

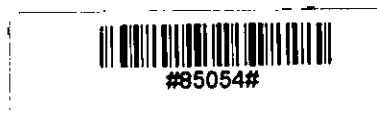
OPTIMAL METHOD FOR PREDICTION OF CHANNEL GEOMETRY IN ALLUVIUM



ZIAUL HAIDER

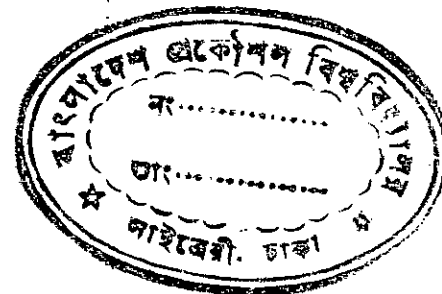
DEPARTMENT OF WATER RESOURCES ENGINEERING
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OPTIMAL METHOD FOR PREDICTION OF CHANNEL GEOMETRY IN ALLUVIUM



BY

ZIAUL HAIDER

A thesis submitted to the Department of Water Resources
Engineering of Bangladesh University of Engineering and
Technology, Dhaka in partial fulfillment of the requirements for
the degree

of

MASTER OF SCIENCE IN WATER RESOURCES ENGINEERING

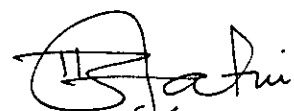
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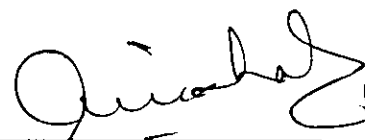
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WE HEREBY RECOMMEND THAT THE THESIS PREPARED BY ZIAUL HAIDER ENTITLED OPTIMAL METHOD FOR PREDICTION OF CHANNEL GEOMETRY IN ALLUVIUM BE ACCEPTED AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN ENGINEERING (WATER RESOURCES).

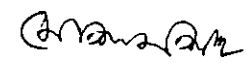
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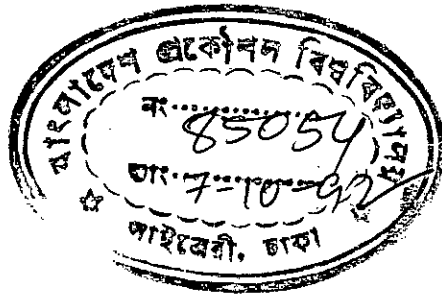
I hereby declare that the research reported in this thesis was performed by me as a research project in partial fulfillment of the requirements for the degree of Master of Science in Water Resources Engineering from the Bangladesh University of Engineering and Technology (BUET) .

This thesis contains no material which has been accepted for the award of any other degree or diploma from any other institution. Further, to the best of my knowledge and belief, the thesis contains no material previously published or written by any other person, except when due reference is made in the text of the thesis.

August, 1992

Ziaul Haider

(Ziaul Haider)



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ABSTRACT

A computational approach has been formulated for the prediction of channel geometry in alluvium. Optimal hypothesis in combination with Engelund Hansen (1967) sediment transport equation and the flow resistance equation have been utilized in this procedure. For imposed values of discharge, longitudinal slope and sediment median size, this approach predicts the stable channel section by maximizing the sediment concentration value. A computer program has been developed for the computational purpose. A total of 209 sets of alluvial and sediment characteristics data collected from different sources have been utilized in order to verify the predictive performance of the present procedure. With the same set of data the procedure has been compared with the Wallingford Optimal method (White et al., 1982) of channel design. In addition to this, Wallingford approach has been revised by incorporating the recent modification of the sediment transport equation (H.R. 1990). In the light of comparison made, it is shown that the author's suggested approach predicts the channel section more closer to the field data than the Wallingford approach. Similar comparison has been made with Lacey's approach of channel design. The result of this comparison have been shown in both tabular and graphical forms. It is expected that the procedure can be used for designing geometry of stable channels for irrigation and drainage projects. Finally application, some suggestions and recommendations for future extension of work has been outlined.

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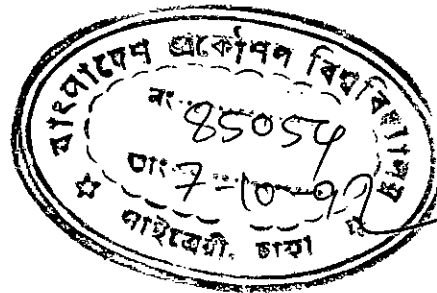
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Symbols

A	- critical mobility number of initiation of motion in Ackers White equation
C	- parameter in Ackers White equation
d	- depth of flow
d'	- depth of flow due to grain roughness
D	- representative sediment grain size
D_{gr}	- dimensionless grain size in Ackers White equation
E_r	- energy dissipation rate
f	- friction factor in Engelund Hansen flow resistance equation
F_r	- Froude number
F_{gr}	- sediment mobility number in Ackers White equation
ϕ	- dimensionless sediment discharge in Engelund Hansen equation
g	- acceleration due to gravity
G_{gr}	- dimensionless sediment transport rate
l	- length of channel reach

m	- parameter in Ackers White equation
n	- parameter in Ackers White equation
q_s	- sediment discharge in Ackers White equation
Q	- water discharge in channel width
s	- specific gravity of sediment grains
S	- longitudinal slope of channel grain
τ	- total bed shear stress
τ'	- shear stress on the dune
τ''	- shear stress caused by the form drag
τ_s	- dimensionless shear stress in Engelund Hansen equation
θ	- dimensionless total shear stress
θ'	- dimensionless shear stress due to grain roughness
θ''	- dimensionless shear stress due to bed form
U_f	- shear velocity
U_p	- unit stream power
V	- mean velocity
X	- sediment concentration in Ackers White equation
z	- side slope of channel

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CHAPTER ONE
INTRODUCTION



1.1 GENERAL

An alluvial channel has a mobile bed and bank, composed of erodible and noncohesive granular materials. Most of the natural water courses and man made canals are the alluvial channels. The geometry of an alluvial channel can be altered by flow conditions. This alteration of geometry always tends towards a configuration that is in equilibrium for the currently obtaining flow. Potentially a stream channel in alluvium changes its cross sectional dimensions and slope in response to the action of the flowing water. Thus the boundary of the channel can either remain stable or there can be erosion from the boundary. Conversely there can be deposition.

The morphology of channel is determined by the interaction of the fluid flow with the erodible materials of the channel boundary. This interaction involves the distinct processes of erosion, entrainment, transportation and deposition. Complete analytical solution of the basic mechanical principles which define these processes is still a long way off. This is because the boundary of the channel is deformable. The problem becomes more complex when the boundary material is cohesive.

The branch of hydraulic science which deals with the transport of sediment laden water is possibly the most complex and inexact. This complexity arises from the number of independent factors which can influence the geometry of a channel for a given discharge. Of these, the important factors are the density and the size of the particles and the sediment concentration.

The nature of an alluvial channel results from an integration of the upstream factors of geology, climate and land use which determine the type of sediment and the manner in which water is supplied from the source area. The river morphology is likely to remain stable when the upstream factors remain constant for the drainage area. Such stability is reflected in many the generalized relationships evolved by various researchers in the field of channel design.

1.2 STATE OF ART OF THE OPTIMAL METHOD OF CHANNEL DESIGN

Efforts in the field of stable channel design have proceeded since late in nineteenth century. Regime and theoretical methods are the two approaches of designing stable channels. Regime method is an empirical method which is based on the data of stable rivers, canals and laboratory experiments. The application of this approach depends on the quality of data and on the validity of the assumptions on which the derived relationship is based. On the

other hand theoretical approach is based on relevant governing equations. The application of this approach depends on the proper identification of dominant process and adequate definition of this process in equational form.

There are six variables governing the process of channel geometry. These are water discharge, sediment concentration, sediment size, channel width, depth and longitudinal slope. Besides theoretical and regime approach there is another approach to predict three variables of channel knowing other three. Therefore atleast three relations are required to solve the problem of channel geometry prediction. Sediment transport and flow resistance are the two relations. The nature of the third relation is conjectural. Some attempts at resolving this conjecture are concerned with bank stability (Parker, 1978; Ackers, 1980) or constancy of total sediment concentration is stable channel (Griffiths, 1984), other invoke an extremal hypothesis or variational principle. The later approach has been vigorously developed and expounded, particularly during the past 10 years (Daves and Sutherland, 1983). In this hypotheses, it is assumed that a channel achieves stability when a specified function of state of the above variables has an extremum (maximum or minimum) subject to given constraints. Recently many research has been done in this approach to predict the channel geometry. Great interest has been shown by Ackers and Charleton (1970), Charleton et al. (1978), Ackers (1980), Chang (1980). A computational technique was

also developed at Wallingford (White et al., 1982).

1.3 SCOPE AND OBJECTIVES OF THE STUDY

The present study is directed towards the development of a procedure to predict the channel geometry based on Engelund Hansen flow resistance and sediment transport equations by combining extremal hypothesis.

The applicability of Engelund Hansen flow resistance and sediment transport equations is well recognized by many investigators. Chang (1980) proved that Engelund Hansen equation produce better result than Einstein Brown and DuBoys (1876) formula for sediment discharge. He also conferred that flow resistance formula by Engelund Hansen is more accurate than the Lacey formula. Bari (1976) also studied different sediment transport equations and he concluded that Colby (1964) and Engelund Hansen (1967) equations yielded better results. Moreover in Bangladesh M.P.O., S.W.S.M.C (1990) intended to use the Engelund Hansen equations in sediment transport model. Their objective was to study the morphological change of the river system in Bangladesh. But the applicability of such equations are yet to be studied. Therefore one of the objective is the assess the applicability of the said equation through the development of a procedure for the prediction of channel geometry in alluvium.

In recent years, Hydraulic Research Station at Wallingford has developed a system for the prediction of channel geometry. The Wallingford procedure is based on Ackers and White (1973) sediment transport equations in combination with the White, Paris and Bettess (1980) flow resistance equation. There have been made a detailed comparison between the Wallingford approach and the approach developed by the author.

Hydraulic Research (1990) revised the earlier Ackers-White (1973) equation with extended set of recent alluvial channel data. In Wallingford approach, it was felt worthwhile to incorporate the revised Ackers-White sediment transport equation. The revised Wallingford approach was then compared with the earlier Wallingford approach.

In summary, the specific objectives of this study are as follows:

1. To develop a computational approach for determining channel geometry in alluvium using Engelund Hansen sediment discharge and flow resistance equations.
2. To compare the developed procedure with the Wallingford approach, and the revised Wallingford approach.

CHAPTER TWO

REVIEW OF LITERATURE

2.1 INTRODUCTION

Traditionally the process of river formation was the interest of geomorphologists and of engineers. Their approaches were descriptive explanations, rather than quantitative concepts. In recent years much more attention has been given to quantitative studies of channel geometry. The reasons of this change of study pattern are considered to be as follows.

Firstly control of river changes cannot be achieved without full knowledge of definite laws that represent the behavior. Secondly economic design of canal cannot be obtained without the thorough knowledge of channel behavior. For these reasons many investigators have been working in the field of channel design since late in the nineteenth century. In the following section there is given a brief review of the various efforts given by different investigators.

2.2 PREVIOUS INVESTIGATIONS

The study of stable alluvial channels in terms of their cross-sectional geometry, longitudinal slope and pattern in plan have been the subject of considerable research over hundred years. Alluvial channel possess three degrees of freedom in which to adjust its cross-section for a given flow condition. These

three degrees of freedom are width, depth and longitudinal slope. Some researchers have suggested the need to consider another degree of freedom. This is velocity of flow [Meekin in (1948), Hay (1978)]. When the channel reach under consideration is straight, the velocity can be related to discharge, width and depth. However natural streams have one more degree of freedom. This involves the channel shape in plan.

In recent years, various investigators have agreed that breadth, depth and slope are all independent variables in alluvial channel flow. Thus there has been the necessity for atleast three design equations to express these variable quantitatively. The problem appears even more complex when one recognizes the fact that sediment transport also affects the channel stability. For describing such effects, other variables, such as sediment concentration and its mean diameter may also require to be taken into consideration.

There are two main approaches of stable channel design. The first approach is regime method and the second approach is theoretical. A brief description of these approaches is as follows:

The regime method is empirical and relies on available data from stable rivers, from canals and from laboratory experiments. In order to correlate different variables the relevant data are examined numerically by the researchers from their different points of view. Once a relationship for a variable has been established, it can be useful in design for predicting the same variable. Kennedy (1895) was the first to propose this empirical approach. By analysing data from stable canals, he derived a relationship between the mean velocity and depth of

flow. Then it was Lindley (1919), who first recognized the need for three independent relationships with non-silting velocity being a function of both depth and width. Lacey (1930) suggested an explicit use of a measure of sedimentological constraint - the 'silt factor' to moderate a set of regime equations. Lacey's equations were based on data from canals, predominantly involving transport of non cohesive sediments. Since their introduction, these equations have been modified in different ways [Mahmood and Shen (1971)]. Inglis (1949) has generalised these equation by introducing a parameter for the bed load concentration. Again, Chien (1957) showed that the value of the use of silt factor. This factor is obtained essentially from the sediment transport equation and is strongly related to load concentration. Also, Blench (1951, 1969) recognised the requirement of allowance for bed load concentration in his sets of equations. Subsequently Simons and Albertson (1963) have adopted a simpler approach to estimate values of the design variables. They used data from American and Indian canals. In 1966 Chitale proposed new empirical equations based on the use of extended sets of Indian canal data. Later he further revised this approach [Chitale 1978], by including resistance law and in allowance for suspended load. Using computational techniques, Smith (1977) developed an alternative regime method for canal design. he considered sediment transport capability and bed material size as explicit function of the combination of width and slope. Many other attempts have been made to improve and rationalize the regime equations. Example are by Acker (1964), Gill (1968), Barr et al. (1980).

