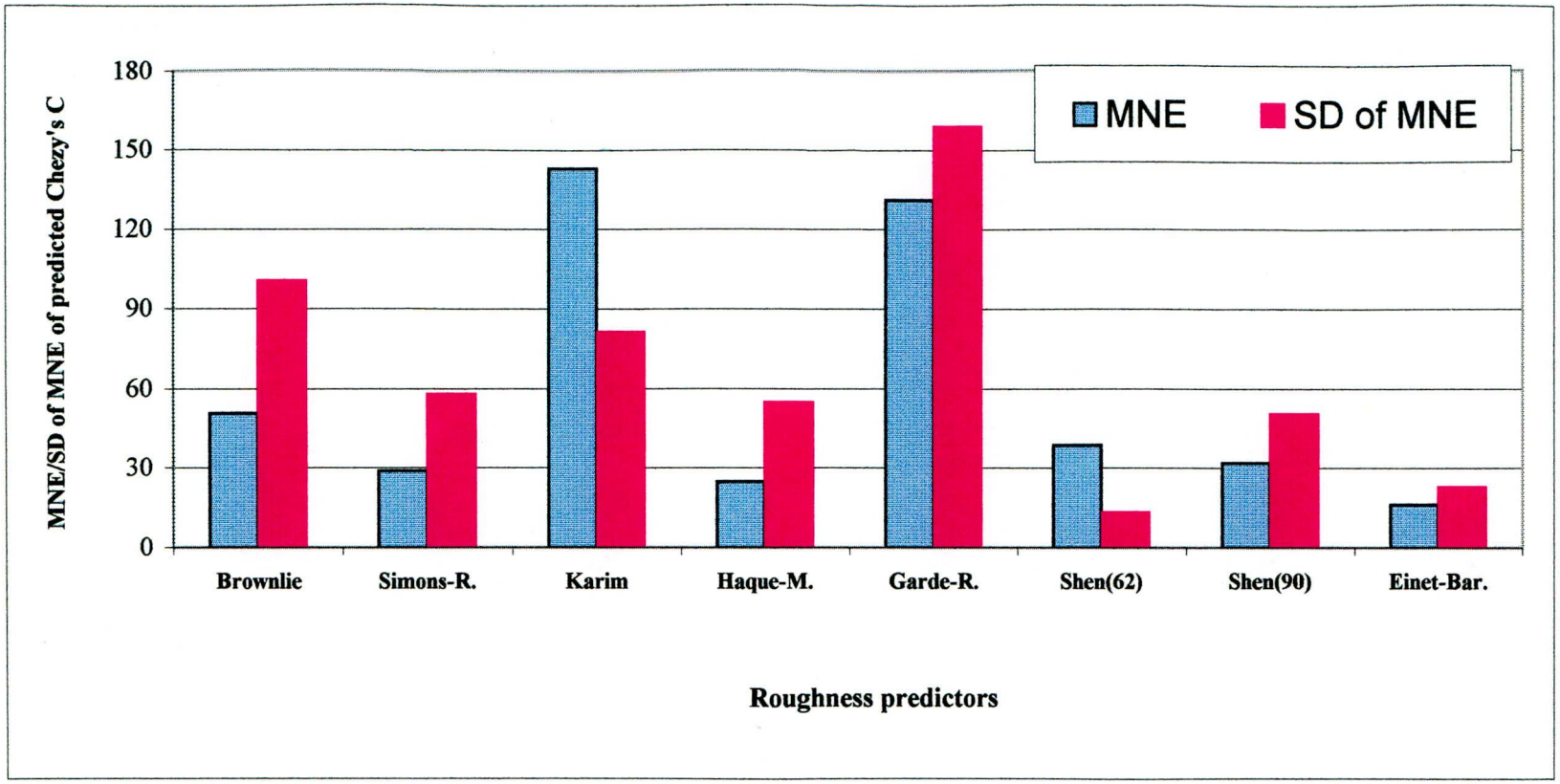

Fig.5.46	**The MNE and standard deviation of MNE of the predicted Chezy's C using different roughness predictors for the Kushiyara river**

