EXPERIMENTAL STUDY ON RESISTANCE IN GRAVEL BED CHANNELS

A Thesis Submitted in Partial Fulfilment of the Requirements for the Degree of

Master of Technology
In
Civil Engineering

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DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA

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Under the guidance and supervision of
Prof. K.K. Khatua

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National Institute of Technology
Rourkela

CERTIFICATE

This is to certify that the thesis entitled "**Experimental Study on Resistancein Gravel Bed Channels**" being submitted by Sumit Kumar Banerjee in partial fulfillment of the requirements for the award of **Master of Technology** in **Civil Engineering** at National Institute of Technology Rourkela, is a bonafide research carried out by his under my guidance and supervision.

The work incorporated in this thesis has not been, to the best of our knowledge, submitted to any other University or Institute for the award of any degree or diploma.

Prof. K.K. Khatua
(Supervisor)
Date:

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Sumit Kumar Banerjee

ABSTRACT

River beds are frequently experienced by gravel beds the analysis of which is very interesting and challenging to the river engineers and researchers. Gravel bed can be categorized it into three bedload condition, i.e. no load, moderate and intense. The present study investigates the roughness characteristics of gravel bed open channel flows under both no load and intense load conditions over various discharges and flow depths. Experimental investigation has been carried out in an open channel flow with Gravel bed surface of grain size of D_{50} values 13.5 mm and 6.5 mm for both no load and intense load conditions respectively.

Measurement for the velocity and the boundary shear stress of the gravel bed condition has been determined. The longitudinal slope of the channel is 0.25%. Hence, for the case of gravel size 13.5mm, gravel is not transported, thus a case of no load condition occurs. Whereas in the case of 6.5mm gravel size, the gravel is transported under the condition of intense load such that in the particular case the bed load transport rate has been determined. Variation of friction factor for both the roughness conditions for different flow depths has been estimated. The intensity of the bed load has also been calculated with the help of sediment transport rate.

It was observed that bed load was falling with increasing grain sizes under both no load and intense load conditions. Hence, an improved model for prediction of friction factor has been devised. This model has been validated with the data set of other investigators, showing satisfactory values pertaining to the actual values of Darcy's *f*. The developed model has also been compared with the models of other researchers and is found to provide better results than those given by others. The bed load transport rate for the intense load conditions was also determined from experimentation on gravel beds of 6.5mm gravel size for different flow depths. By the use of the data set of other researchers and present experimental data, a new model to formulate intensity of the bed load transport with respect to Shield's parameter has been formulated. The modified model is found to provide satisfactory results as compared to previous works.

Keywords: *gravel bed open channel, Darcy-Weisbach friction factor, bed load, sediment transport, Shield's parameter, boundary shear stress.*

CONTENTS

LIST OF ABBREVIATIONS AND SYMBOLS

A	Area of channel cross section
A_b	Area static by the bed resistance
A_w	Area balanced by the sidewalls resistance.
B	Channel Base width
C	Chezy's channel coefficient
C_b	Bed load Concentration
d	Diameter of the preston tube
d_*	Dimensionless particle parameter
D	Diameter of the gravel
D_m	Hydraulic mean depth of the channel
D_{50}	50 % grain size diameter at
E	Nash–Sutcliffe coefficient
f	Darcy-Weisbach's friction factor
F_r	Froude's number
δ_b	Saltation height
g	Acceleration due to gravity
h	Flow depth
H	Depth of the channel
I_d	Index of agreement
n	Manning's roughness coefficient
P	Wetted Perimeter
q_b	Bedload sediment transport rate
Q	Channel discharge
R	Hydraulic radius, defined as flow area/wetted perimeter
R^2	Coefficient of Determination
Re	Reynolds number
R_e^*	Roughness Reynolds number
S	Channel bed slope
SF_w	Percentage error
T	Transport stage parameter
u	Measured velocity by manometer
ub	Sediment velocity

$u*$	Shear velocity
v	Velocity at any point in the vertical plane of the flow
x'	Calibration coefficient
y'	Calibration coefficient
Δh	Difference in water elevation in manometer
Δp	Pressure difference in manometer
θ	Shields parameter for bed shear stress
θ_c	Critical Shields parameter for bed shear stress
τ	Boundary shear stress of the channel
τ_c	Critical Boundary shear stress of the channel
v	Kinematic viscosity of water
z	Distance between the free surface with arbitrary datum
z_0	Fraction of the bed roughness
ρ	Density of water
α	Angle of manometer
λ	Lateral shear
Γ	Secondary flow effects coefficients
σ	Standard deviation
Φ	Dimensionless intensity of the bed load rate
γ	Specific weight of water
γ_s	Specific weight of sediment

CHAPTER1

INTRODUCTION

INTRODUCTION

1.1 General

The most essential resources of water flow are either through pipe or open channels. It comes to earth through precipitation and then goes through open channels. In the open channel flow the free surface study is most important. Despite the similarity between the pipe flow & open channels flows, it is a lot harder to figure out and analyse the problems associated with the flow in open channels rather than in pipes. Generally, open channel flows are complicated. The depth of flow, the discharge, and the bottom slope of the channel are totally dependent upon the prediction of depth and velocity of free surface flow. Roughness is the main factor in open channels flow study. The roughnessof a pipe is varying from poised metal to rusted iron but for open channels, it can be of poised metal, vegetation, a different type of sediment etc.

It has been found that the open channel flow depth changes, irrespectively of large and low value for different geometry and flow conditions. For case in point of water treatment plants, the flow depth differs between a few centimetres while in large rivers, depth of the flow differs over 10 m. In chemical farm the total discharges range is extended from 0.001 l/s but on the other hand in large rivers or spillways, it can be larger than 10000 m^3/s. In all cases, roughness changes and is important for study which influences the prediction of flow parameters.

1.2. Velocity distribution in Open channels

Velocity in an open channel flow continuously fluctuates because of friction generate from the boundary. The velocity distributions in open channels are usually unsymmetrical due to the presence of the free surface and bed surface. The maximum velocity occurs just below the free surface and the shear stress is zero at the free surface.

1.3. Boundary shear stress distribution in Open Channels

Boundary shear play a critical role in estimating flow carrying capacity of a channel, sediment transportation, erosion of the river. For smooth and rigid channels this has been studied by many investigators. Boundary shear distribution in straight channel shaving roughness variation is less to come across in literature.

The study shown in this thesis is based upon a series of lab experiments in which a straight channel having sediment loads are under consideration. A little consideration has been devoted to the direct effects of bed load on the friction factor *f*, perhaps because sediment transport rates are frequently so low to consider channel bed as rigid bed conditions.