Over the past few decades, there have been attempts to generalise the regime concepts. This has involved the analysis of large number of data from field sources. Again such attempts have been made by many researchers.

Examples are Nixon (1959), Kellerhals (1967), Bray (1973), Neill (1973), Khalil (1975), Peterson (1975) and Nishat (1981).

On the other hand, theoretical approach for prediction of channel geometry is based on the determination of the relevant governing equations. These equations are therefore required to describe the individual process, such on sediment transport, flow resistance and bank stability. This approach can only be successful if the dominant process are truly identified and can be described adequately in appropriately equational form.

One of the examples of theoretical approach is the tractive force method. This method was evolved from the work of Lane (1955). He applied this method successfully to design an erodible channel carrying almost silt free water. Also some of the older design practices are based on permissible velocity. In this case, the minimum permissible velocity is intended for the non-silting situation. At same time there must be application of maximum permissible velocity. This must be done to prevent erosion [Fortier and Scoby (1926)].

These methods, regime and theoretical, are considered complementary. In addition to these approaches another approach is gradually becoming more popular in the recent past. This approach is termed as rational or analytical approach. It employs equations to describe the dominant processes; notably sediment transport, flow resistance and bank stability. Equations are available for sediment transport and flow resistance, but the nature of the third relationship is conjectural. Attempts, at resolving this conjecture, had been made by different researchers. These are a bank stability [Parker, 1978; Ackers, 1980]

constancy of total sediment concentration in a stable channel [Griffiths, 1983]; Minimum variance theory as described by Langbein (1964). The principle of maximum transport efficiency as described by Kirkby (1977); principle of lateral diffusion or turbulent momentum, as described by Parker (1978); extremal hypothesis or variational principle [White et al., (1981a, 1982)]. The last approach has been widely recognised and extensively practised during the past 28 years [Davis & Sutherland, 1983]. More recently, a similar approach has been postulated by the Hydraulic Research Limited at Wallingford.

There are five extremal hypotheses which were developed by various researchers. Minimum stream power as stated by Chang [1979, 1980]. Yang and Song (1979) gave the definition of minimum unit stream power hypothesis. Yang et al. (1981) provided the most recent hypothesis, Minimum energy dissipation rate. Davies and Sutherland [1980] introduces the hypothesis maximum friction factor. Maximum sediment transport rate was stated by White et al. (1982).

The present study involves the development of a procedure for prediction of a channel geometry by using the sediment transport and flow resistance equations in conjunction with maximisation of sediment concentration. Detail of this will be discussed in the next chapter.

CHAPTER THREE

OPTIMAL METHOD FOR CHANNEL GEOMETRY PREDICTION: A REVIEW

3.1 INTRODUCTION

An artificial channel in natural alluvium is generally straight in configuration. Thus design variables for such regular channels are limited. These variables are water discharge, sediment discharge, sediment size, channel width, depth and longitudinal slope. Among these six variables frequently the discharge, slope and sediment size are fixed by the project requirement and valley conditions. Knowing these three variables the rest three variables of stable channel can be obtained from three independent relations. Among those three relations flow resistance and sediment transport formula have been widely used. The third condition is the optimal hypothesis. In the following sections attempts have been made to describe briefly about these three conditions for the optimal methods of channel geometry prediction.

3.2 STABLE CHANNEL

A stable channel is one in which gross dimensions (slope, width, depth, bed elevation, bed material size distribution) of component reaches remain essentially constant over an extended period of time [Bondurant (1975)]. For engineering considerations an appropriate time scale is the design life of structures or control works (50-100 year). The definition does not preclude shifts in alignment

or local aggradation or degradation; but it requires a balanced average between erosion and rebuilding of channel banks and reach, input and output sediment discharges. In other words, a river may vary its channel dimensions locally in space or time without interfering with overall stability provided these variations fluctuate about constant values. According to this definition most natural channels are stable.

3.3 OPTIMAL HYPOTHESIS

Optimal hypothesis states that a channel arrives at stability function if the controlling variables has an extremum (maximum or minimum) subject to given constraints. Definition of five optimal hypothesis together with brief descriptions of their usage are given in the following articles.

3.3.1 Minimum Stream Power (MSP)

This hypothesis is stated as [Chang, 1980b]. For an alluvial channel the necessary and sufficient condition of equilibrium occurs when the stream power per unit channel length γQS , is a minimum subject to given constraints. Since Q is a given parameters, minimum γQS also means minimum channel slope S .

3.3.2 Minimum Unit Stream Power (MUSP)

Yang and Song [1979] gave the definition. For subcritical flow in an alluvial channel, the channel will adjust its velocity, slope, roughness and

geometry in such a manner that a minimum amount of unit stream power is used to transport a given sediment and water discharge.

Unit stream power U_p is defined as

$$U_p = VS \quad 3.3.1$$

Where V is mean flow velocity

3.3.3 Minimum Energy Dissipation Rate (MEDR)

Yang et al. (1981) provide this definition. A system is in an equilibrium condition when its rate of energy dissipation is at a minimum value. The energy dissipation rate E_r is given by

$$E_r = (\gamma Q + \gamma_s Q_s) l S \quad 3.3.2$$

where l is reach length.

3.3.4 Maximum Friction Factor (MFF)

Davies and Sutherland (1980) gave the definition. The flow of fluid deforms the boundary from plane boundary to a nonplanar shape in such a way that the friction factor increases. The deformation will ease when the shape of the boundary is that which gives rise to a local maximum friction factor. The friction factor f is defined as

$$f = \frac{8gdS}{V^2} \quad 3.3.3$$

3.3.5 Maximum Sediment Transport Rate (MSTR)

White et al. [1981a,1982] have suggested a new approach to the prediction of alluvial channel geometry. In this approach they used interactively a definition of the frictional resistance of channel. In addition they utilised a condition of optimization considering a form of extremal hypothesis.

White et al.(1982) states this hypothesis as, "For a particular water discharge and slope the width of the channel adjusts to maximize the sediment transport rate".

It is worth noting that this assumption is similar to approaches proposed by Yang (1978), Remette (1980) and Chang (1980), all involving the stream power concept. In this study this extremal hypothesis has been used for channel section prediction.

3.3.6 Optimal Hypothesis for Channel Section Prediction

Based on the foregoing hypothesis, Hydraulic Research Limited(HR) presented a method for prediction of channel section. This method is known in literature as Wallingford optimal method (Novak,1983). In this method Ackers White(1973) sediment transport equations and White et al. (1980) flow resistance equations were used and optimises their joint solution by either minimum gradient or the maximum sediment load for a given water discharge and sediment size.

In the present study similar approach has been developed by using Engelund Hansen(1967) flow resistance and sediment transport equations.

3.4 FLOW RESISTANCE FORMULA

The factors governing the flow in alluvial channels are numerous and complex. This is because a large part of the hydraulic resistance in the alluvium may be caused by a form drag or bed forms which have a configuration determined by the sediment transport and the flow. Many attempts have been made by the investigators to solve the problem of resistance to flow in alluvial channels. Some of these efforts have been explained in the following sections.

3.4.1 Einstein's Approach

Einstein (1950) presented two relations for solving the problem of resistance to flow with movable boundaries and bed forms. The first relation applies to the resistance due to skin friction, the second utilizes graphics to define the resistance to flow caused by bed forms.

Einstein's approach was checked with a limited amount of canal data from Pakistan. The variance did not exceed 0.018. Nordin (1964) has confirmed the value of this method. Harrison and Melema (1967) with data from the Missouri and the Mississippi rivers found a good agreement between computed and measured results using Einstein's approach. However, when this relation is applied to natural water courses with small slopes that transport fine sand the discrepancies are large (Senturk, 1965).

3.4.2 Shen's Approach

The idea proposed by Einstein to associate a defined resistance variation to plane bed and another resistance variation to the deformed or wavy bed has been adopted by many investigators. In order to define the resistance to flow caused by bed forms, Einstein's approach uses graph which has a single curve. This curve is based on data for flows with different bed configurations. Also it is clear that the parameter (ψ') which Einstein used is not sufficient to characterize and predict bed forms. As for examples the fall velocity of bed material significantly influences the bed configurations.

3.4.3 Alan and Kennedy's Approach

Alan and Kennedy (1969) developed a functional relation for the bed form friction factor. They also presented the functional relationship graphically by using different flume and field data.

3.4.4 Simmons and Richardson's Approach

The relations given in the previous sections did not consider the nature of the bed configuration. In this approach Simons and Richardson (1969) have suggested different formulas for different bed forms.

3.4.5 Engelund and Hansen Flow Resistance Equation

When the bed is covered by bed forms, the total bed shear stress, τ can be divided into two parts, τ' and τ'' . τ' is the shear stress acting on the gently curved upstream surface of the dunes and τ'' is caused by the form drag on the dunes.

Engelund and Hansen (1967) expressed the total roughness in terms of grain roughness and bed form roughness; the following relation was obtained:

$$\theta = \theta' + \theta'' \quad 3.4.1$$

where θ , θ' and θ'' are dimensionless total shear stress, shear stress due to grain roughness, and shear stress due to bed form roughness, respectively. Among them θ and θ' are defined as follows:

$$\theta = \frac{Sd}{(s-1)D} \quad 3.4.2$$

$$\theta' = \frac{Sd'}{(s-1)D} \quad 3.4.3$$

In these equations, s is specific gravity of sediment grains; S is longitudinal slope of channel; D is mean grain size of the bed material, d is depth of flow, d' is the depth of flow due to grain roughness. Using the flume data reported by Gay, Simons, and Richardson (1966), Engelund and Hansen obtained the following relationship for lower flow regime with a dune bed:

$$\theta' = 0.06 + 0.4\theta^2 \quad 3.4.4$$

For upper flow regime with standing waves and flat bed, the relationship is

$$\theta = \theta' \quad 3.4.5$$

3.4.6 White et al. Flow Resistance Equation

The flow resistance model developed by white et. al. (1980) is an extension to the Ackers White sediment transport model, which will be described in section (3.5.2).

White et al. suggested that the alluvial roughness is a function of three dimensionless variables as defined below:

(1) A dimensionless grain size, defined as the cube root of the ratio of immersed weight to viscous forces.

$$D_{gr} = D[g(s-1)/\nu^2]^{1/3} \quad 3.4.6$$

where ν is the viscosity of the water.

(2) A general sediment mobility number given by

$$F_{gr} = \frac{U_f^n}{\sqrt{gD(s-1)}} [V/\sqrt{32} \log(10d/D)]^{1-n} \quad 3.4.7$$

in which n is a function of D_{gr} ranging from 0 (coarse sediment) to 1 (fine sediment).

(3) A fine grain sediment mobility number given by $n = 1$ in equation (3.4.7)

$$F_{fg} = \frac{U_f}{\sqrt{gD(s-1)}} \quad 3.4.8$$

Where U_f is the shear velocity.

Based on a large number of experimental data White et al. found a functional relation of the form:

$$\frac{F_{gr} - A}{F_{fg} - A} = f(D_{gr}) \quad 3.4.9$$

Where A is the critical mobility number of initiation of motion. It is a function of D_{gr} . The range of applicability for equation (3.4.9) is $F_r < 0.8$ and $1 < D_{gr} < 60$

The expression for right hand side of the equation (3.4.9) is as follows

$$f(D_{gr}) = 1 - 0.76[1 - 1/\exp|(\log D_{gr})^{1.7}|] \quad 3.4.10$$

The solution procedure is straight forward. The general sediment mobility number can be expressed in terms of fine grain mobility, viz.

$$F_{gr} = (F_{fg})^nC \quad 3.4.11$$

Where C is a known parameter. Inserting equation (3.4.11) into equation

(3.4.9) yields

$$F_{fg} = \frac{C}{f(D_{gr})} F_{fg} \quad 3.4.12$$

This formula can be used iteratively to calculate F_{fg} , because both $c/f(D_{gr})$ and n are larger than 0 but smaller than 1.

3.5 SEDIMENT TRANSPORT FORMULA

It is a common knowledge that water flowing in a natural or artificial channel often has the ability to scour sand, gravel or even large boulders from bed on banks and sweep them downstream. This phenomenon is usually known as sediment transport. Knowledge of the rate of total sediment transport for given flow, fluid and sediment characteristics is necessary in the study of many aspects of alluvial streams. Alluvial channel must be designed to carry definite water and sediment discharge. The process of suspension is an advanced stage of traction along the bed, therefore the total sediment transport rate should be related to the shear parameter. There are several methods using a single representative size of bed material for the determination of load transport. In the following articles brief description of each of the methods have been given.

3.5.1 Laursen's Method

Laursen (1956) used primarily flume data to determine sediment concentration graphically. The application of this relation to rivers by different investigators has shown large errors and the method is thus of doubtful accuracy over a wide range of data.

3.5.2 Yang's Equation

Yang (1973) contended that the rate of sediment transport in an alluvial channel is primarily governed by the rate of expenditure of potential energy per unit weight of water, i.e. the unit stream power. By analysis of a vast amount of data he obtained a relation for the load concentration in ppm by weight. His equation was found to work satisfactorily both for laboratory and field data.

3.5.3 Shen and Hung's Equation

By performing regression analysis of laboratory data, Shen and Hung (1972) obtained a dimensional expression for the total load concentration in ppm by weight. Yang and Molinas (1982) found that this equation does not work very well for large rivers.