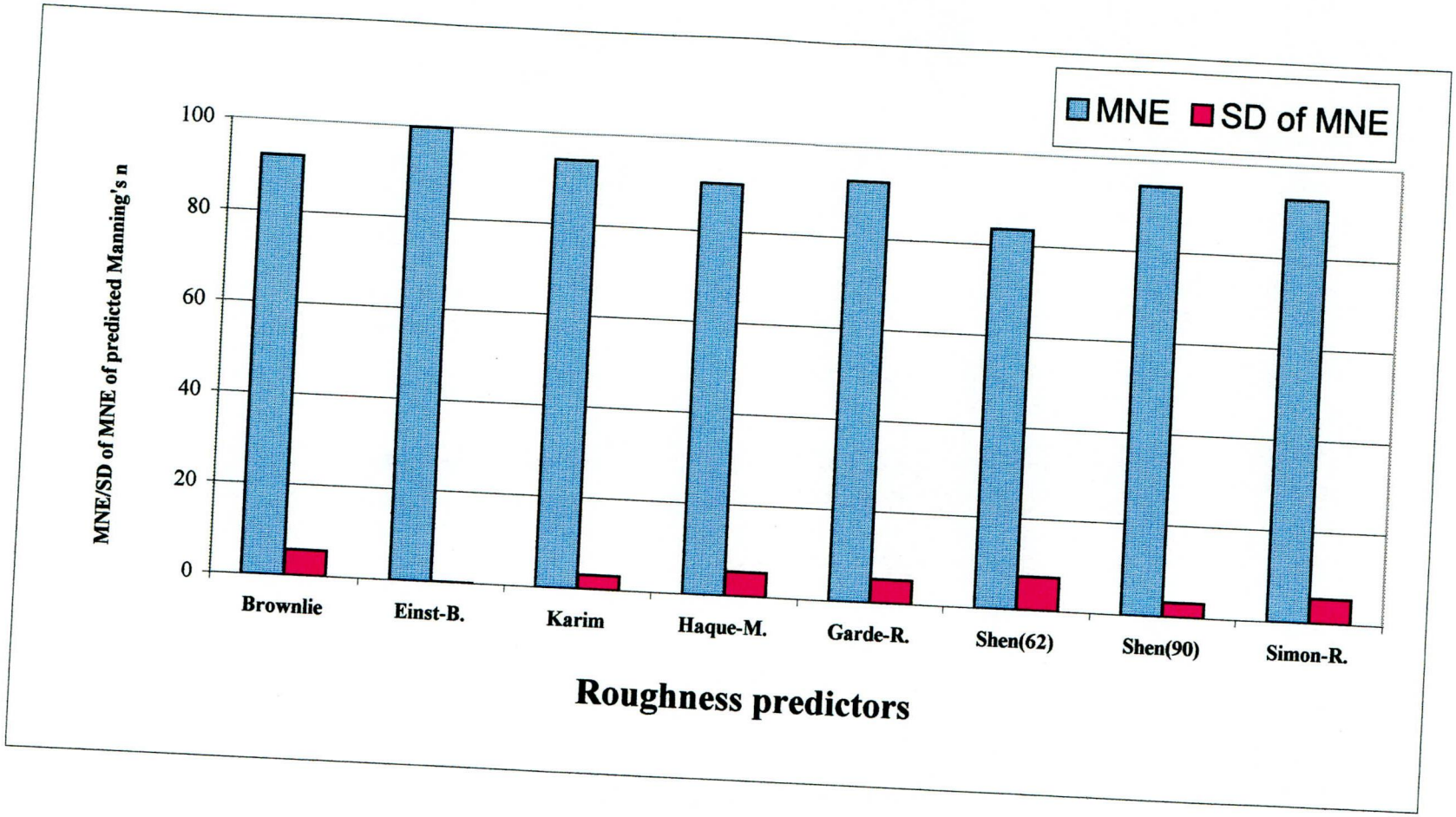

Fig.5.47 The MNE and standard deviation of MNE of the predicted Manning's n using different roughness predictors for the Mohananda river