In many instances of lab research, experiments were conducted on straight channels with different geometries and depths and then by measuring the point boundary shear stress (τ) across the straight section. The distribution of boundary shear stress around the wetted perimeter in open channels is known to depend on upon the form of the cross section, the longitudinal variation in platform geometry, the boundary roughness distribution and the structure of secondary currents. The importance of understanding boundary shear stress distributions is shown by the habit of local or mean boundary shear stress in many hydraulic equations concerning resistance, sediment, and dispersion or cavitation problems. For estimation of the bed load transfer in open channel flows, one must divide the bottom shear stress from the total shear stress. Likewise, one must know about the sidewall shear stress to study channel movement or to stop the erosion of the bank. Moreover, a sidewall correction procedure is frequently required in laboratory flume studies in velocity profiles, bed form resistance and sediment transfer. Nevertheless, accurate computation of the local or mean shear stress is a difficult task even using sophisticated turbulence models. As an alternative, various empirical, analytical or simplified computational methods were developed. More or less of them relies on splitting the channel cross section into sub-regions in which, the weight of fluid is balanced by a shear force acting along the corresponding wall sections for computation of the local, mean wall, and the mean bed shear stress in channels.

1.4. Sediment Transport

Sediments can be identified as a fragmentary part of the earth material eroded, transported and deposited elsewhere naturally by causes like water and air. Sediment process is a natural occurring process and hence control over it is very difficult. The problem of sedimentation of an open channel flow consists of

1. Erosion at the place of source
2. Transportation through the water
3. Deposition in the channel

Generally, two types of sediment transport can be distinguished:

- A very irregular transport, which is placed near to the bottom, namely bed load. Sediments are under the lift and drag force control and move by rolling, sliding and saltation over short lengths. This kind of transports is like sand, gravel, and cobbles.

- A transport under turbulence and local flow velocity control are suspended load. This transport concerns fine (sand) to very fine (clay, silt) sediments, which can move over very long lengths.

1.4.1.Effects of Bed load on the friction coefficient

The effects of bed load on the estimation of friction factor *f*, is possible because of sediment transport rates are generally so low (Hey 1979, Van Rijn 1982, Whiting and Dietrich 1990).The bed load is commonly taken into account only in terms of the additional resistance caused by bed forms in the case of sandy rivers (Einstein and Barbarossa 1951, Van Rijn 1982, Wu and Wang 1999). Yet evidence does exist from earlier studies that bed load may have, at least under some conditions, a substantial impact on the friction factor. Long ago Meyer-Peter and Muller (1948) proposed using a correction factor to take into account such effects, even over flat beds (Wong and Parker 2006). By comparing resistance produced by flow over a mobile bed, Bathurst et al. (1982a) described a sharp increase in flow resistance with a slope (varying from 3% to 9%) as a direct consequence of the bed load concentration. Wiberg and Rubin (1989) observed that in upper plane bed conditions, the friction factor associated with sediment transport could reach much higher value than those measured with clear water flows. More generally, it is largely accepted that the introduction of suspended sediment into a clear flow can either amplify or dampen turbulence depending on the relative magnitude of flow and sediment transport variables (Carbonneau and Bergeron 2000), it is only in the last few years research has taken a clear interest in the effects of bed load on the estimation of friction factor.

All experimental procedures have compared the resistance of a clear water flow with that caused by the injection of sediments. Injection of sediment into a clear water flow increases the resistance gradually with the quantity injected until it attains a plateau when the sediment rate is close to equilibrium conditions (Bergeron and Carbonneau 1999, Carbonneau and Bergeron 2000, Omid et al. 2003, Gao and Abrahams 2004, Calomino et al. 2004, Mahdavi and Omid 2004, Campbell et al. 2005). The effect of bedload on the friction factor in equilibrium flow conditions has received only very little attention in the literature. A few studies (Song et al. 1998, Gao and Abrahams 2004) have shown that under such conditions the friction factor increases with the sediment concentration. A step has been taken to study experimentally the friction factor and intensity of bed load in a gravel bed carried by a trapezoidal channel for different sediment diameters.

1.5. Objective of Present Study

Flow in an open channel is generally turbulent in nature. It has been studied from literature that friction factor and intensity of bed load depend on the lateral distribution of depth-averaged velocity and boundary shear stress distribution of a gravel bed channel. However, lack of qualitative and quantitative experimental data on the depth-averaged velocity and boundary shear stress in a simple trapezoidal channel with different sediment size is still a matter of concern. The present study aims to collect velocity and boundary shear stress data of gravel bed channels from different flow depths and focuses the following objectives:

- To study the distribution of streamwise depth-averaged velocity and boundary shear stress at different flow depths of gravel bed channels.
- To verify the applicability of two hydraulic software tools, i.e. CES & ANSYS-FLUENT for the experimental gravel bed channels under different flow conditions for experimental results of velocity distribution, boundary shear stress distribution.
- To develop models for predicting friction factor for both no load conditions and intense load conditions and validating the same, corresponding to models of other investigators.
- Also, to formulate an expression to estimate the bed load intensity for intense load conditions and verifies the same with other investigators.

1.6. Scope of the Study

Friction factor prediction is of primary importance in open channel hydraulics. Present experimental investigation at NIT Rourkela is performed on straight trapezoidal channels with different gravel bed conditions. This would assist as a means to understand the flow processes. Different gravel beds used were found to implicate two different bed load conditions i.e. no load and intense load with respect to the bed slope and the gravel size. Hence, the expense of the study has been broadened to both these load conditions, involving the bed load transport rate.

The study also presents the distribution of velocity and boundary shear stress distribution, a variation of friction factor and in comprehending the intensity of bed load, in, unlike bed load conditions. The analysis of friction factor and intensity of bed load provides an insight into the flow mechanism and resistance relationships for gravel bed channels. Mathematical models to predict Darcy's friction factor and the intensity of bed load developed for these flow conditions will be helpful in predicting velocity, discharge, and resistance relationships of an open channel flow with gravel bed surfaces.

1.7. Organization of the Thesis

The thesis comprises of six chapters.

Chapter 1 gives a brief introduction to the research. The need of research, scope and objectives of the present study along with the relevant background information is mentioned in this chapter.

Chapter 2 presents the follow up of the literature of various pioneer investigators in the field of the straight open channel with no load and intense bed load conditions. The work done by the various researchers on hydraulic resistances including velocity distribution, boundary shear stress, friction factor for both no load and intense load conditions and intensity of the bed load are given in this chapter

Chapter 3 defines an experimental program which contains experimental procedure, construction of channels, sediment loads and experimental measurement of water velocity, discharge, boundary shear stress, bed load sediment transport rate.