3.5.4 Karim and Kennedy's Equation

Karim and Kennedy (1983) carried out a regression analysis of the sediment data from laboratory flumes and natural streams and gave highly involved equation for the average sediment concentration. Karim Kennedy found that their

equation predicts better for a large amount of flume and field data. However, Le Cries (1983) found the accuracy of this method to be less than those of Engelund-Hansen (1967) and Ackers-White (1987).

3.5.5 Engelund Hansen Sediment Transport Equation

According to Engelund and Hansen(1967) sediment transport equation is as following:

$$f\Phi = 0.1\tau_*^{3/2} \quad 3.5.1$$

where,

f is friction factor, τ_* is dimensionless shear stress, and Φ is dimensionless sediment discharge. The expression for f , τ_* and Φ is as follows:

$$f = 2\left(\frac{U_f}{V}\right)^2 \quad 3.5.2$$

$$\tau_* = \frac{\tau}{\rho(s-1)gD} \quad 3.5.3$$

$$\Phi = \frac{q_s}{\sqrt{(s-1)gD^3}} \quad 3.5.4$$

Where q_s is sediment discharge in pound per second per unit width of channel. The total amount of sediment discharge in a channel may be obtained by integration.

3.5.6 Ackers White Sediment Transport Equation

In 1973 Ackers and White published a theory to predict the transport of non-cohesive sediments. They expressed sediment transport rate G_{gr} defined by

$$G_{gr} = \frac{Xd}{sD} \left(\frac{U_f}{V} \right)^n \quad 3.5.5$$

where, x is sediment transport, mass flux per unit mass flow rate n is model constant.

This dimensionless sediment transport rate was related to the sediment mobility F_{gr} . It was assumed that there is a critical value of F_{gr} , denoted by A , below which no sediment movement takes place.

For F_{gr} greater than A it is assumed that

$$G_{gr} = C \left(\frac{F_{gr}}{A} - 1 \right)^m \quad 3.5.6$$

Where C , A and m are parameter

The expressions for n , m , C and A are given as following for transitional size, $D_{gr} < 60$

$$n = 1 - 0.56 \log D_{gr} \quad 3.5.7$$

$$m = \frac{9.66}{D_{gr}} + 1.34 \quad 3.5.8$$

$$\log C = 2.86 \log D_{gr} - (\log D_{gr})^2 - 3.53 \quad 3.5.9$$

$$A = \frac{0.23}{\sqrt{D_{gr}}} + 0.14 \quad 3.5.10$$

for coarse sediments, $D_{gr} > 60$

$$n = 0.0 \quad 3.5.11$$

$$m = 1.50 \quad 3.5.12$$

$$C = 0.025 \quad 3.5.13$$

$$A = 0.17 \quad 3.5.14$$

3.5.7 Revised Ackers White Sediment Transport Equation [H.R. 1990]

In 1990, Hydraulic Research Limited (H.R.) revised the earlier Ackers White (1973) sediment transport equations. Four empirical parameters of this equations were modified. In the original development, little data had been available when deriving the empirical parameters. Since the development of the theory much more sediment transport data has become available, the derivation of the parameters n , A , m , C was repeated on this extended set of data. The revised equations for the parameters, m and C are derived as follows. For transitional size :

$$m = \frac{6.83}{D_{gr}} + 0.14 \quad 3.5.15$$

$$\log C = 2.79 \log D_{gr} - 0.98 (\log D_{gr})^2 - 3.46 \quad 3.5.16$$

for coarse sediments, $D_{gr} > 60$.

$$m = 1.78 \quad 3.5.17$$

However, significant improvement in the relationship for A and n was not made in this recent revision.

3.5.8 Comparison of Various Approaches of Prediction of Bed Material Load

There are numerous sediment transport formulas developed by different investigators for the prediction of bed material load in alluvial channels. Comparisons of the accuracy of these formulas were made by White et al. (1975), Yang (1976), the ASCE Task Committee ("Relationships between Morphology", 1982), Yang (1988), Brownlie (1981), among others. These comparisons emphasize the overall accuracy of formulas without given detailed information of the hydraulic and sediment conditions under which measurements were made.

Recently Yang and Wan (1991) made a comparative study of the overall accuracy as well as the accuracy within different ranges of sediment concentration, Froude number, slope for seven bed material load formulas. From that study it is found that Engelund Hansen equation is better than Ackers White equation.

CHAPTER FOUR

COMPUTATIONAL METHODOLOGY

4.1 INTRODUCTION

As stated in the earlier chapter (chapter one), the present study is directed towards the development of an approach for prediction of alluvial channel geometry based on the optimal hypothesis together with flow resistance and sediment transport theory of Engelund Hansen(1967). The optimal hypothesis adopted here is the maximisation of sediment concentration. In order to solve such complex equations a computational technique is needed. The present chapter deals with these developments of computational technique of solution of various adopted equations.

4.2 INPUT AND OUTPUT VARIABLES

In all the three approaches, that will be used in this study, the input parameters are: water discharge, Q (m^3/s); longitudinal slope, S (m/m); sediment size, D (mm). The output parameters are average width, B (m); depth, D (m) and sediment concentration, X (ppm).

The side slope, z , of the trapezoidal section was imposed by Smith's [Smith (1974)] relation. This is :

$$z = 0.5 \quad \text{for } Q < 1 \text{ m}^3/\text{s} \quad 4.1.1$$

$$= 0.5 Q^{0.25} \quad \text{for } Q > 1 \text{ m}^3/\text{s}. \quad 4.1.2$$

The computational steps of each of the approaches are given in the following sections.

4.3 COMPUTATIONAL METHODOLOGY OF THE AUTHOR'S APPROACH

The computational steps for predicting channel geometry developed by the author is given below:

1. For an assumed width and depth, the cross-sectional area and hydraulic radius are computed by assuming the section as rectangular.
2. Now dimensionless total shear stress, θ and dimensionless shear stress due to bed form, θ' are calculated using Engelund Hansen resistance equation [equation (3.4.2) and (3.4.4) respectively]. With the help of θ' , d' is calculated from equation (3.4.3). Knowing these parameters mean flow velocity is determined.
3. At this stage the condition of continuity has to be checked. This is done by calculating the discharge and then comparing it with given discharge. If those two discharges are not equal

then steps (1) and (2) are repeated for new depth but the width remaining unchanged. This iteration loop is continued until calculated and given discharges are almost equal.

4. When the continuity condition is satisfied the next step is to check the condition of maximum sediment concentration. Sediment concentration is calculated from the following steps: For each set of channel dimension; friction factor, f and dimensionless shear stress, τ_* are computed from the definition of Engelund Hansen (1967). This definition is given in equation (3.5.2) and (3.5.3) respectively. After obtaining f and τ_* ; dimensionless sediment discharge, ϕ is calculated from equation (3.5.1). Now sediment concentration is calculated from Engelund Hansen sediment transport formula [equation (3.5.4)]. For different values of average width the entire calculation [step 1 to 3] is repeated. Finally that channel dimension is chosen which yields maximum sediment concentration. A typical example of the nature of this maximization of sediment concentration is shown in figure 4.3. This figure shows that, with the given value discharge $3049.99 \text{ m}^3/\text{sec}$, sediment median size 0.32 mm and slope 0.15×10^{-3} , the maximum sediment concentration is 788.79 ppm when optimum width is 47.50 m .
5. When the criteria for maximum sediment has been achieved with desired accuracy, then the final values of channel breadth and depth are converted to those of trapezoidal channel. This has

been done by utilizing the following relations:

$$W' = B + zH \quad 4.2.5$$

$$H = \frac{BH}{W'} \quad 4.2.6$$

$$W = W' + zH \quad 4.2.7$$

The major steps for this computational procedure is shown in the flow chart figure (4.1).

4.3 COMPUTATIONAL METHODOLOGY OF WALLINGFORD APPROACH

The computational steps for obtaining channel geometry is given below:

1. The initial parameters D_{gr} , $f(D_{gr})$, C , A , m , n in the sediment transport and flow resistance equations are calculated from the definition shown in Ackers White (1973) sediment transport and White et al. (1980) flow resistance equations [equations (3.4.6), (3.4.10), and (3.5.7) to (3.5.14) respectively].
2. Following assumptions have been made for the input of average breadth (W) and mean depth (H):

$$W = KQ^{1/2} \quad 4.2.1$$

$$H=0.6Q^{1/3}$$

4.2.2

3. Now for the assessment of width and depth, cross sectional area and hydraulic radius are determined. With the help of these initial values, shear velocity (U_f) is determined from the following expression

$$U_f = \sqrt{gRS}$$

4.2.3

4. Using these known quantity fine grain sediment mobility number, F_{fg} ; sediment mobility number, F_{gr} ; dimensionless sediment transport rate, G_{gr} and sediment concentration, X are calculated from equations (3.4.12), (3.4.11), (3.5.6) and (3.5.5) respectively.
5. At this stage the condition for continuity of flow has to be checked for initial values of channel dimensions. This is done by comparing calculated discharge with given discharge. If these discharges are not equal then calculation steps 3 and 4 should be repeated with the now depth the new depth can be revised according to the following proportion:

$$H_{new} = H_{old} * \frac{Q}{Q_{cal}}$$

4.2.4

This iteration loop should be continued until the discharge given and calculated are almost equal.

6. When the continuity condition is satisfied, the next step is to check for the condition of maximum sediment concentration. For different values of average width the entire calculation (step 3 to 5) is repeated. The channel dimension will be ascertained by maximizing sediment concentration.
7. When the criteria of maximization of sediment concentration is achieved then the final values of channel breadth and depth are converted to those of trapezoidal section. The major steps for the computational procedure is shown in the flow chart as shown in figure 4.2.

4.4 COMPUTATIONAL PROCEDURE FOR REVISED WALLINGFORD APPROACH

As stated in section 3.5.3, H.R. (1990) reported the rederivation of the empirical parameters on an extended data set. In this study it is therefore considered worthwhile to incorporate this recent modification of Ackers White (1973) in the Wallingford approach. Thus it appears that the computational steps of this revised Wallingford approach for obtaining channel geometry is same as original Wallingford approach except following:

The parameters C and m are obtained from the revised equations (3.5.15) to (3.5.17).

CHAPTER FIVE

ANALYSIS AND DISCUSSION ON RESULTS

5.1 INTRODUCTION

In this study three approaches for the prediction of channel geometry have been discussed. These are: (i) Wallingford approach, (ii) Revised Wallingford approach, and (iii) Suggested approach. In the following sections comparative assessments regarding the efficiency of prediction achieved from these approaches have been discussed. To examine the author's approach a total of 209 sets of alluvial channel data have been utilised. Two criteria for the assessment of the degree of compliance are adopted in this study. The first criterion is the determination of statistical properties i.e. mean, standard deviation and the maximum and minimum values of discrepancy ratio. The discrepancy ratio is the ratio of predicted variable to that of observed variable. Another criterion is the estimation of percentage of data coverage between certain discrepancy ratio ranges.

5.2 FEATURES OF THE DATA

There are many relationships which seek to describe various aspects of the alluvial channel phenomenon. The majority of these are adjusted using laboratory and field data. Thus alluvial channel data collection was undertaken by many investigators.

5.2.1 Sources of Data

The original data used by the various investigators is from published sources. But these sources are scattered through research papers, thesis and report. Many people have gathered and compiled data. However, there is one sequence which occupies a dominating position because of its coverage and eventual availability in disk format. This sequence of compilation was initiated by Cooper and Peterson (1970). Peterson and Howells (1978) revised this compilation. In 1981, Brownlie again revised this data compendium. He corrected four types of apparant errors. These were mainly as follows:

- i. incorrect individual entries
- I. conversion errors
- iii. misinterpretation of data and
- iv. source errors.

Matin (1988) organized a data-bank for which he used the

following sources. Predominantly he used Strathclyde data and Brownlie data [Brownlie, 1981a]. In addition, other data from individual published sources are also included in the data bank.

Following the convention established by Peterson and Howells, 10 variables are recorded for each channel observation. The following 10 variables exist in each observation:

1. Discharge
2. Width
3. Depth
4. Slope
5. Medium particle size of bed material
6. Gradation
7. Specific Gravity
8. Concentration
9. Temperature
10. Bed Form.

5.2.2 Features of the Data

Data pertinent to alluvial channels can be classified into three broad categories:

- i. Laboratory data
- ii. Canal data
- iii. Natural channel or river data

Laboratory data: Most of the data originates from laboratory studies with fixed channel breadth. Also, there have been obtained some trapezoidal flume data under mobile bed conditions. Flow in laboratory flumes is less complex than in nature. The control in experimentation and scale of experiments allows more detailed observations related to sediment transportation. But in field such arrangement is extremely difficult. Obviously, laboratory data are useful in studies related to resistance, bedform and sediment transportation. But there had been comparatively little usage in the development of relationships describing channel shape and geometry.

Canal data: Canal data are obtained under quasinatural or field conditions. Reliable observations are available for discharge, slope, depth, breadth and bed material size. But in the past, the sediment load has seldom been correctly measured or recorded. However canal data are important for shape and regime studies.

Natural channels or rivers data: Flow phenomenon in natural alluvial channels is extremely complex. Discharge and sediment load are inevitably unsteady in the longer term. Thus the channel bed is always subjected to change in sequence of scour and deposition. Despite such complexities, reliable observations of river data are valuable.

5.2.2 Features of the Data Used in this Study

In order to examine the applicability of the author's approach it is worthwhile to collect data from wide range of flow condition. Data used in this study has been taken from the data bank organised by Matin (1988). The major sources of this data bank are the Strathclyde data and Brownlie data (Brownlie, 1981a). In this study a total of 209 sets of alluvial channel data has been extracted covering wide range of flow condition with all necessary sediment characteristic information. Of these 209 sets of data, 95 sets are canal data and rest of the data are from river. Table 5.1 summerises the source of data used for the author's investigation.