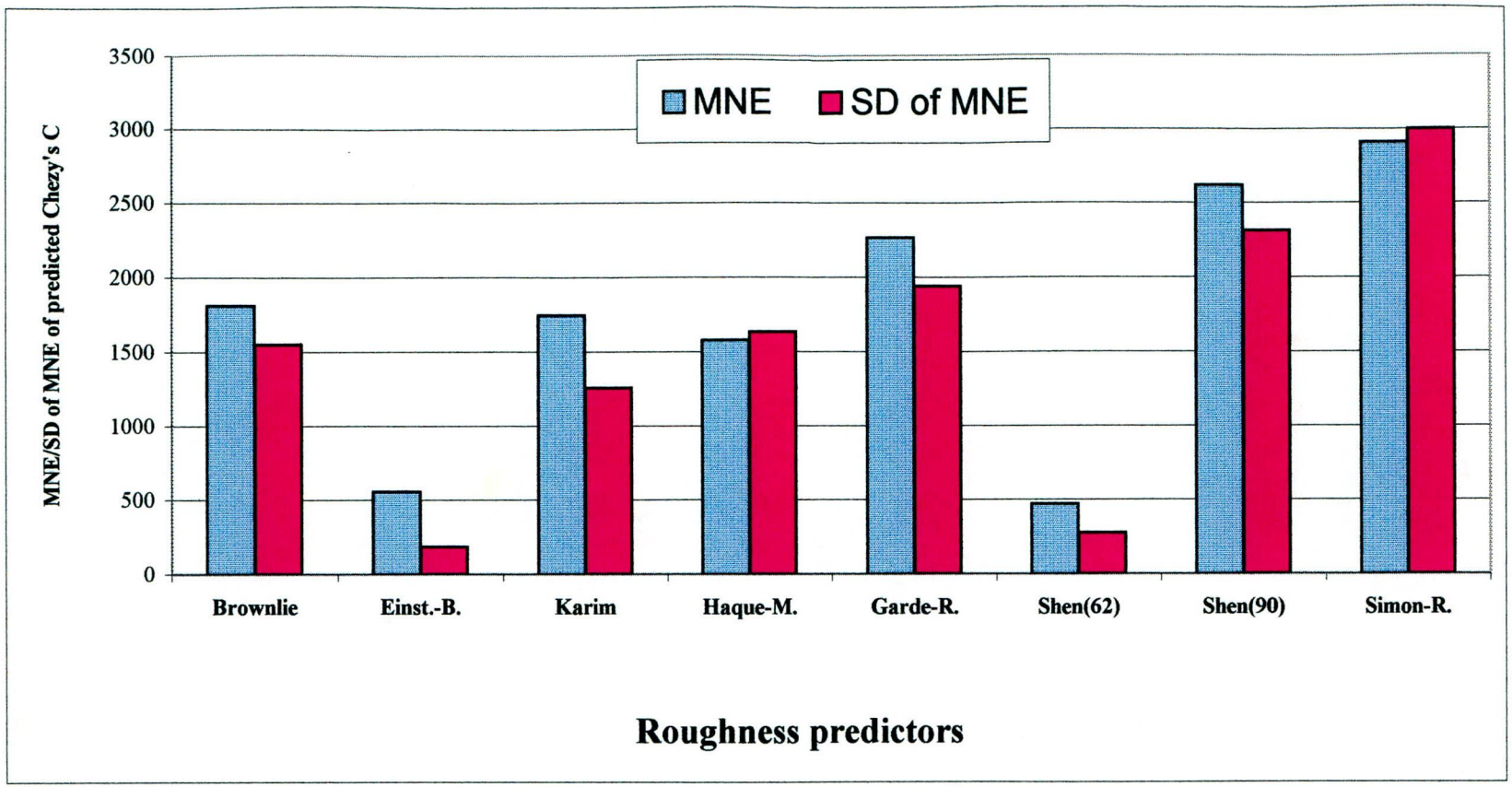

Fig.5.48	**The MNE and standard deviation of MNE of the predicted Chezy's C using different roughness predictors for the Mohananda river**