Chapter 4 consists of theoretical considerations for evaluation of friction factor and the intensity of the bed load. Various formulations to calculate the friction factor and the bed load intensity are also discussed. Important hydraulic software like Conveyance Estimation System (CES) and ANSYS are elaborated and methodology of such techniques are applied sequentially to develop flow predictive models.

Chapter 5 presents the results and discussions of various experimental data obtained in the study. Results concerning different Friction Factor, Bed load Intensity, Boundary Shear

Stress, Velocity Distributions, development of models to compute friction factor for both the conditions and bed load intensity are presented. Verification the models of other investigators as well as the present models are also narrated in this chapter.

Chapter 6 is the last chapter of the thesis. Significant conclusions of the research work for both friction factor and the intensity of the bed load are briefly demonstrated in this chapter.

Scope for the future research work is also presented after the end of this chapter. All the references that have been used in the present thesis work are listed at the end of the thesis.

CHAPTER2

LITERATURE REVIEW

LITERATURE REVIEW

2.1. Velocity distribution

Sarma *et al*. (2000) formulate velocity distribution law by taking velocitydipinto account in open channel flows. He uses generalized form of binary version of velocity distribution, in which for the inner region logarithmic law and for the outer region parabolic law are combined.

Wilkerson *et al*. (2005) developed two models for predicting depth-averaged velocity distributions. The 1st model is used when the depth-averaged velocity data is available and the 2nd model is used only when predicted depth-averaged velocities are within the range of 20% of actual velocities. He uses data of three previous studies for straight trapezoidal channels having a small width due to which form drag on the fluid exerted by the bank is dominant and thereby the depth-averaged velocity distribution is controlled. The data they used for building up the model are free from the effect of secondary current. The 1st model required to measure velocity data for fine-tuning the model coefficients, whereas the 2nd model used prescribed coefficients.

Knight *et al*. (2007) calculate the lateral distributions of depth-averaged velocity and boundary shear stress by using new approach Shiono and Knight (1988) Method (SKM) for flows in straight prismatic channels which also accounted secondary flow effect. It justifies for bed shear, lateral shear, and secondary flow effects coefficients- $\tau, \lambda,$ and Γ —thus incorporating some key 3D flow feature into a lateral distribution model for stream wise motion. This method used to analyse in the straight trapezoidal open channel. The number of secondary current varies with aspect ratio. For aspect ratio less than equal to 2.2, numbers of secondary current are three and for aspect ratio greater than equal to 4, number of secondary current four.

Afzal *et al*. (2007) examine the power law velocity profile in terms of envelope of the friction factor for fully developing turbulent pipe and channel flows. The model so developed gives a good approximation for low Reynolds number in designing process of actual system compared to large law.

Yang (2010) examines depth-average shear stress and velocity in rough channels. The equations are derived for the depth-averaged shear stress in typical open channels based on a theoretical relation between the depth-averaged shear stress and boundary shear stress. He also developed an equation for depth mean velocity in a rough channel which include the effects of water surface (or dip phenomenon) and roughness. For verification, experimental data available in the literature have been used. The results obtained from the developed model shows close resemblance with the measured data.

Oscar Castro-Orgaz (2010) uses the available data on turbulent velocity profiles in steep chute flow; to investigate the general law by taking into account both the laws of the wall and wake. Once the velocity profile is defined, an equivalent power-law velocity approximation is proposed, with generalised coefficients determined by the rational approach. The results obtained for the turbulent velocity profiles were applied to analytically determine the resistance characteristics for chute flows.

Albayrak *et al*. (2011) conduct experiment on a wide channel having a rough movable (not moving) bed, with a higher bed roughness and at higher Reynolds number of analyse secondary current dynamics within the water column and free surface of an open channel flow. He combined the results of three instruments, acoustic Doppler velocity profiler (ADVP), large-scale particle image velocimetry (LSPIV) and hot film for detailed measurements.

Kundu and Ghoshal (2012) proposed that there are two regions for outer region of the wake layer; one the weak outer region and the other one are strong outer region. His study is focused on flume experimental data for open channel flows to re-investigating the velocity distribution. He proposed an explicit equation for the mean velocity distribution of steady and uniform turbulent flow through straight open channels by combining the log law for inner region and the parabolic law for the relatively strong outer region and verified it with the experimental data. It is found that sediment concentration plays an important role and has a significant effect on the velocity distribution for the relatively weak outer region.

2.2. Boundary shear stress distribution

Earlier works on open channel hydraulics involves experimental studies on the simple straight rectangular channel. Seven decades ago, Leighly (1932) suggested an idea of using conformal mapping to express the boundary shear stress distribution in open-channel flow. He focused that, if the secondary currents is absence then the boundary shear stress at the bed surface must be static.

Einstein's (1942) hydraulic radius separation method is still broadly used in laboratory studies and as well as in field. Einstein distinguished the cross-sectional area into two different areas A_b and A_w and anticipated that at downstream component of the fluid area A_b was static by the bed resistance. Similarly, A_w was balanced by the sidewalls resistance. No friction was occurring at the intermediate position the two areas A_b and A_w. The potential energy delivered by A_b was decreased by the bed surface, and the potential energy providing by area A_w was decreased by the sidewalls. However, he did not propose any method of determining the exact location of division line.

Knight and Sterling (2000) use Preston-tube technique and observed the distribution of boundary shear stress in circular conduits flowing partially full with and without a smooth, flat bed for a data ranging from 0.375<τ<1.96 and 6.5*104<τ<3.42*105. His study shows that the distribution of boundary shear stress depends on geometry and Froude number. The results have been analysed in terms of variation of local shear stress with perimetric distance and the percentage of total shear force acting on wall or bed of the conduit. The results of $\%SF_W$ have been shown to agree well with Knight's (1981) empirical formula for prismatic channels. The interdependency of secondary flow and boundary shear stress has been established and its implications for sediment transport have also been examined.

Yang and McCorquodale (2004) developed a method by applying an order of magnitude analysis to incorporate the Reynolds equations in smooth rectangular channels to compute the three-dimensional Reynolds shear stresses and boundary shear stress distribution. He hypothesized a simplified relationship between the lateral and vertical terms so that the Reynolds equations become solvable. The relationship was developed in the form of a power law with an exponent of n = 1, 2, or infinity. The semi-empirical equations for the boundary shear distribution and the distribution of Reynolds shear stresses were compared with measured data in open channels. The power-law exponent of 2 gave the best overall results while n = infinity gave good results near the boundary.