5.3 THE ASSESSMENT OF CHANNEL BREADTH PREDICTION

5.3.1 Suggested Approach

For lower discharge i.e., discharge less than $300 \text{ m}^3/\text{s}$, the channel breadth calculated from this approach is found to be slightly overpredicted than the observed value. The mean discrepancy ratio for this range of discharge, with 95 data is found to be 1.06 with a range of maximum 2.46 and minimum 0.31. Table 5.2 summarizes the statistical properties of the distribution of discrepancy ratio. For lower discharge, this approach is capable of predicting breadth for 41.14% of data with a discrepancy ratio

between 0.833 and 1.20, and 63.92% of data when the range is widened to 0.714 to 1.40. Table 5.3 represents this statistics.

For higher discharge i.e., discharge greater than 300 m³/s the mean discrepancy ratio (for 114 data) is 1.148 with 3.36 and 0.30 upper and lower boundary respectively. The standard deviation is 0.519. The percentage of data coverage for the two discrepancy ratio ranges (0.833 to 1.20 and 0.714 to 1.4) has been summarized in table 5.2. 43.14% and 56.86% of data are covered in higher discharge for 0.833 to 1.20 and 0.714 to 1.40 discrepancy ratio ranges.

The channel breadth calculated from this approach is found to be uniformly distributed around the line of perfect agreement. This can be observed in figure 5.1.1.

5.3.2 Wallingford Approach

For lower discharge, this approach shows a tendency to under predict the channel breadth. The mean discrepancy ratio is 0.693 (Table 5.2) with a range of 1.51 to 0.35. Standard deviation is 0.221. On the other hand at higher discharge this approach shows a tendency to over-predict channel breadth. The mean discrepancy ratio is 1.121 with a range of 3.79 to 0.37. Standard deviation is 0.645.

At lower discharge 21.52% and 43.04% data are covered for the discrepancy ratio range 0.833 to 1.20 and 0.714 to 1.40 respectively. At higher discharge slight improvement in percentage of data coverage has been observed i.e 21.57% and 45.10% of data are covered for the two discrepancy ratio bands.

In fig. 5.2.1 it is observed that at lower discharge many points are below the line of perfect agreement. On the other hand at higher discharge range the channel breadth calculated from this approach is found to be uniformly distributed around the line of perfect agreement.

5.3.3 Revised Wallingford Approach

This approach under-predict the channel breadth for lower discharge. The mean discrepancy ratio is 0.691 (Table 5.2) with a range of 1.33 to 0.36. Standard deviation is 0.207. At higher discharge this approach over predict width, but this over prediction is less than Wallingford approach. The mean discrepancy ratio is 1.085 with 2.69 and 0.33 maximum and minimum values respectively. Standard deviation is 0.513.

At lower discharge 21.52% and 44.94% of data fall in discrepancy ratio range 0.833 to 1.20 and 0.714 to 1.40 respectively (Table 5.3). At higher discharge, 23.53% and 52/94% data fall in 0.833 to 1.20% and 0.714 to 1.40 bands of discrepancy ratio .

From fig. 5.3.1 it is observed that at lower discharge greater number of points are below the line of perfect agreement. On the other hand at higher discharge range the channel breadth calculated from this approach is found to be uniformly distributed around the line of perfect agreement.

5.3.4 Comments on Results

At lower discharge, percentage of data coverage in suggested approach is higher than the other approaches. At higher discharge percentage of data coverage in suggested approach is high in 0.833 to 1.20 discrepancy ratio range. But in 0.714 to 1.40 discrepancy ratio range there is no significant improvement in percentage of data coverage in the suggested approach as compared to the other approaches. Summarizing, from the foregoing discussions on results, it can be concluded that for all ranges of discharge the suggested approach predicts width very closer to observed width. But other three approaches underestimate the breadth.

Comparing the data of revised Wallingford approach with that of Wallingford approach, it can be concluded that, at lower discharge there is no significant improvement in of percentage of data coverage in the selected bands of discrepancy ratio. But at higher discharge revised Wallingford approach yields better results.

5.4 THE ASSESSMENT OF CHANNEL DEPTH PREDICTION

5.4.1 Suggested Approach

The mean discrepancy ratio for lower discharge is 1.061 with a range of 2.14 to 0.49 and standard deviation 0.335. For higher discharge the mean discrepancy ratio is 0.904 with a range of 0.48 and standard deviation is 0.368.

For lower discharge the percentage of data coverage is 49.37% and 77.22% for 0.833 to 1.20 and 0.714 to 1.40 range of discrepancy ratio. For higher discharge these percentage is 66.67% and 41.18% as shown in table 5.5.

From figure 5.1.2 it has been observed that the channel depth calculated from this approach is found to be uniformly distributed around the line of perfect agreement.

5.4.2 Wallingford Approach

The mean discrepancy ratio at lower discharge is 2.483 with a range of 5.76 to 1.22 and standard deviation 0.873. At higher discharge this mean is reduced to 2.039 as shown in Table 5.4.

At low discharge very few data fall in selected bands of discrepancy ratio 0.0% and 0.63% data fall in 0.833 to 1.20 and 0.714 to 1.40 discrepancy ratio ranges respectively. At high discharge 11.76% and 13.73% of data fall in 0.833 to 1.20 and 0.714 to 1.40 range of discrepancy ratio.

From figure 5.2.2 it is observed that almost all the points are above the line of perfect agreement. This means that this approach over-estimated channel depth.

5.4.3 Revised Wallingford Approach

The mean discrepancy ratio at lower discharge is 2.50 with a range of 5.76 to 1.4 and standard deviation 0.889. At higher discharge the mean is reduced to 2.039, as shown in table 5.4.

The percentage of data coverage in selected bands of discrepancy ratio is same of the previous approach i.e. Wallingford approach.

In this approach all data fall above the line of perfect agreement as shown in figure 5.3.2. It mean that this approach over predict the channel depth.

4.4.4 Comments on Results

The performance of depth prediction in suggested approach is significantly closer to observed value as compared to other approaches. Comparing the results of Wallingford approach with that of revised Wallingford approach, it can be concluded that there is no improvement in percentage of data coverage in selected bands of discrepancy ratio. Both this approach over predict channel depth.

5.5 ASSESSMENT OF SEDIMENT CONCENTRATION

5.5.1 Suggested Approach

For lower discharge value the mean discrepancy ratio for sediment concentration is 1.44 with a range of 0.10 to 6.95 and 1.205 standard deviation. For higher discharge the mean discrepancy ratio rises to 1.669. This is shown in table 5.6.

For lower discharge, 86.07% and 57.59% of data are covered in 0.25 to 4.00 and 0.50 to 2.00 bands of discrepancy ratio. In higher discharge the percentage of data coverage is reduced to 64.71% and 43.14% .

From fig.5.1.3 it is observed that sediment concentration calculated from this approach is found to be uniformly distributed around the line of perfect agreement.

5.5.2 Wallingford Approach

For lower discharge the mean discrepancy ratio for sediment concentration is 0.347 with a range of 0.07 to 2.80 and standard deviation 0.457. At higher discharge the mean discrepancy ratio is reduced to 0.295 with a range of 0.07 to 3.84.

For lower value of discharge, the percentage of data coverage in 0.25 to 4.00 and 0.50 to 2.00 discrepancy ratio ranges are 36.08% and 14.56% respectively. For higher value of discharge these percentage is reduced to 25.49% and 1.96% respectively.

In fig. 5.2.3 it is observed that the sediment concentration calculated from this approach is not uniformly distributed around the line of perfect agreement.

5.5.3 Revised Wallingford Approach

For lower discharge the mean discrepancy ratio is 0.335, but for higher discharge it is slightly improved. The mean is 0.461. This is shown in table 5.6.

The percentage of data fall in lower discharge range is 32.28% and 12.02% in 0.25 to 4.00 and 0.50 to 2.00 discrepancy ratio ranges. For higher discharge range these percentage fall drastically reduced to 15.69% and 0.00% .

From fig. 5.3.3 it is observed that greater number of data fall below the line of perfect agreement.

5.5.4 Comment on Result

Suggested approach over predict sediment concentration, on the other hand Wallingford and revised Wallingford approach under predict sediment concentration. But results obtained by suggested approaches are much closer than the other two approaches.

5.6 LACEY'S APPROACH FOR ASSESSMENT OF CHANNEL DIMENSION

Lacey's approach is widely used in Bangladesh for prediction of channel geometry. It is considered worth while to compare the suggested approach with Lacey's approach. In this comparative study same 209 sets of data have been used.

This approach under predict the width of channel. From table 5.2 it is observed that at lower discharge the mean discrepancy ratio is 0.576 with a range of 1.01 to 0.20 and standard deviation 0.212. At higher discharge the mean discrepancy ratio reduced to 0.554 with a range of 1.81 to 0.91. Standard deviation is 0.208. At lower discharge 6.25% and 10.94% data fall in the discrepancy ratio range 0.833 to 1.20 and 0.714 to 1.40 respectively. Whereas

at higher discharge 9.75% and 11.11% of data fall in the same bands of discrepancy ratio.

This approach over predict the depth of channel. From table 5.4 it is observed that at lower discharge the mean discrepancy ratio is 2.048 with a range of 5.36 to 1.13 and standard deviation 0.614. At higher discharge the mean discrepancy ratio reduced to 1.195 with a range of 4.92 to 0.75. Standard deviation is 0.244. At lower discharge 8.42% and 11.58% data fall in the discrepancy ratio range 0.833 to 1.20 and 0.714 to 1.40 respectively. Whereas at higher discharge 23.68% and 50.87% of data fall in the same bands of discrepancy ratio.

Comparing the results of Lacey's approach with that of Suggested approach it can be concluded that Lacey's approach predicts much narrower and deeper channel than the other approach.

CHAPTER SIX

CONCLUSIONS AND SUGGESTIONS FOR FURTHER STUDY

6.1 CONCLUSIONS

Summarizing the author's investigations the following conclusions are drawn :

- 1) Good agreement is obtained for width prediction in suggested approach, although it slightly over predicts. Wallingford, revised Wallingford and Lacey's approach predict much wider section than suggested approach.
- 2) In higher discharge range i.e discharge greater than 300 m^3/s , revised Wallingford approach gives better prediction in width calculation than Wallingford approach. On the other hand in lower discharge range i.e discharge lesser than 300 m^3/s there is no significant change in width prediction in these two approaches.
- 3) Prediction of channel depth in suggested approach is much better than Wallingford approach, revised Wallingford and Lacey's approach. Wallingford, revised Wallingford and Lacey's

approach predicts deeper channel.

4) In depth prediction there is no significant improvement in revised Wallingford approach as compared to Wallingford approach.

5) Sediment concentration is not computed as accurately as depth or width computation by any of the approaches that have been studied in this study. Suggested approach over predict and Wallingford and revised Wallingford approach under predict sediment concentration. Suggested approach predicts sediment concentration which is much closer to observed value than predicted by the other two approaches.

6) In suggested approach equations are simpler than Wallingford and revised Wallingford approach. Thus the computation time in Suggested approach is less than the other two approaches.

7) Tables showing the channel geommetry variables as generated from the suggested approach have been presented. From these tables (Table 6.1 to 6.3) suitable sizes of channel can be obtained for given discharge, slope and sediment size.

6.2 SUGGESTIONS FOR FURTHER STUDY

The suggested approach for the prediction of channel geometry has been found to be reasonably satisfactory in its present form. But there is still need for additional studies towards its greater consolidation. The author suggests the following recommendation for future studies:

1. With the help of faster computer facilities a set of tables (as shown in Table 6.1 to 6.3) can be generated for wide range of various conditions.
2. It can be verified, whether the suggested approach is also applicable for meandering and gravel bed channels.
3. The applicability of the procedure presented in this study can further be varified against the alluvial canal data collected from rivers and canals in Bangladesh.

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Table 5.1 : Summary of the data sets used for the investigations (nominal values from sources) .

Source	No. of Data	Discharge (cumec)		Sediment conc. (ppm)		Slope *(1000)		Sediment median size (mm)		Water surface breadth (m)		Mean depth (m)	
		min ⁿ	max ⁿ	min ⁿ	max ⁿ	min ⁿ	max ⁿ	min ⁿ	max ⁿ	min ⁿ	max ⁿ	min ⁿ	max ⁿ
Simons Bender	7	4.14	29.19	52.0	447.0	0.058	0.33	0.096	0.715	9.82	22.19	0.89	2.60
UP Canals	28	0.42	281.90	58.0	2010.0	0.102	0.434	0.110	0.410	12.59	56.90	0.49	3.80
ACOP Canals	60	29.47	297.15	15.0	1007.0	0.045	0.150	0.090	0.360	10.76	128.0	0.76	2.53
Atchafalaya River	32	2044.4	14186.3	12.5	7567.0	0.020	0.504	0.105	0.303	316.99	503.92	6.92	10.72
Mississippi River	65	1885.80	28825.67	10.4	329.1	0.02	0.133	0.173	1.129	459.0	1109.0	3.35	10.25
Chitale	8	13442.5	52298.4	14.0	27.2	0.047	0.47	0.10	5.00	274.32	3848.8	1.52	10.12
Snake River	9	971.24	3511.18	13.8	25.0	0.245	1.210	0.051	0.640	137.16	198.12	4.03	5.91

TABLE 5.2 Statistical properties of discrepancy ratio (D.R) for the comparison of results of channel breadth obtained from Suggested, Wallingford, Revised Wallingford and Lacey's approaches against different group of data.