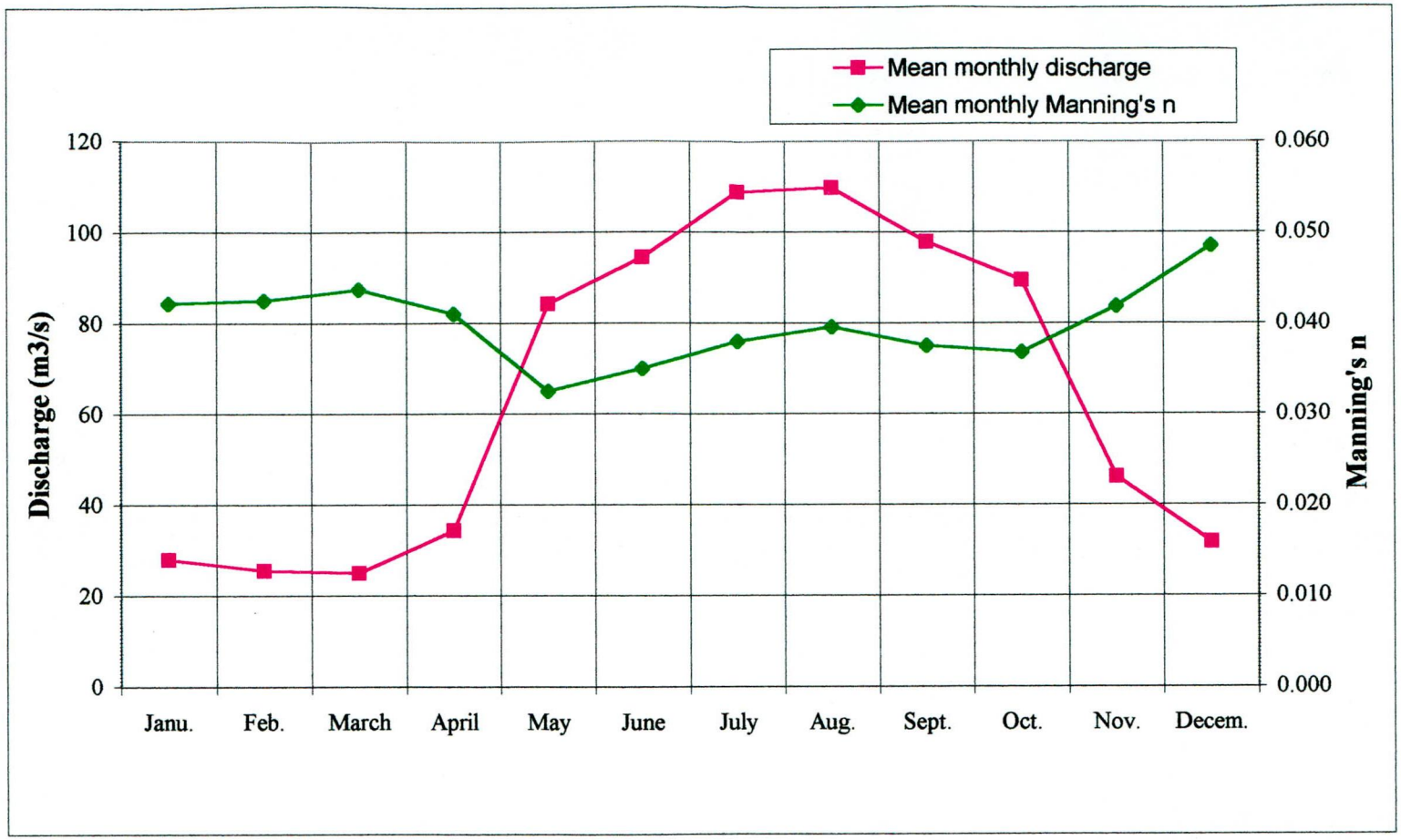

Fig.5.49 **Mean monthly discharge and Manning's n of the Gumti river at Jibanpur station**