Guo and Julien (2005) solve the continuity and momentum equations for smooth rectangular open-channel flows and proposed a method for determining the average bed and sidewall shear stresses. The study shows that the shear stresses depend on three components: (1) gravitational, (2) secondary flows, and (3) interfacial shear stress. An analytical solution was obtained in terms of the series expansion for the case of constant eddy viscosity with no inclusion of secondary currents. The method proposed is slightly overestimated the average bed shear stress measurements and underestimated the average sidewall shear stress by 17% when the aspect ratio becomes large when compared with laboratory measurements. He introduced two empirical correction factors for the formulation of second approximation. The second approximation agreed very well (R^2> 0.99 and average relative error less than 6%) with experimental measurements over a wide range of width aspect ratios. Lashkar and Fathi (2010) conducted experiments on the rectangular channels to determine the effect of wall shear force on total boundary shear force. He develops equations and analyse the results using nonlinear regression-based technique to determine the percentage of wall and bed shear force on the wetted perimeter for the rectangular channels.

2.3. Friction Factor

Friction factor is another important resistance parameter that affects the flow in an open channel. The values of friction factor depend on several parameters and primarily on those parameters that affect Manning's roughness coefficient. Amongst a host of factors, the vegetation, sediment, sand and its type, height and density are the primary dominant factor that influences the magnitude of friction factor.

Manning-Strickler (1923) proposed a model for determining the mean velocity flow there he introduced a roughness value later which was defined by K_s=21. $1/D^{1/6}$ (Graf and Altinakar 2000).

Keulegan (1938) showed that for a two-dimensional rough turbulent flow (i.e. When R_e^*>70), $u/u*$ may vary slightly with the shape of the canal between 6 (wide open or rectangular channels) and 6.25 (trapezoidal open channel).

Cao (1985) proposed a semi-logarithmic model by as well as the exponential model for uniform flow conditions.

Graf et al. (1987) proposed shear stress and regression approaches for determining the bed load transport rate respectively. The method predicts best Φ values for the optimum values of R^2, σ, slope, E and I_d.

Recking (2006) approached a simple method for calculating friction factor with respect to the relative depth for different region for the gravel bed flume.

2.4. Intensity of Bed load

There are different approaches has been formulated by various researchers for bed load transport in terms of shear stress, Energy slope, Discharge, Probabilistic, Regression Equal-

Mobility etc. Many textbooks provide a good overview about sediment transport processes and formulae, e. g. Graf (1971), Yalin (1972), Graf (1984), Raudkivi (1990), Dittrich (1998), Graf and Altinakar (2000), Scheuerlein and Schöberl (2001) and Yalin and da Silva (2001). In many experimental investigations the bed material used has been rather uniform. For this reason, a top layer effects are not treated explicitly.

Since bed-load transport is related to drag or traction (Graf 1971), the first important contribution of the drag principle was advanced by Du Boys (1879). Investigating the Rhone River in France, stream flow depth and slope have been used to develop a quantitative bed-load formula. Important investigations concerning incipient motion have been conducted by Hjulström (1935). The beginnings of the movement of sediments has been related to the grain diameter and mean flow velocity which was presumed to be about 40 % greater than the bottom velocity for a flow depth exceeding 1.0 m.

Shields (1936) performed experimental investigation in the laboratory with uniform grain size bed material. Based on these experiments and theoretical considerations, a new method has been proposed incipient motion criterion using the grain size Reynolds number and a dimensionless shear stress parameter (Shields factor). Explicit formulations for the entire Shields diagram are given by Van Rijn (1984a), Yalin and da Silva (2001) and Cao et al. (2006).

Schoklitsch (1930, 1950) study includes both uniform sands and sand mixtures and proposed that the rate of sediment movement is proportional to the excess power. He developed an expression for the critical flow rate which depends on the relative sediment density, grain diameter, and energy slope.

Meyer-Peter and Müller (1948) performed a large number of tests having bottom slopes between 0.04 % and 2.3 % and established the well-known Meyer-Peter and Müller (MPM) formula. The sediments used were characterized by a both uniform and wide grain size distribution.

Ackers and White (1973) use the discharge, flow velocity and flow depth and sediment characteristics as main input parameters for developing a bed-load transport formula.

Smart and Jäggi (1983) carry out the work of Meyer-Peter and Müller (1948) for channels with steeper slopes up to 20 % and provides the extended MPM-formula. He incorporated the influence of a wider grain size distribution in the formula and suggests that the value of the critical Shields parameter varies with the grain Reynolds number instead of being kept constant. Currently, the Smart and Jäggi (1983) formula has been most widely used bed-load transport formulae in Switzerland.

Van Rijn (1984a) follows the approach of Bagnold (1966) and assumed that the motion of bed-load particles being dominated by gravity forces while the effect of turbulence on the overall trajectory is supposed to be of minor importance. The bed-load transport formula has been proposed using the dimensionless particle diameter (d^*) and a transport stage parameter (T). The bed-load transport is the product of the saltation height (δ_b), the particle velocity (u_b) and the bed-load concentration (c_b).

Hunziker (1995) introduced a modified bed-load formula for wide sediment mixtures based on the one of Meyer-Peter and Müller (1948). Based on dimensional analysis with empirically determined exponents, the fraction wise sediment transport has been introduced.

Wilcock et al. (2001) study is focused on determining the sediment transport rate for gravel beds. He uses an equal mobility method for estimating the sediment transport rate.

Wilcock-Crowe (2003) proposed a bed load transport model for mixed sand and gravel beds. His study shows that the transport rate depends on the size of bed materials. Thus the proposed model is simple such that flow was remained in transient conditions.

Camenen et al (2006) approached a new relationship between the height of the bed materials and the main hydrodynamic and sediment parameters for gravel beds under steady flow conditions.

Wong and Parker (2006-a) described that sediment transport in rivers or open channel flow, in the case of gravel-bed streams, is developing more accurate the bed-load transport rate.

Castillo et. al. (2013) proposed the methodology of Computational Fluid Dynamics (CFD), which is based on numerical solution of the Reynolds Averaged Navier-Stokes (RANS) equations together with turbulence models of different degrees of complexity, simulates the interaction between different fluids, such as the sediment transport and the air-water two-phase flow that appear in the phenomenon of intake systems.

Recking (2013) study was focused on rivers and open channel flow having sand bed and gravel bed. He proposed a simple method for calculating the reach-averaged bed-load transport for both sand bed and gravel bed.