(Value Within the bracket indicates number of data used)

Data Group	Geometric Approach Adopted	Max ⁿ D.R	Min ⁿ D.R	Mean D.R	Standard Deviation
Discharge $Q_3 < 300$ m ³ /sec (95)	Suggested	2.46	0.31	1.06	0.367
	Wallingford	1.51	0.35	0.693	0.221
	Revised Wallingford	1.33	0.36	0.691	0.207
	Lacey	1.01	0.20	0.576	0.212
Discharge $Q_3 > 300$ m ³ /sec (114)	Suggested	3.36	0.30	1.148	0.519
	Wallingford	3.79	0.37	1.121	0.645
	Revised Wallingford	2.69	0.33	1.085	0.513
	Lacey	1.81	0.19	0.554	0.208
All Data Together (209)	Suggested	3.36	0.30	1.08	0.416
	Wallingford	2.79	0.35	0.796	0.417
	Revised Wallingford	2.69	0.33	0.773	0.352
	Lacey	1.81	0.19	0.564	0.409

TABLE 5.3 The percentage of data coverage in selected bands of discrepancy ratio for the comparison of results of channel breadth obtained from Suggested, Wallingford, revised Wallingford and Lacey's approaches against different groups of data.

(value within the bracket indicates number of data used)

Data Group	Geometric Approach Adopted	Percentage of data coverage between discrepancy ratio (D.R) band	
		0.833-1.20	0.714-1.40
Discharge $Q_3 < 300$ m^3/sec (95)	Suggested	41.14	63.92
	Wallingford	21.52	43.04
	Revised Wallingford	21.52	44.94
	Lacey	6.25	10.94
Discharge $Q_3 > 300$ m^3/sec (114)	Suggested	43.14	56.86
	Wallingford	21.57	45.10
	Revised Wallingford	23.53	52.94
	Lacey	9.75	11.11
All Data Together (209)	Suggested	41.63	62.20
	Wallingford	21.53	43.54
	Revised Wallingford	22.00	46.89
	Lacey	8.16	10.51

TABLE 5.4 Statistical properties of discrepancy ratio (D.R) for the comparison of results of channel depth obtained form Suggested, Wallingford, Revised Wallingford and Lacey's approaches against different group of data.

(Value Within the bracket indicates number of data used)

Data Group	Geometric Approach Adopted	Max ⁿ D.R	Min ⁿ D.R	Mean D.R	Standard Deviation
Discharge $Q_3 < 300$ m ³ /sec (95)	Suggested	2.14	0.49	1.061	0.335
	Wallingford	5.76	1.22	2.483	0.873
	Revised Wallingford	5.76	1.35	2.508	0.889
	Lacey	5.36	1.13	2.048	0.614
Discharge $Q_3 > 300$ m ³ /sec (114)	Suggested	2.30	0.48	0.994	0.368
	Wallingford	5.51	0.91	2.039	0.902
	Revised Wallingford	5.55	1.00	2.073	0.878
	Lacey	4.92	0.75	1.195	0.244
All Data Together (209)	Suggested	2.30	0.48	1.024	0.443
	Wallingford	5.76	0.91	2.241	1.271
	Revised Wallingford	5.76	1.00	2.271	1.269
	Lacey	5.36	0.75	1.583	0.641

TABLE 5.5 The percentage of data coverage in selected bands of discrepancy ratio for comparison of results of channel depth obtained from suggested, Wallingford, revised Wallingford and Lacey's approaches against different groups of data.

(value within the bracket indicates number of data used)

Data Group	Geometric Approach Adopted	Percentage of data coverage between discrepancy ratio (D.R) band	
		0.833-1.20	0.714-1.40
Discharge $Q_3 < 300$ m ³ /sec (95)	Suggested	49.37	77.22
	Wallingford	00.00	1.05
	Revised Wallingford	00.00	1.05
	Lacey	8.42	11.58
Discharge $Q_3 > 300$ m ³ /sec (114)	Suggested	41.18	66.67
	Wallingford	11.76	13.73
	Revised Wallingford	03.92	13.73
	Lacey	23.68	50.87
All Data Together (209)	Suggested	44.90	71.46
	Wallingford	6.41	7.97
	Revised Wallingford	2.14	7.97
	Lacey	16.74	33.01

TABLE 5.6 Statistical properties of discrepancy ratio (D.R) for the comparison of results of sediment concentration obtained from Suggested, Wallingford and Revised Wallingford approaches against different group of data.

(Value Within the bracket indicates number of data used).

Data Group	Geometric Approach Adopted	Max ⁿ D.R	Min ⁿ D.R	Mean D.R	Standard Deviation
Discharge $Q_3 < 300$ m^3/sec (95)	Suggested	6.95	0.10	1.44	1.205
	Wallingford	2.80	0.07	0.347	0.457
	Revised Wallingford	2.33	0.05	0.335	0.427
Discharge $Q_3 > 300$ m^3/sec (114)	Suggested	4.77	0.45	1.669	1.081
	Wallingford	3.84	0.07	0.295	0.562
	Revised Wallingford	4.10	0.06	0.461	0.983
All Data Together (209)	Suggested	6.95	0.10	1.417	1.114
	Wallingford	3.48	0.07	0.331	0.448
	Revised Wallingford	7.86	0.05	0.353	0.582

TABLE 5.7 The percentage of data coverage in selected bands of discrepancy ratio (D.R) for the comparison of results of sediment concentration obtained from suggested, Wallingford and revised Wallingford approaches against different groups of data.

(value within the bracket indicates number of data used)

Data Group	Geometric Approach Adopted	Percentage of data coverage between discrepancy ratio (D.R) band	
		0.50-2.00	0.25 -4.00
Discharge $Q_3 < 300$ m^3/sec (95)	Suggested	57.59	86.07
	Wallingford	14.56	36.08
	Revised Wallingford	12.02	32.28
Discharge $Q_3 > 300$ m^3/sec (114)	Suggested	43.14	64.71
	Wallingford	1.96	25.49
	Revised Wallingford	00.00	15.69
All Data Together (209)	Suggested	54.67	80.86
	Wallingford	11.48	33.49
	Revised Wallingford	9.09	28.22

TABLE 6.1: CHANNEL CHARACTERISTICS AS A FUNCTION OF DISCHARGE AND SLOPE

PARTICLE SIZE: 0.15 (mm)

- WIDTH (m)
- DEPTH (m)
- SEDIMENT CONCENTRATION (ppm)

SLOPE ($\times 10^{-3}$)	DISCHARGE (m^3/s)				
	10	20	50	100	200
0.10	86.17	69.85	88.95	110.89	160.72
	0.74	1.13	1.51	1.84	2.48
	28.66	29.62	49.72	76.05	152.96
0.40	35.83	43.23	54.47	71.31	102.30
	0.60	0.74	0.99	1.17	1.58
	301.37	448.12	742.18	1200.98	2401.87
0.80	36.12	36.60	44.97	56.16	80.01
	0.42	0.57	0.77	0.94	1.28
	1473.60	1871.99	3054.34	4704.26	9324.21
1.20	23.53	28.57	39.15	47.50	71.13
	0.43	0.53	0.68	0.84	1.11
	2458.98	3731.79	6782.19	10130.19	21201.84
1.60	26.40	28.09	37.57	44.12	63.66
	0.35	0.47	0.60	0.76	1.02
	5374.74	7176.07	12664.03	18351.86	36891.06
2.00	20.73	24.54	32.41	40.49	60.30
	0.36	0.45	0.58	0.71	0.94
	7139.78	10414.89	18228.04	28124.15	58567.72

TABLE 6.2: CHANNEL CHARACTERISTICS AS A FUNCTION OF DISCHARGE AND SLOPE

PARTICLE SIZE: 0.30 (mm)

- WIDTH (m)
- DEPTH (m)
- SEDIMENT CONCENTRATION (ppm)

SLOPE ($\times 10^{-3}$)	DISCHARGE (m^3/s)				
	10	20	50	100	200
0.10	71.61	70.86	100.96	120.62	178.05
	0.96	1.33	1.70	2.12	2.85
	9.42	11.40	20.81	30.34	61.76
0.40	40.14	47.26	64.60	79.75	116.64
	0.68	0.86	1.10	1.34	1.80
	122.96	177.37	319.41	487.48	992.60
0.80	32.23	38.27	52.16	63.62	91.51
	0.55	0.68	0.87	1.07	1.45
	477.27	711.06	1282.25	1926.14	3862.67
1.20	36.00	36.48	44.79	55.90	79.55
	0.42	0.57	0.77	0.94	1.28
	1355.15	1721.21	2807.73	4323.82	8567.86
1.60	25.02	30.56	45.49	51.31	73.55
	0.44	0.54	0.66	0.85	1.15
	1858.32	2809.37	5544.98	7704.84	15426.30
2.00	23.84	28.80	36.43	46.74	68.80
	0.41	0.50	0.67	0.80	1.07
	2950.59	4399.91	7409.55	11761.47	24171.25

TABLE 6.3: CHANNEL CHARACTERISTICS AS A FUNCTION OF DISCHARGE AND SLOPE

PARTICLE SIZE: 0.60 (mm)

- WIDTH (m)
- DEPTH (m)
- SEDIMENT CONCENTRATION (ppm)

SLOPE ($\times 10^{-3}$)	DISCHARGE (m^3/s)				
	10	20	50	100	200
0.10	51.83	66.02	115.28	126.04	192.98
	1.18	1.53	1.85	2.46	3.29
	3.10	4.33	9.17	11.96	24.70
0.40	44.95	54.47	71.17	90.81	131.74
	0.77	0.96	1.26	1.52	2.05
	51.31	75.47	128.19	201.88	406.43
0.80	35.82	43.58	68.49	73.35	106.60
	0.63	0.77	0.92	1.21	1.63
	196.63	294.04	609.51	805.15	1630.28
1.20	34.38	38.15	51.99	63.39	91.10
	0.53	0.68	0.87	1.07	1.45
	469.78	653.82	1178.58	1770.11	3549.09
1.60	34.52	36.37	45.53	57.36	82.89
	0.46	0.61	0.81	0.98	1.32
	917.06	1200.84	1999.78	3112.50	6274.68
2.00	25.87	37.05	44.40	54.78	76.55
	0.48	0.54	0.74	0.90	1.23
	1154.69	2055.65	3275.38	4986.00	9701.87

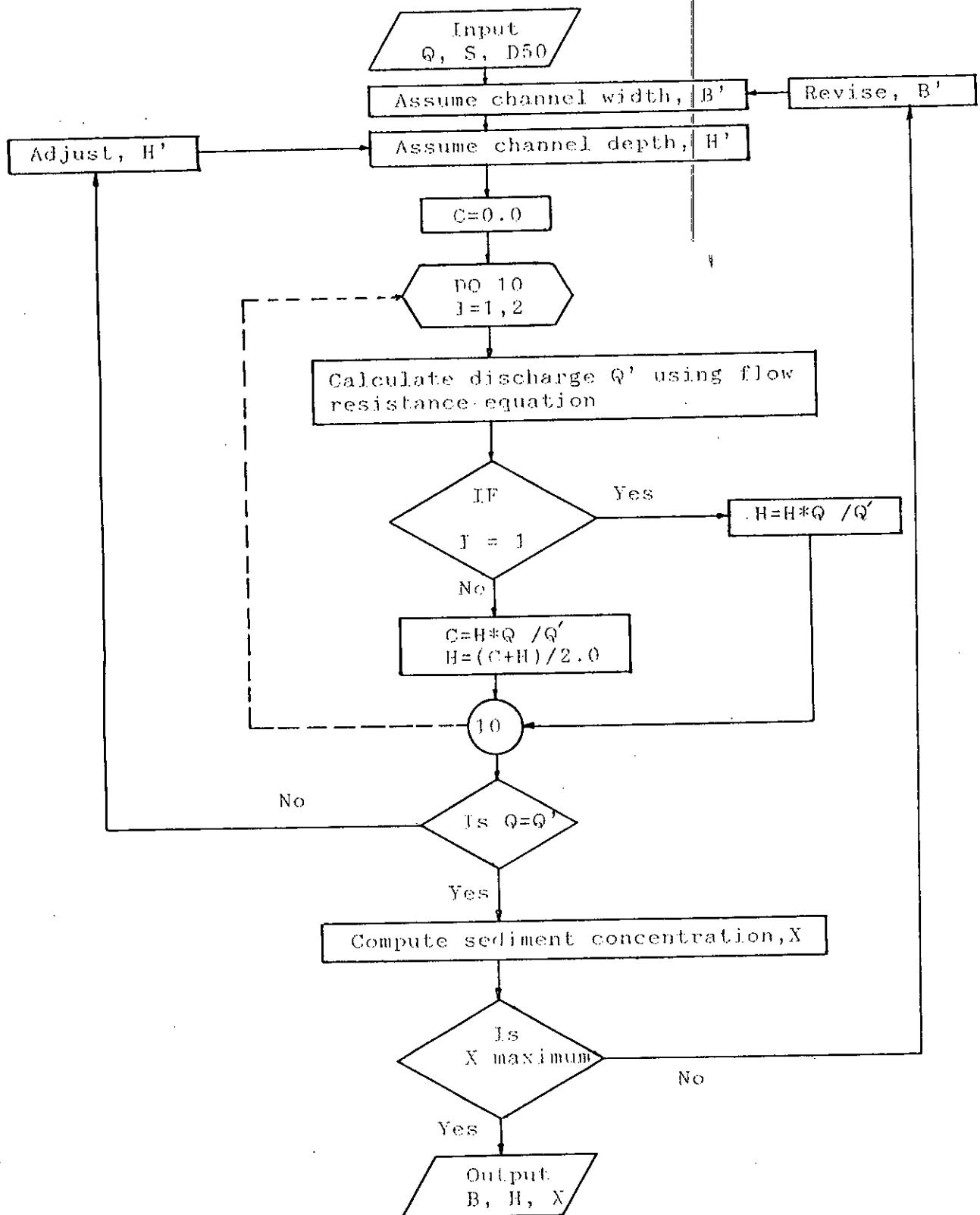


Fig. 4.1 Flow chart for computation of channel geometry adopted in author's approach.