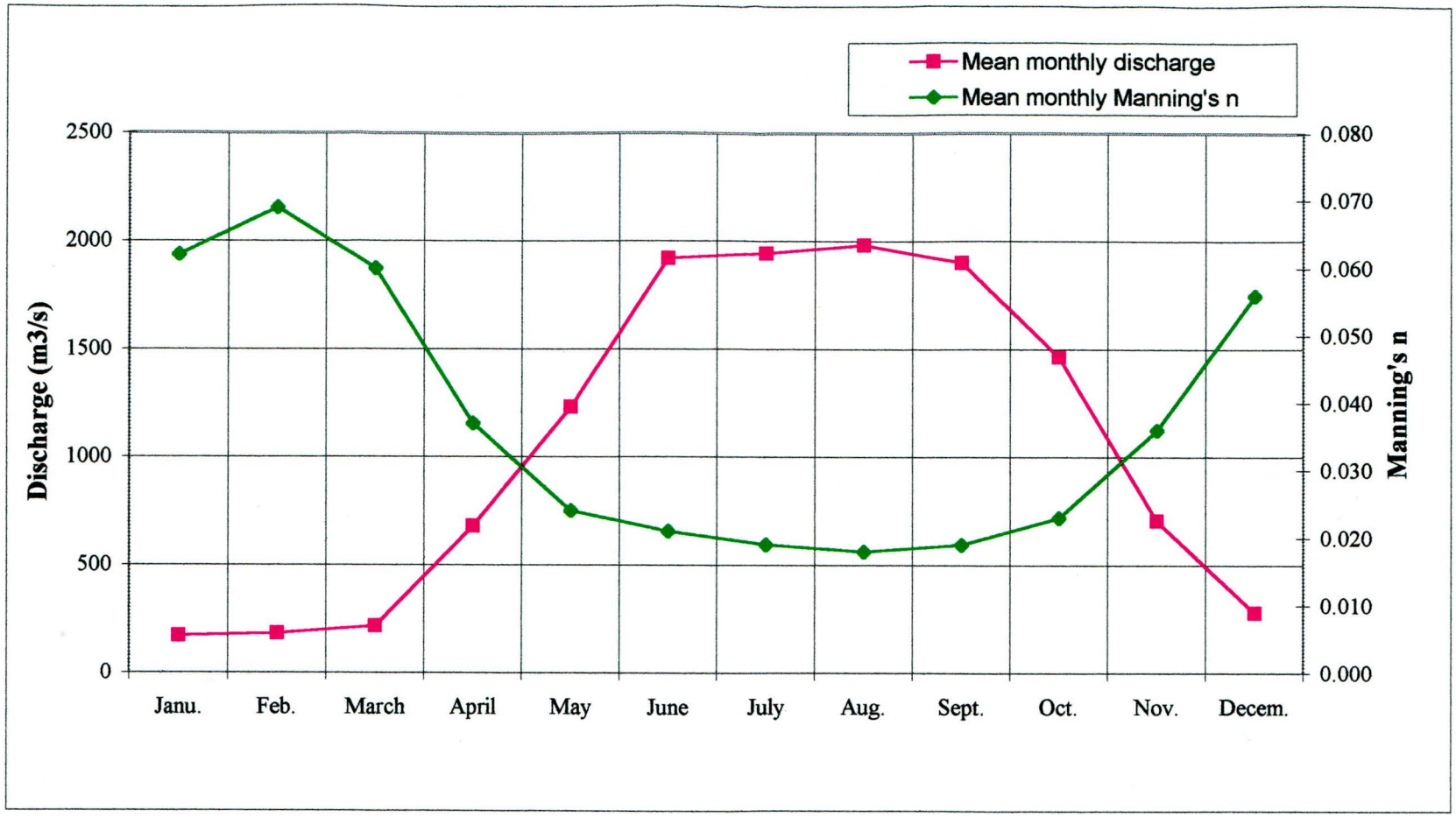

Fig.5.50 **Mean monthly discharge and Manning's n of the Kushiyara river at Sherpur station**