Bareš et. al. (2014) examined the horizontal velocity component in open-channel flow with intense transport of coarse sediment of given grain size (other experiments with different grain sizes are planned). Measurement of the velocity distribution is performed using three independent methods (Prandtl tube, Ultrasonic Velocity Profiler, Acoustic Doppler Velocity Profiler). The aim of the experiments is to evaluate and compare the different methods for their potential to be used further in the project, which focuses to intense sediment transport phenomena

Castillo et. al. (2014) focused on the study of bottom rack intake systems for discontinuous and torrential streams. The cases of clear water and water with gravel sediments have been analyzed. Different tests have been carried out to quantify the influence of the solids passing through the racks.

2.5. Critical Review

Darcy-Weisbach formulated a relation of friction factor without considering *R/D* ratio similarly Keulegan (1938) proposed a new formula for the same by using *R/D* (undefined).Simons-Richardson (1966) followed the work of Kulegan (1938) for gravel bed and categorised it into three bedload condition i.e. no load, moderate and intense. Recking (2006) tested the Keulegan (1938) formula and concluded that the particular equation of Keulegan was for R/D>8.6.In the proceeding, he also determined new empirical relation where *R/D* was less than 8.6 for no load condition. Similarly for intense load condition R/D<16.9 has been considered and Recking (2006) proposed a model to determine the friction factor. Review of the various research works by the past, researchers reveal that there is a need to evaluate the hydraulic parameters like the different hydraulic resistances and velocity profiles of gravel bed open channels under both load flow conditions. Eienstien (1950) has proposed a model for intensity of bed load, Φ under intense conditions which depends upon the Shields parameter, θ. If it is greater than 0.1 it is called as intense load condition. Taken this assumption many researchers have proposed different model to determine the intensity of bed load for the intense load condition.

CHAPTER 3

EXPERIMENTAL PROGRAM

EXPERIMENTAL PROGRAM

3.1 General

Experiments were conducted under controlled laboratory conditions in the Fluid Mechanics Laboratory of the Civil Engineering Department at the National Institute of Technology, Rourkela, India in order to find out the impact of friction factor, Velocity distribution, Bed load intensity and Boundary shear stress on various hydraulic characteristics of open channel flow. The various hydraulic characteristics of flow studied were hydraulic resistances including relative depth, *R/D* Shields Parameter, θ Manning's roughness coefficient, *n* and velocity distribution of flow under uniform conditions. Experiments were also conducted to find out Velocity, *u* Boundary shear stress, τ Darcy-Weisbach's friction factor, *f* and bed load sediment transport rate, q_b at different discharges and flow depths and correlate the various hydraulic parameters to develop models to predict Friction Factor, *f* and Bed load intensity, Φ in a trapezoidal gravel bed channel flow under uniform flow conditions. This chapter describes the experimental channel design, construction of roughness components and measurement techniques of experimental data, including the velocity of flow and development of stage-discharge relationship and the boundary shear stress of the hydraulic flume. Details of the various instruments used to measure the experimental data are also discussed in this chapter.

3.2 Experimental Channel Design

3.2.1. Tilting Flume

For the present study a straight simple trapezoidal channel in the form of a tilting flume having dimensions of length 10 m, top width 0.9 m, bottom width 0.65 m and depth of 0.125 m is used. The tilting flume is made of metal frame with glass walls at the test reach. At the starting of the flume after inlet and before the head gate a baffle wall are fitted for indulgence of energy because of reducing the turbulence and make a uniform flow throughout the