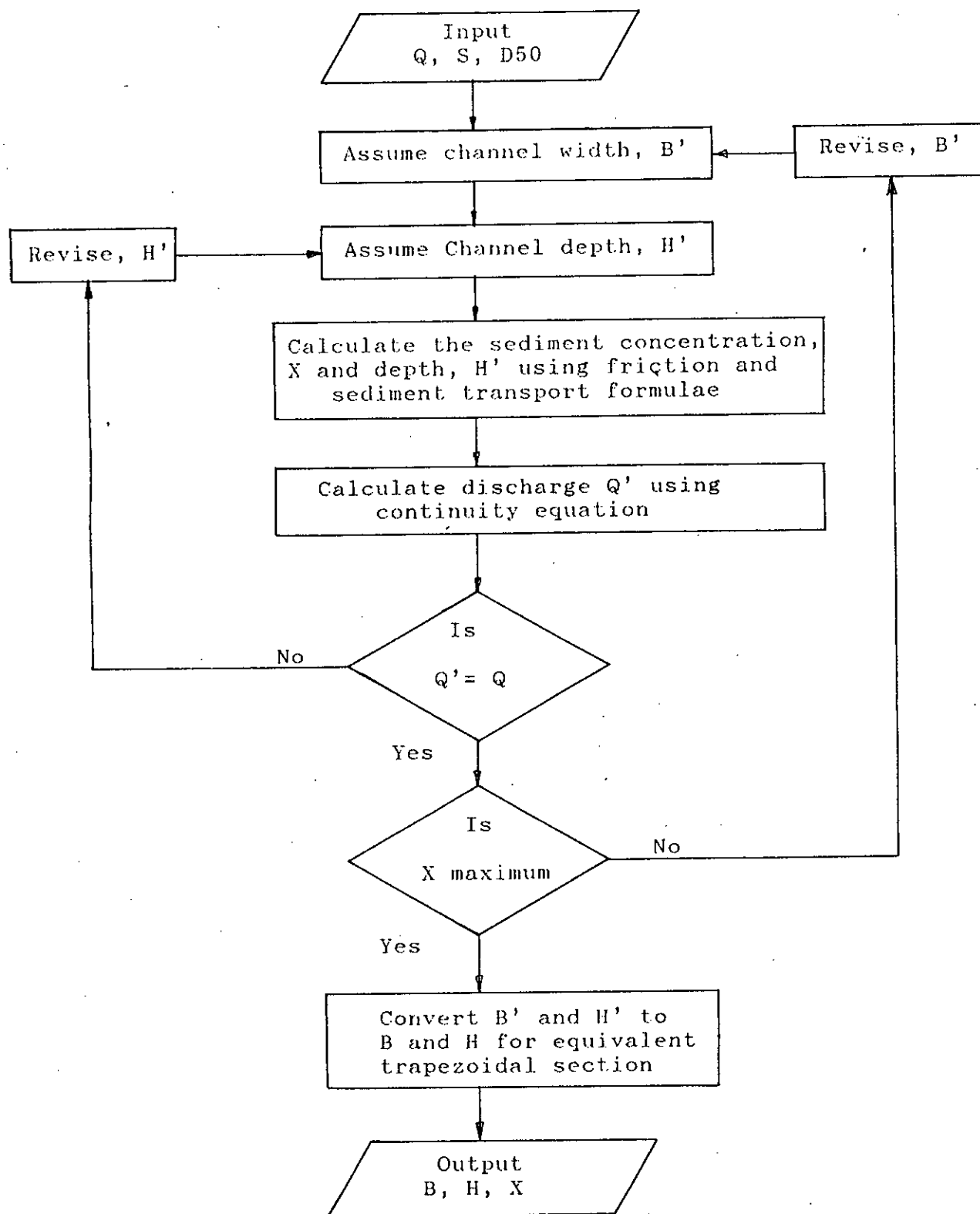


Fig.4.2 Flow chart for computation of channel geometry adopted in Wallingford approach.

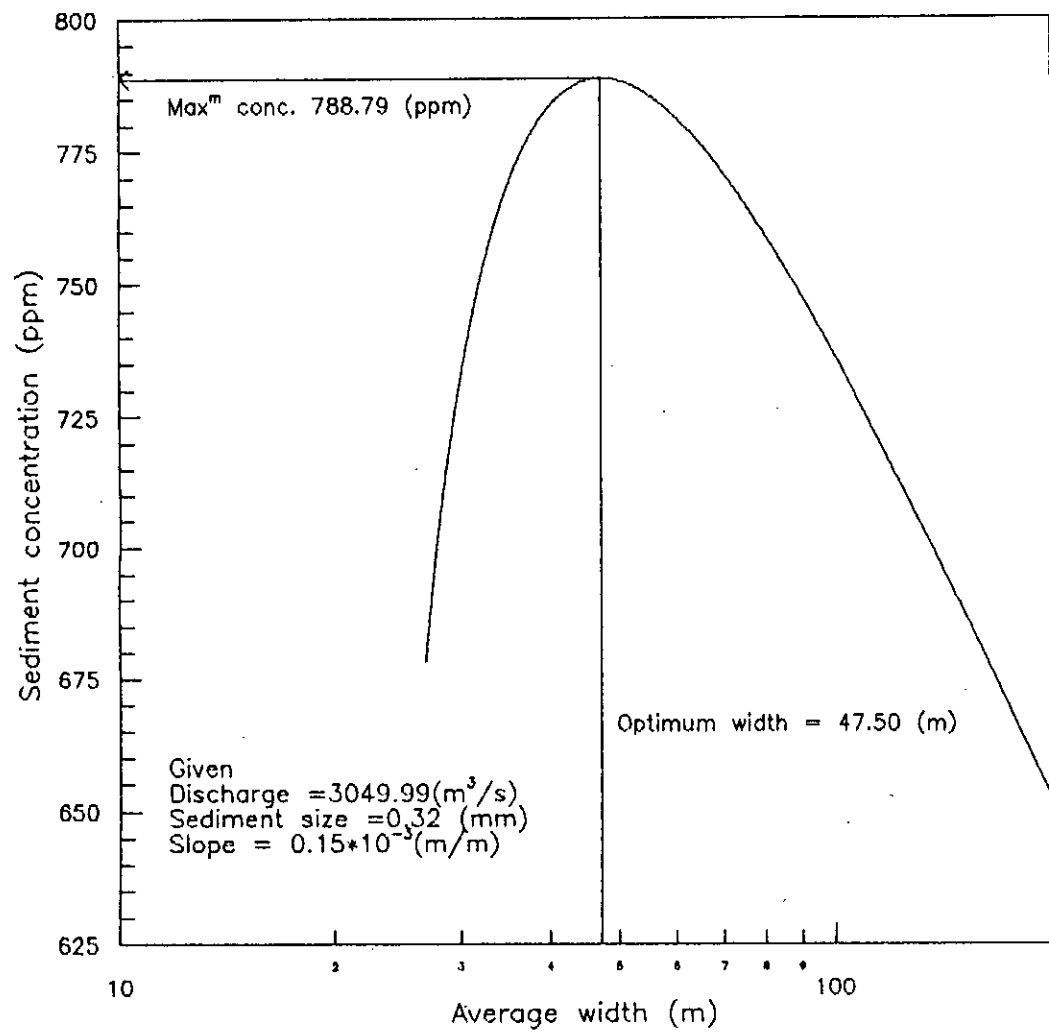


Fig. 4.3 Sediment concentration against width for maximisation of sediment concentration.

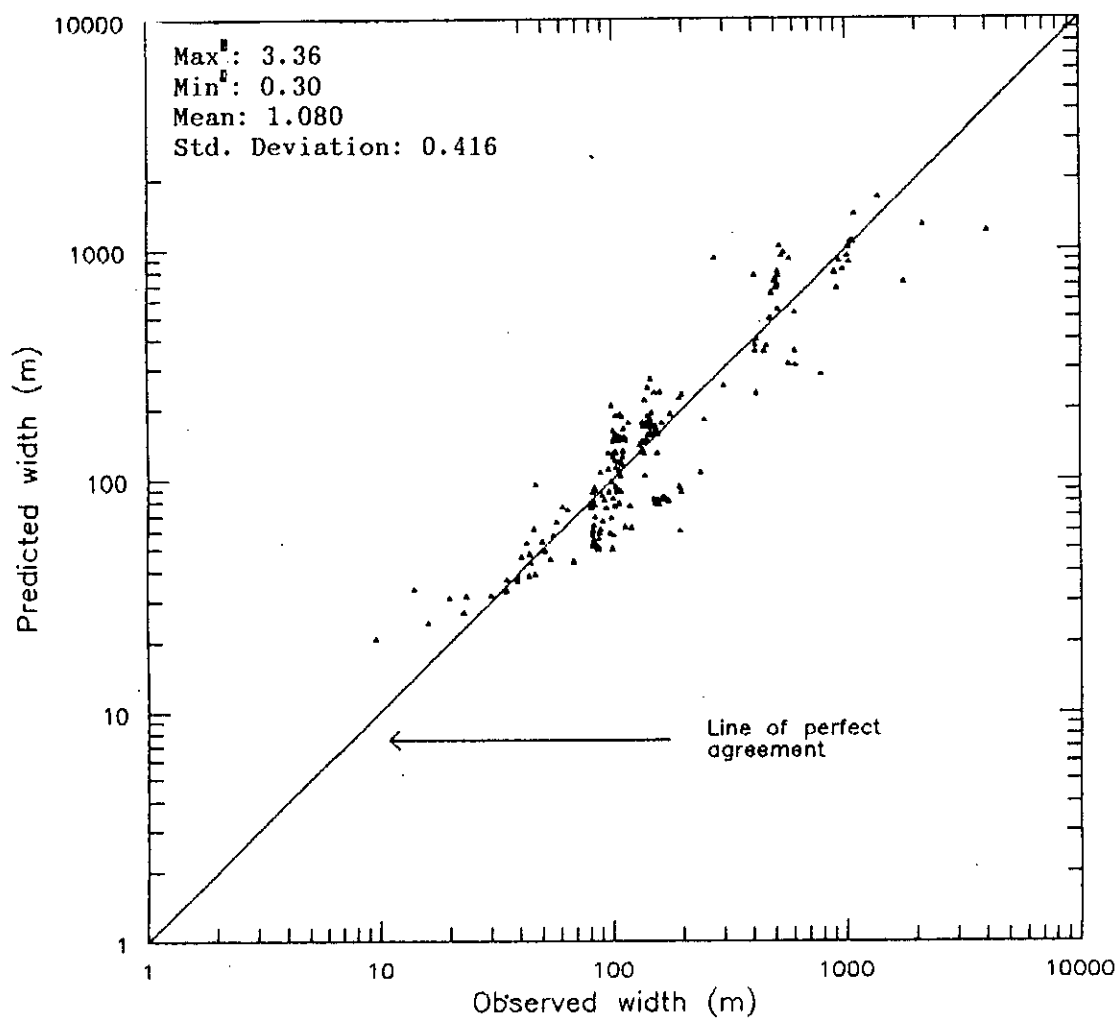


Fig. 5.1.1 Predicted vs observed channel width for suggested approach.

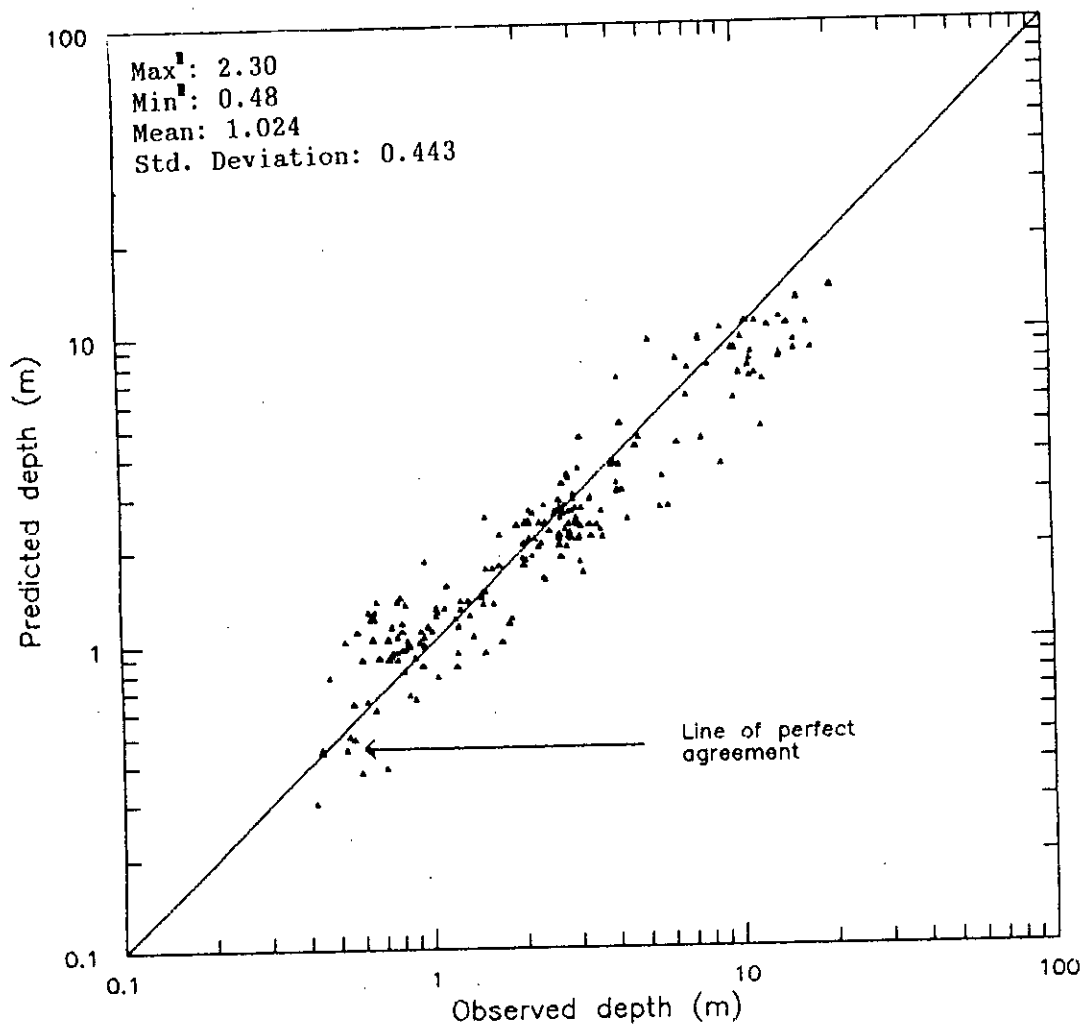


Fig. 5.1.2 Predicted vs observed channel depth for suggested approach.