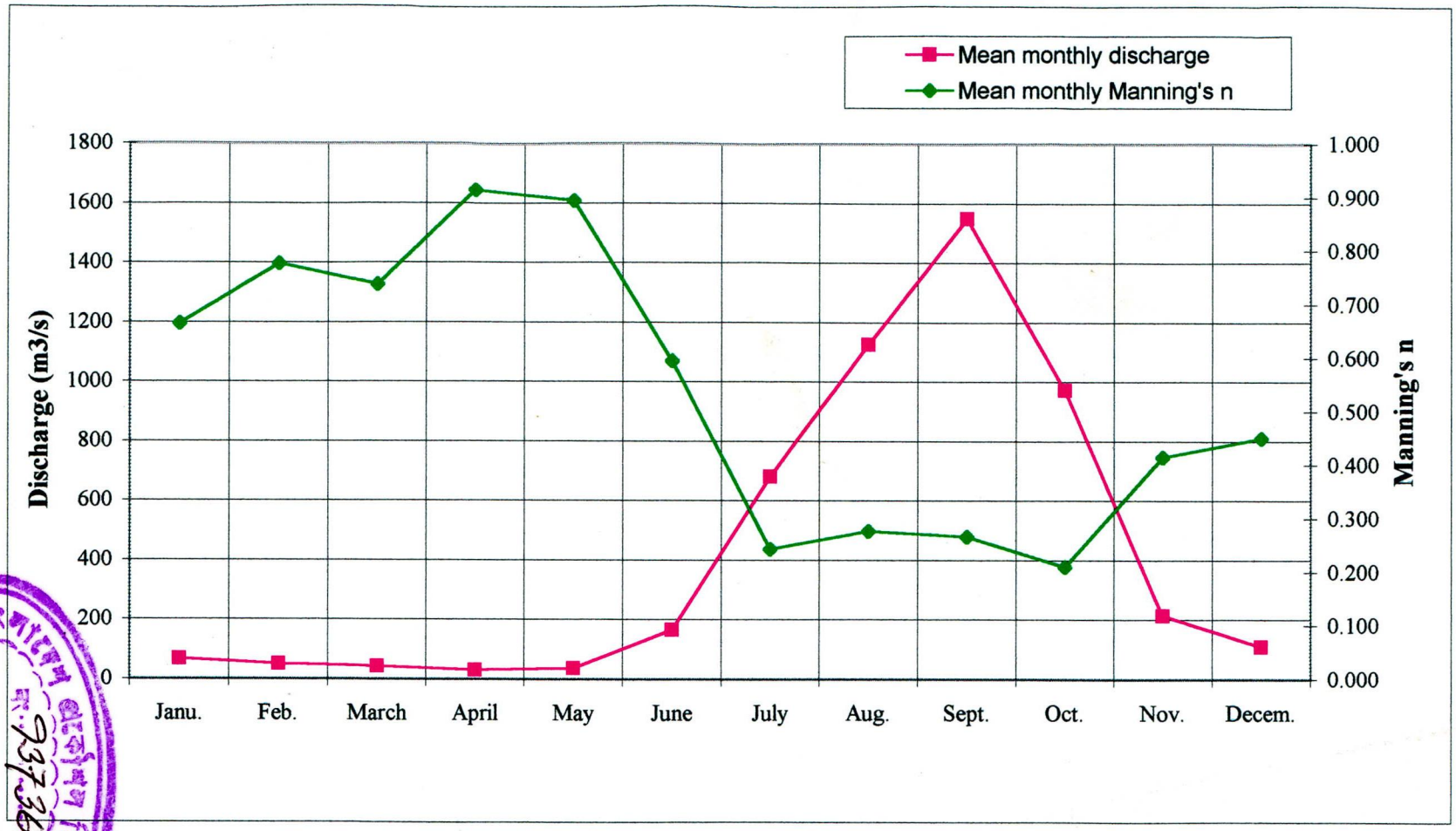

Fig.5.51 **Mean monthly discharge and Manning's n of the Mohananda river at Chapai Nawabganj station**