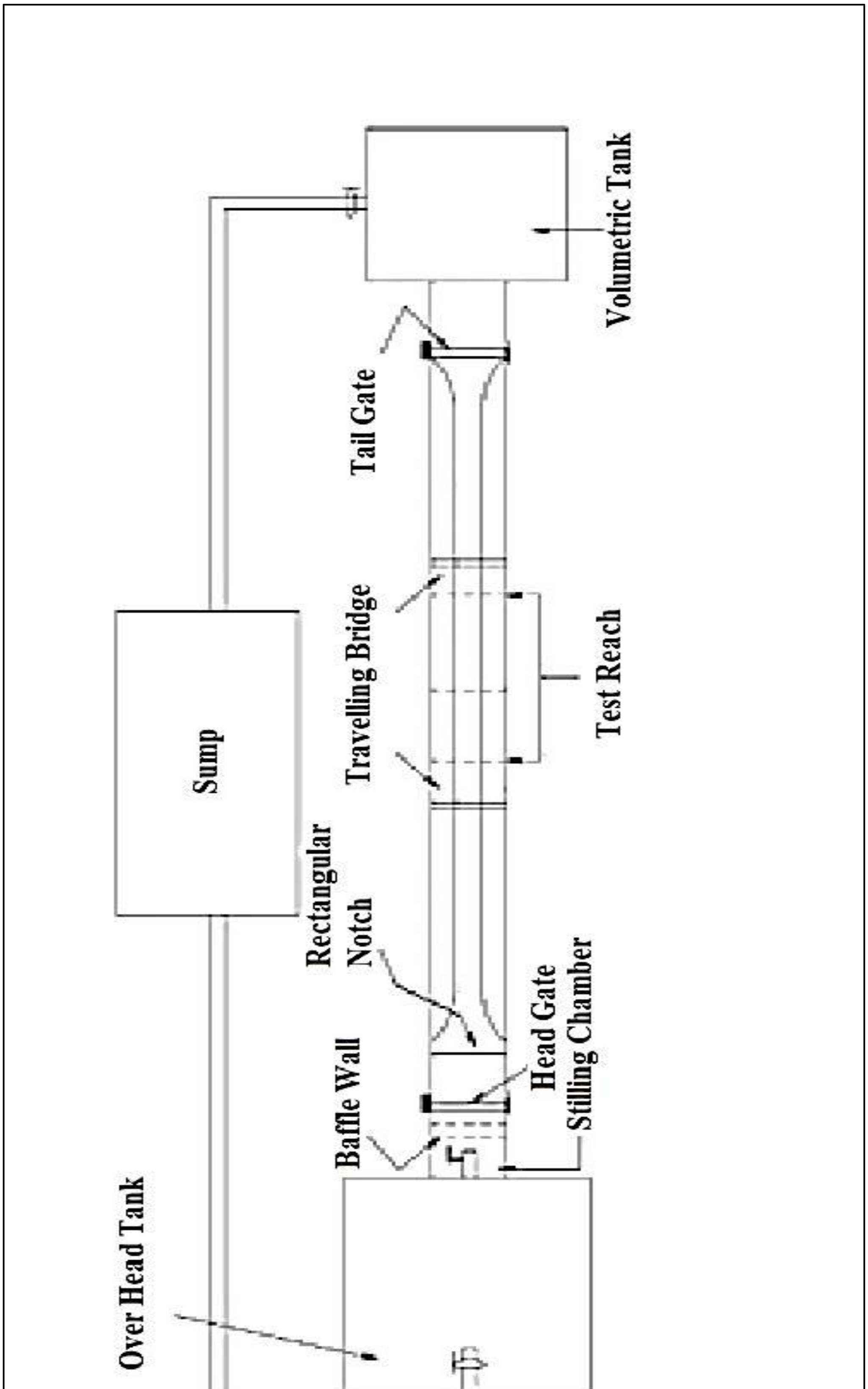

Fig. 3.1 Plan view of the experimental channel

channel section. Headgate reduces the waves if formed in the water body before it passes over the channel and in this way head-gate plays a vital role in maintaining uniform flow. For measuring the bed slope tailgate was fitted at the end point of the flume. There was provision of an over bridge platform in the flume which helps in experimental works. The flume was supported on a hinge at the centre and made tilting by providing hydraulic jack arrangement at starting point of the flume. The plan view of the experimental channel used in the present study is shown in Fig. 3.1. The overall view of the flume with experimental setup is shown in Fig. 3.2.

3.2.2. Experimental Channel

Experiments are conducted in a straight simple trapezoidal channel with uniform cross section built inside a metallic flume. The dimensions of the channel section are of length 10 m, top width 0.9 m, bottom width 0.65 m and depth of 0.0125 m. Experiments are conducted for gravel bed no load and intense conditions. The whole channel is fabricated by using gravel of13.5 mm diameter for no load and 6.5 mm for the intense load condition in the bed. The roughness height is found to be 2.5 cm. The slope of the flume is fixed at 0.0025 (0.25%) for all runs. The geometrical parameters of the experimental channel are mentioned in Table 3.1

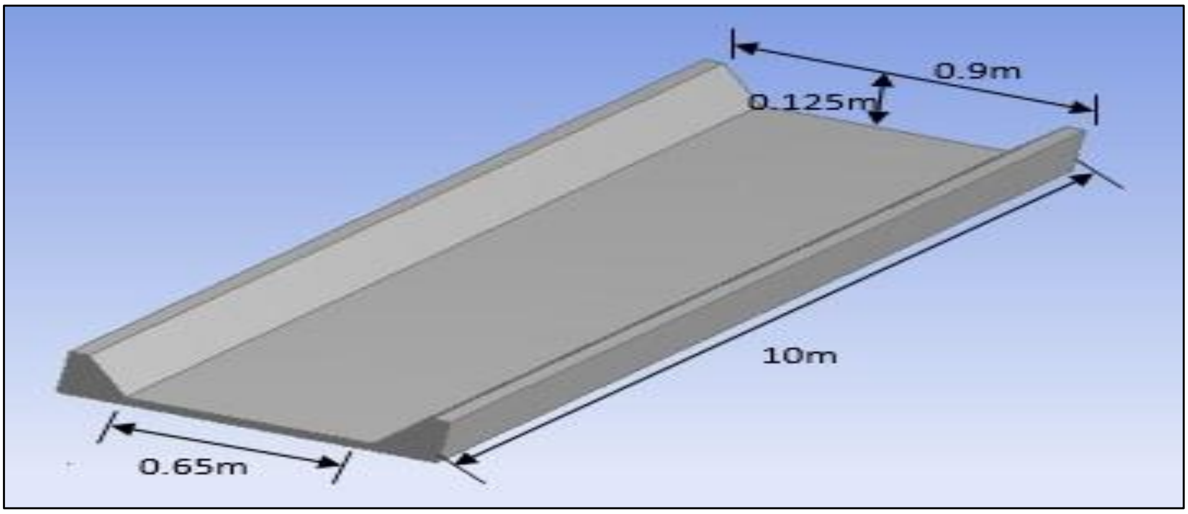

Fig. 3.2 Overall view of the flume with experimental set up

Table 3.1 Geometrical parameters of the experimental channel section

Sl no.	**Item description**	**Experimental channel**
1	Channel type	Straight
2	Geometry of channel section	Trapezoidal
3	Channel base width (B)	0.65 m
4	Top width of channel (B)	0.9 m
5	Depth of channel (H)	0.0125 m
6	Bed slope of the channel (S)	0.25%
7	Length of whole channel	10 m
8	Test Reach Length	6 m
9	Nature of surface of bed	Rough, Gravel bed

3.2.3. Water Supply System

Water for the experiment is provided from an overhead tank to which a water level indicator is attached to show the maintaining of constant water level in the overhead tank. Two parallel pumps are installed for pumping water from an underground sump to the overhead tank. Water delivers to the stilling chamber from the overhead tank, passing over the experimental channel under gravity and is made to fall into the volumetric tank situated at the end of the flume. From the volumetric tank, water is allowed to flow into an underground sump. The water is recirculated back from the sump.

3.3 Details of Roughness Components

In the present study, a trapezoidal concrete channel constructed throughout the flume. Dry gravel of diameter 13.5 mm for flow without sediment transport and 6.5 mm for flow with sediment transport are used for making bottom as a rough surface. Height of the roughness is 2.5 cm. A test reaches of length 6 m from the starting of the tilting hydraulic flume has chosen. Detail geometrical features of the roughness elements used in the present experiment are mentioned in Table 3.2. In this present study, D_{50} size has been used. Sieve analysis has been done for both the gravel sizes. After doing the sieve analysis density of the sediment has measured. The density of the sediment for both 13.5 mm and 6.5 mm gravel size is 1520 kg/m^3 and 1220 kg/m^3 respectively.

Table 3.2. Detailed geometrical features for both the load conditions

Sl no.	Bed load characteristics	Description
1	Material	Gravel
2	Diameter of gravel (d)	13.5 mm and 6.5 mm
3	Sediment density (ρ_s)	1520 kg/m^3 for 13.5 mm 1220 kg/m^3 for 6.5 mm
4	Height of roughness	0.025 m
5	Distribution pattern	Normal
6	Test reach	6 m

3.4. Experimental Procedure

3.4.1 Apparatus and Methodology

Main parameters to be measured during the present experiment are discharge, bed slope, depth of flow and the velocity of flow and boundary shear stress. The measurement procedure of these parameters is briefly described as follows. Depth of flow in the channel is measured by using a point gauge fixed into the travelling bridge and operated manually. Point velocities are measured using a Micro-Pitot tube of 4.77 mm external diameter with suitable inclined manometer at a number of locations across the predefined channel section. Guide rails are provided at the top of the experimental flume on which a travelling bridge is moved in the longitudinal direction of the entire experimental channel. The point gauge attached to the travelling bridge can also move in both longitudinal as well as in the transverse direction of the experimental channel. The Micro - Pitot tube is also attached to the bridge on the other side of the point gauge. The Pitot tube is physically rotated normal to the mainstream direction till it gives a maximum deflection of manometer reading.