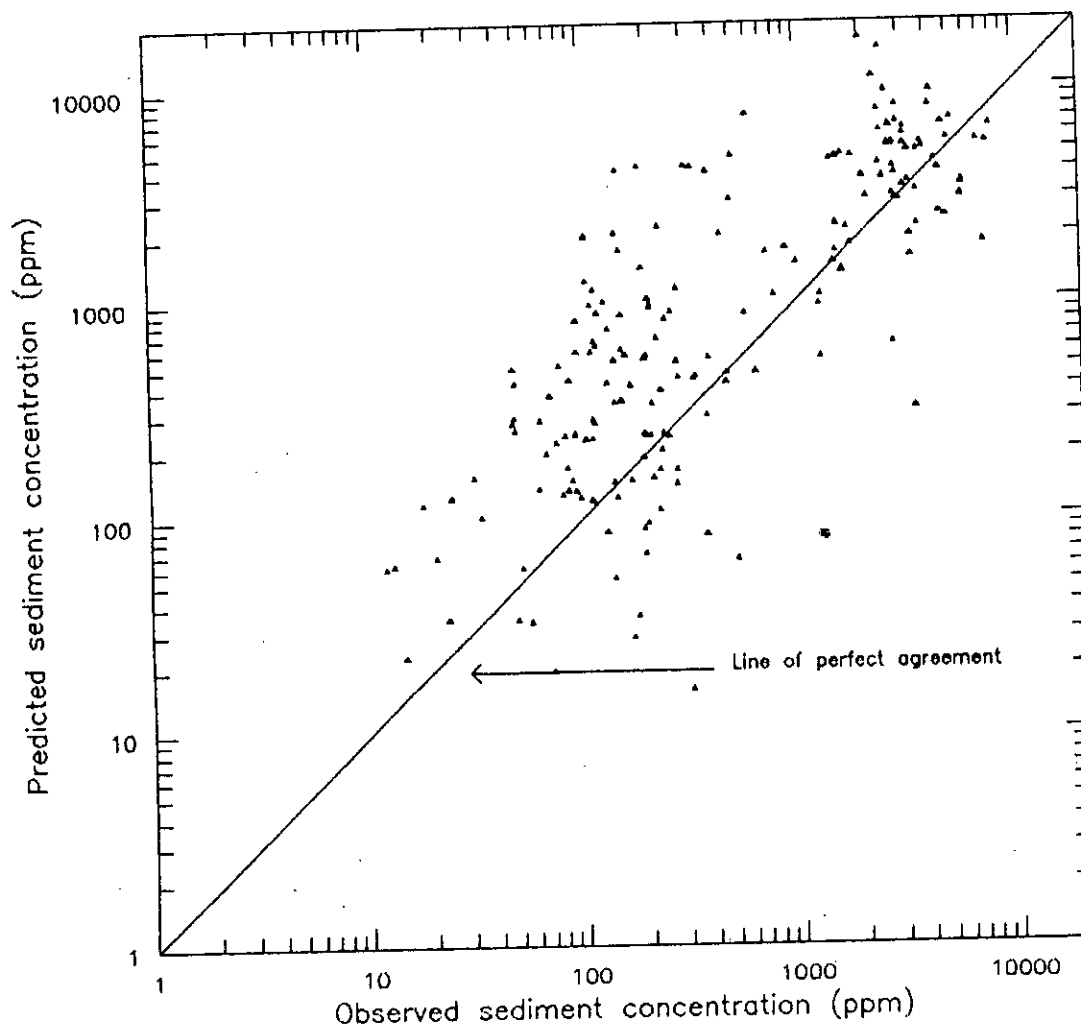


Fig.5.1.3 Predicted vs. observed sediment concentration for suggested approach.

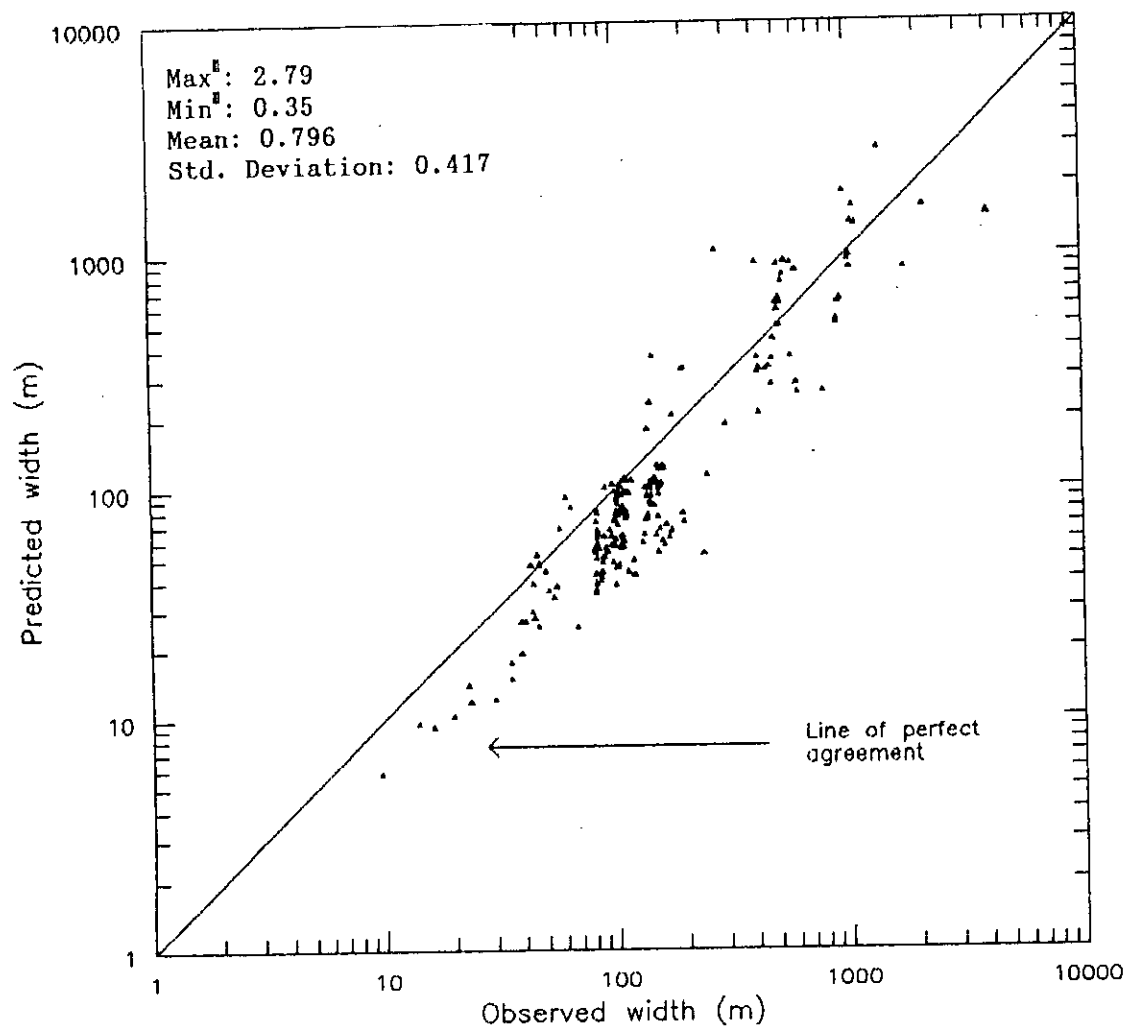


Fig. 5.2.1 Predicted vs. observed channel width for Wallingford approach.

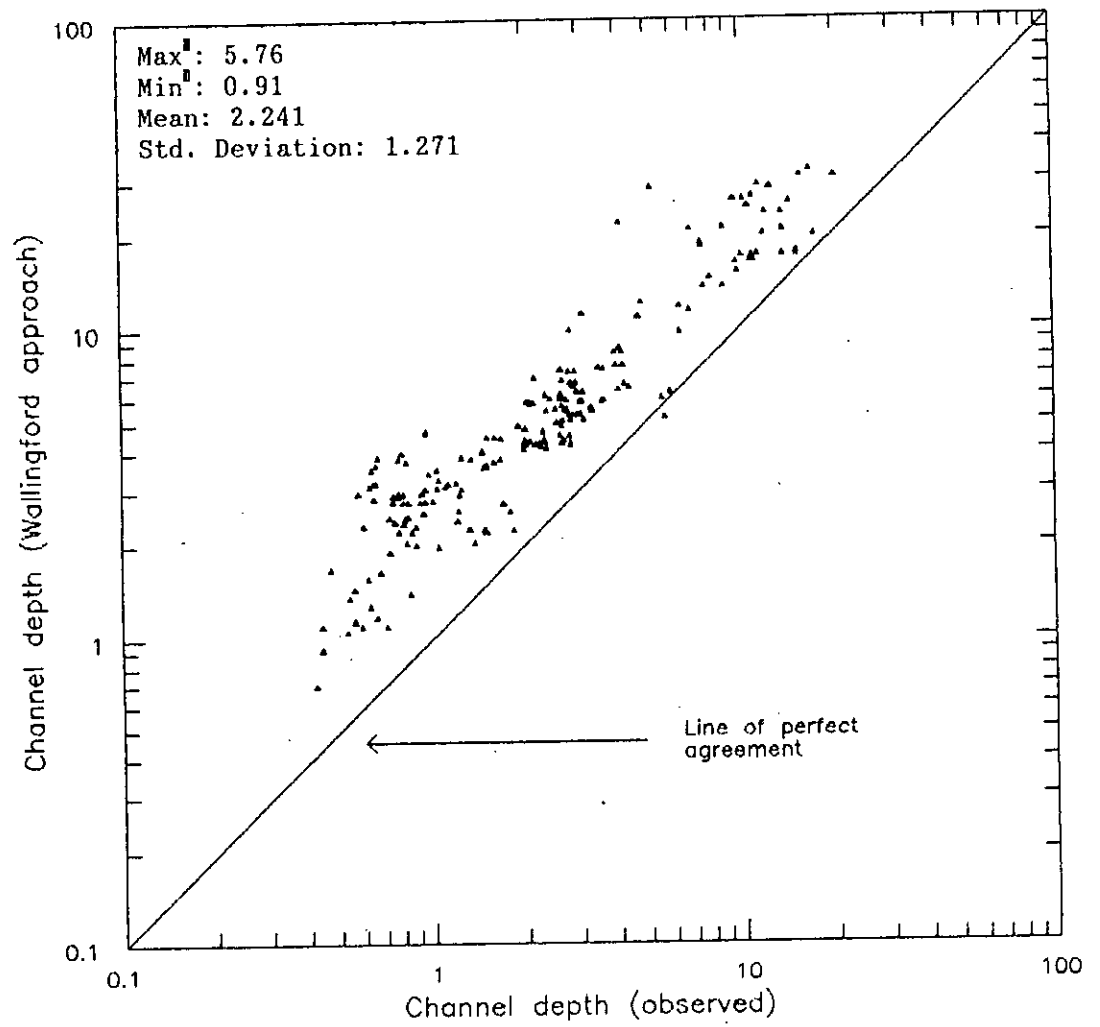


Fig. 5.2.2 Channel depth (Wallingford approach) vs. observed channel depth .