Fig. 3.3 Photographs of experimental procedure (a) Gravel bed (b) Manometer fitted with scale (c) Head gate (d) Tail gate (e) Piezometer (f) Pitot tube and Point gauge fitted in travelling bridge arrangements

Discharge in the channel is measured by the time rise method. The water flowing out at the downstream end of the experimental channel leads to a volumetric tank of area 20.866 m^2.

The change in the depth of the water with time is measured by stopwatch in a glass tube indicator with a scale having least count of 0.01 mm.

A hand-operated tailgate weir is constructed at the downstream end of the channel to regulate and maintain the desired depth of flow in the flume. The bed slope is set by adjusting the whole structure, tilting it upwards or downwards with the help of a lever, which is termed as a slope changing leever. Details of measurement techniques of different parameters are described as below.

3.4.2. Calculation of Bed Slope

To find out the channel bed slope, the tailgate of the flume was closed. The channel was pounded with water. The depth of water at two end points along the centreline of experimental range was measured with the help of a manually handled point. The point gauge was attached to the travelling bridge for which the point gauge was able to move in transverse as well as in longitudinal direction of the channel. The slope of the bed was measured by separating the difference in water depth at both ends of the experimental reach. With the help of this measurement, the bed slope was found 0.0025(0.25%) and was kept constant for all runs under uniform conditions.

3.4.3. Discharge Measurement

A volumetric tank located at the end of channel receives water flowing through the channels. Depending on the flow rate, the time of collection of water in the volumetric tank varied; the lower one for higher rate of discharge. The time is recorded using a stopwatch. Change in the mean water level in the tank over the time interval is recorded. From the knowledge of the volume of water collected in the measuring tank and the corresponding time of collection, the discharge flowing in the experimental channel for each run is obtained. Simultaneously, the depth of water flowing in the channel is also measured by the point gauge. This depth of flow is termed as a gauge or stage. In the next step, the depth of water in the channel is changed and for this new depth of flow/stage/gauge, the discharge flowing over the channel section is measured as described above. In this way, a set of data of stage and discharge is collected and then the data are used to develop a stage-discharge relationship. This stage-discharge relationship is helpful in the computation of discharge in the channel at different flow depths for the present study.

3.4.4. Measurement of Velocity using the Micro - Pitot tube

A micro-pitot tube of external diameter 4.77 mm in conjunction with a suitable inclined manometer was used to measure velocity. The Pitot tube is fixed to a main scale having vernier scale with least count of 0.1 mm. The main whole of the Pitot tube is faced in the flow direction to give total pressure while the surface holes of the Pitot tube give static pressure. Both the pressures are seen as heights of water in two limbs of inclined manometer. The difference in water elevation gives the velocity at the particular point (u) where the Pitot tube was mounted and also the pressure difference (Δp) by using the following Bernoulli equations (White (1999).

$$u = \sqrt{2g\Delta h \sin\alpha} \tag{3.1}$$

$$\Delta p = \rho g \Delta h \sin\alpha \tag{3.2}$$

Where g is the gravitational force, ρ is the density of water, $\Delta h$is the difference in water elevation in the manometer, and α is the angle of manometer with horizontal base.

While taking velocity readings using Pitot tube, the tube is placed facing the direction of flow and then is rotated along a plane parallel to the bed, till it registers relatively a maximum head difference in the attached manometer. The total head h read by the Pitot tube at the location in the channel is used to give the magnitude of the total velocity vector as

$$u = \sqrt{2gh \sin\alpha} \tag{3.3}$$

Where g is the acceleration due to gravity. While doing so, the tube coefficient is taken as a unit and the error due to turbulence in the computation of u is considered negligible. Using the data of velocities measured by Pitot tube the longitudinal velocity profiles of both no load and intense load cases are studied and the velocity data are used to evaluate the various hydraulic characteristics of the gravel bed open channel.

In the present study, all measurements were carried out under uniform flow condition by regulating the outflow through downstream tailgate. Experiments were conducted under no load condition. For no load condition, total seventeen depths taken for stage-discharge purposes and five depths out of seventeen has been taken for velocity distribution and boundary shear stress under no load flow condition. Meanwhile, for intense load condition total, eighteen depths are taken for the stage-discharge purposes. Fig. 3.4and Fig. 3.5 shows gravel size, which was used for both the condition.

Fig. 3.4gravel bed channel D_{50} 13.5 mm gravel

Fig. 3.5 gravel bed channel D_{50} 6.5 mm gravel

3.4.5 Evaluation of boundary shear stress

As boundary shear stress represents the local force by the fluid on a surface and is the main reason of sediment transport, erosion it has a great importance in hydraulic research. There are several methods used to evaluate boundary shear stress in an open channel. The Preston tube technique and Energy gradient method has been popularly used in laboratory experiments as well as in field survey where the channel surface was either smooth or rough, Ackerman et al. (1994), Atabey's (2001) Birmingham data. In this study for 13.5 mm gravel size Preston tube technique has been used and for 6.5 mm gravel size energy gradient method has been used.

3.4.5.1. Preston tube technique

In this present study for the 13.5 mm gravel size, this technique has been used to determine the boundary shear stress. As the theory of an inner law concerning the local shear to the velocity distribution close the wall, Preston (1954), established a technique for determining local shear (τ) in a turbulent boundary layer with the help of a pitot (Preston) tube. The tube is retained in interaction with the bottom surface. Calculation of the wall velocity distribution is empirically depending upon the differential pressure (Δp) between total and static pressure at the wall. The difficulty of this method is evaluating the proper calibration equation or curve for the given tube diameter. A non-dimensional relationship between differential pressure (Δp) and local shear suggested by Preston (τ) as:

$$\frac{\Delta p}{\rho}\frac{d^2}{v^2} = F\left[\frac{d^2\tau}{\rho v^2}\right] \qquad (3.4)$$

Where v is the kinematic viscosity of fluid, ρ is the density of the fluid and d is the diameter of Preston tube and functional relationship F needs to be calculated. Preston suggested the calibration equation

$$x^* = log_{10}\left(\frac{\Delta p d^2}{4\rho v^2}\right) \qquad (3.5)$$

$$y^* = log_{10}\left(\frac{\tau\, d^2}{4\rho v^2}\right) \qquad (3.6)$$

Patel (1965), recommended a relationship for F in Eq. (11) valid in three regimes (y^* between 1.5 -5.5).