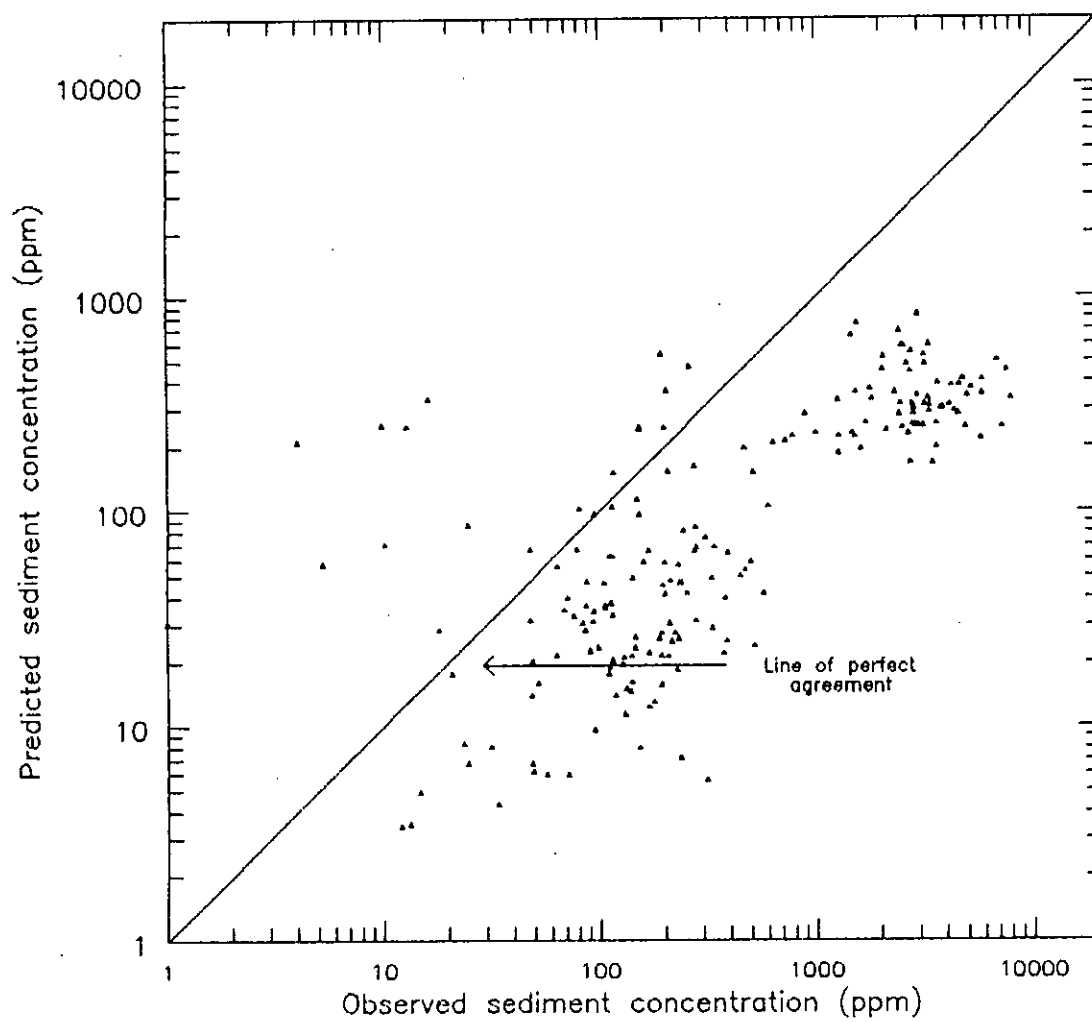


Fig.5.2.3 Predicted vs. observed sediment concentration for Wallingford approach.

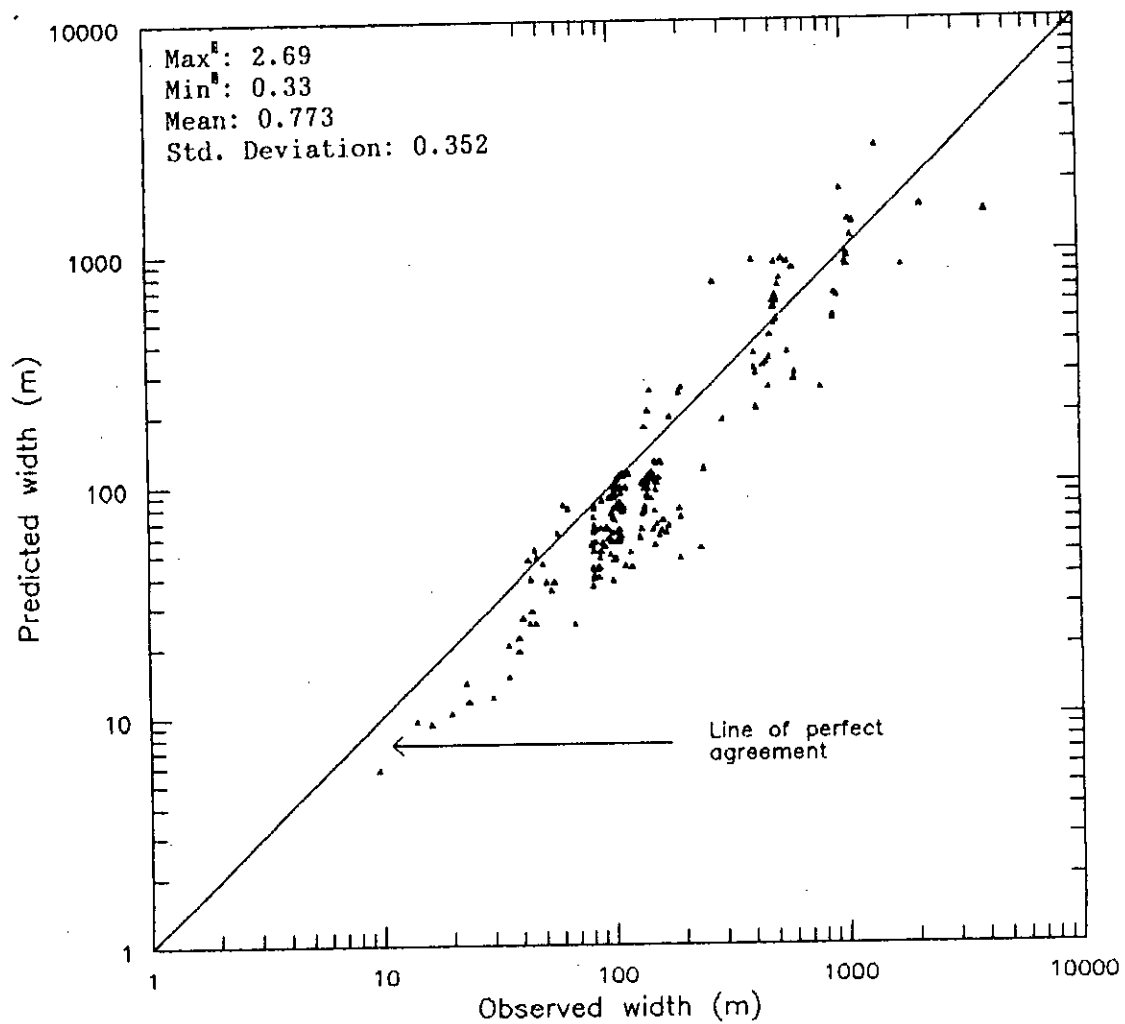


Fig. 5.3.1 Predicted vs. observed channel width for revised Wallingford approach.

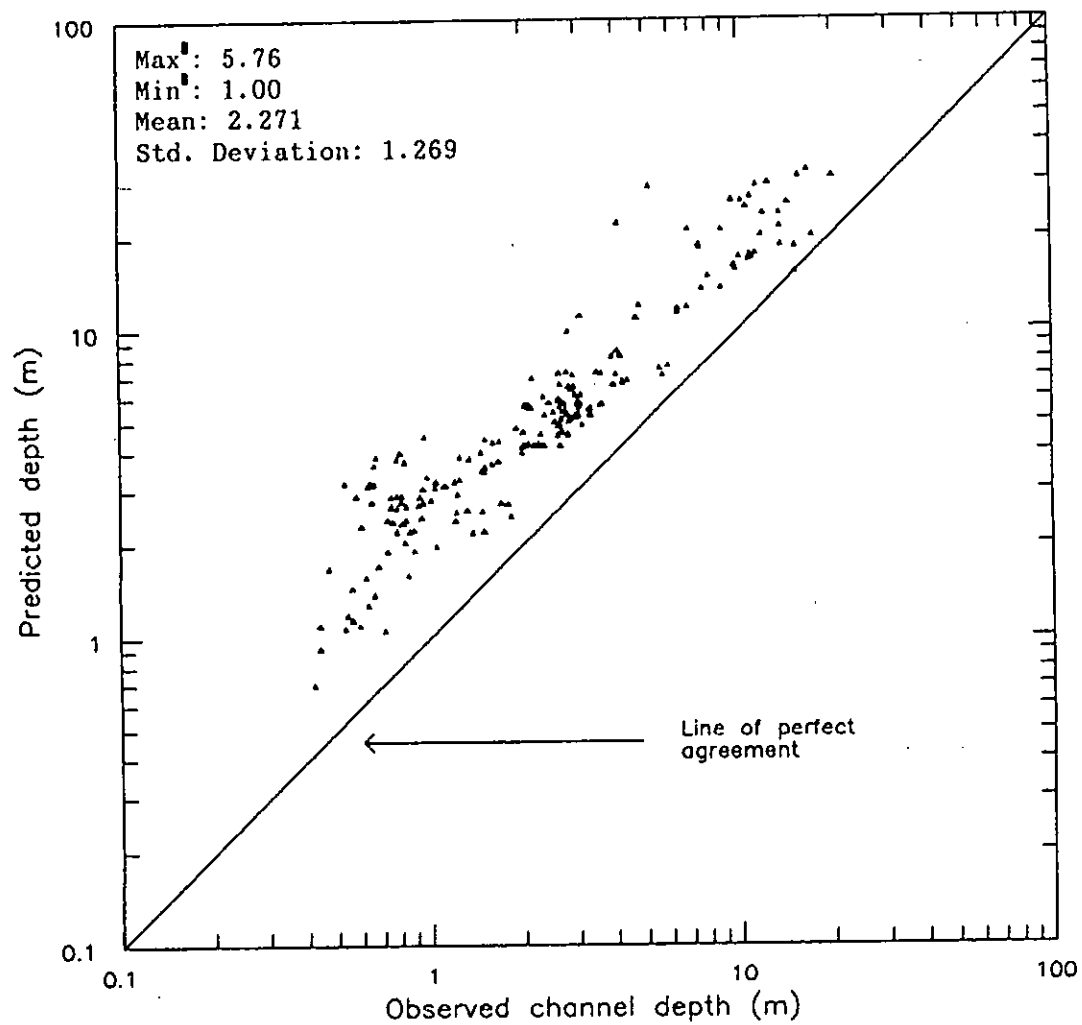


Fig. 5.3.2. Predicted vs. observed channel depth for revised Wallingford approach.

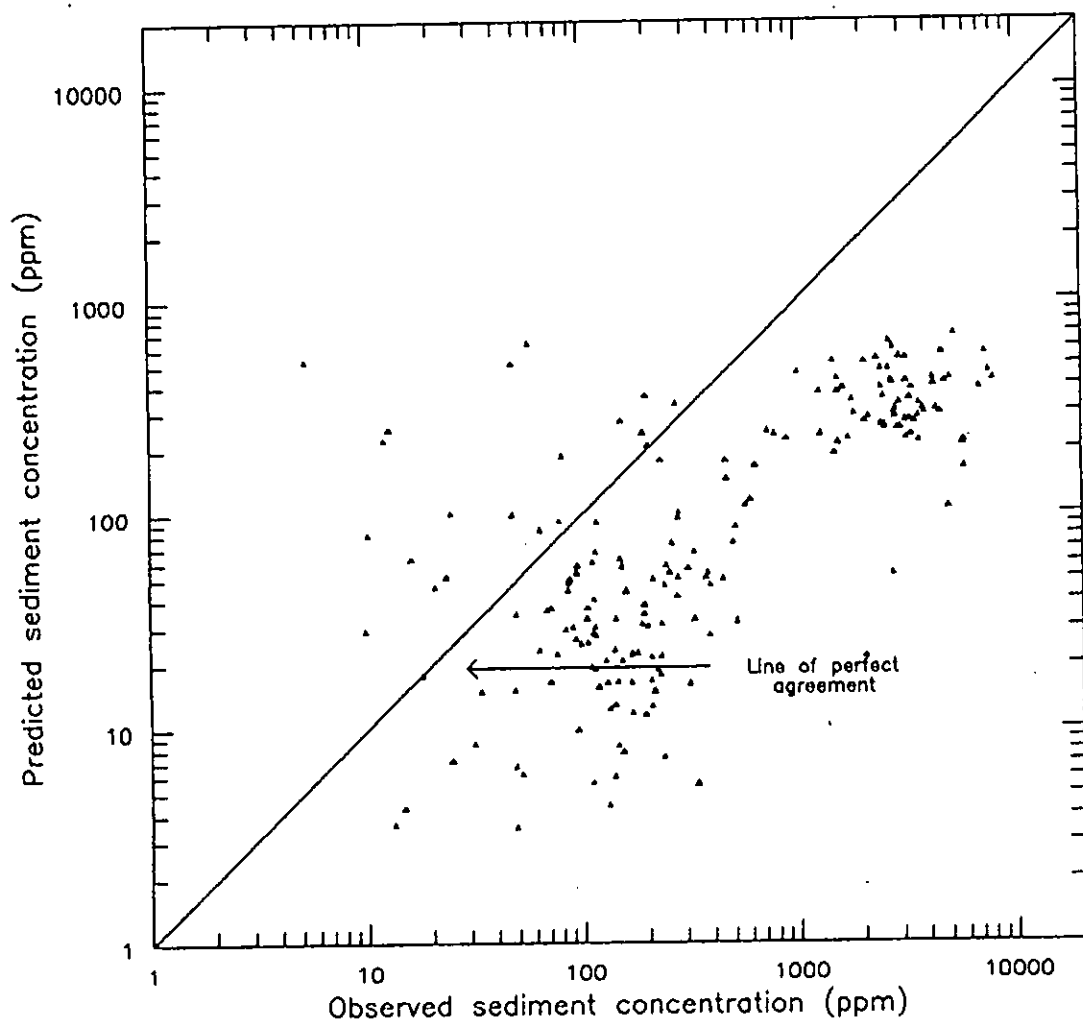


Fig.5.3.3 Predicted vs. observed sediment concentration for revised Wallingford approach.