$$y^* = 0.5x^* + 0.037 \qquad \text{for } 0 < y^* < 1.5 \qquad (3.7)$$

$$y^* = 0.8287 - 0.1381x^* + 0.1437x^{*2} - 0.0060x^{*3} \qquad \text{for} 1.5 < y^* < 3.5 \qquad (3.8)$$

$$x^* = y^* + 2\log_{10}(1.95y^* + 4.10) \text{ for } 3.5 < y^* < 5.3 \quad (3.9)$$

3.4.5.2. Energy gradient method

For a prismatic channel under uniform flow conditions, the boundary shear forces acting along the wetted perimeter must be equivalent to the weight force alongside the direction of current. Supposing the mean boundary shear stress (τ) to be fixed over the boundary of the channel. Boundary shear stress can be expressed (τ) as:

$$\tau = \rho g R S \quad (3.10)$$

Where g = gravitational acceleration, ρ= density of flowing fluid, S = slope of the energy line, *or*= hydraulic radius of the channel cross section *(A/P), A* = area of the channel cross section, and P = wetted perimeter of the channel cross section. It is known as energy gradient method. This method is not appropriate for local, small-scale deviations in the shear stress due to the larger length necessity for the evaluation of the energy slope. Moreover, measurement of energy slope is not accurate which normally affect the method. Most of the circumstances, it is expected that energy slope is as similar as to the bed slope which is essentially not correct for all flow conditions.

3.4.6 Bedload sediment transport rate measurement

A portable rectangular transparent container fitted at the end of the flume for collecting the bed load. Height of the container is 30 cm, width is 90 cm and length is 30 cm, and it is placed at the end of flume. When the container gets overflowed it has a gate to remove the particles from it. A stopwatch has provided to compute the time till the end of experiment for determining the sediment transport rate. It can be possible at the middle of the experiment the container get filled up at that time gates will use to remove the bed particles and placed it into a safe place. At this particular time,stopwatch will remain pause. After the end of the experiment volume of the sediment has been calculated and its divide with the time and we got the sediment transport rate.

CHAPTER4

THEORETICAL CONSIDERATIONS

THEORETICAL CONSIDERATIONS

4.1. General

The flow resistance models shows the relation between the linear energy losses to the mean flow velocity, out of which three have been widely used namely Manning, Chezy and Darcy-Weisbach's equations. All these models are semi-empirical and validations of these are done on experimental or field data. Consequently, literature had already proposed a wide range of calibration coefficients.

It was observed that bed load has a significant relation over the large increment in resistance in terms of friction factor *f*. Observations suggest that to properly model flow resistance in a gravel bed channel for a wide range of slopes and relative depths, it is required to identify three flow regimes: no, low and high sediment transport.

In this present study, two regimes no load and high load are considered and thus for noload condition as well as high sediment transport flows are characterized by a resistance coefficient *f* that decreases as the relative depth decreases.

We have seen that for a uniform flow in open channel systems, the friction factor *f* can be expressed by Darcy-Weisbach relation given below

$$\sqrt{\frac{8}{f}} = \frac{u}{\sqrt{gRS}} = \frac{u}{u*} \tag{4.1}$$

Where,

$u*$ is Shear velocity, u *is* Measured velocity by manometer, R is Hydraulic radius, g is Acceleration due to gravity, f is Darcy-Weisbach's friction factor and S is Channel bed slope.

For rough flows, the friction coefficient f is not related on the basis of Reynolds number and a linear relation is usually expressed between $(8/f)^{1/2}$ and corresponding log(R/D) values. The friction factor f is reported in the Darcy form of $(8/f)^{1/2}$.

With the help of the experimental data of this present work, a simple graphical representation have been made between *(8/f)* $^{1/2}$and relative depth, *R/D*. Fig. 4.1 shows variation of friction factor, *f* for both no load and intense load conditions.

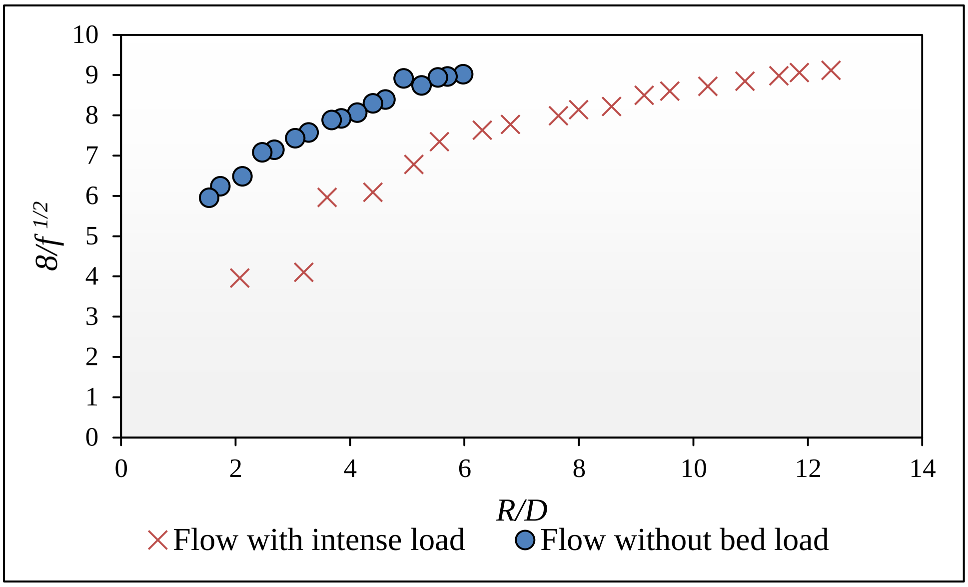

Fig. 4.1 Variance of friction factor for both bed load conditions

We observed that bed load was significantly responsible for a large increase in friction factor *f*. Observations also suggested that for correct model of flow resistance for a wide range of slopes and relative depths, it is necessary to distinguish three flow regimes i.e. no, low and high sediment transport.

Whereas high sediment transport flows and clear water were characterized by a resistance coefficient *f* that decreases with the relative depth. Measurements showed that there was an intermediate regime (low sediment transport) which is characterized by a constant resistance coefficient, for any relative depth *R/D*. Where *R* is the hydraulic radius and *D* is the diameter of gravels.

Three regimes are easily distinguished:

-Regime 1: there is no sediment transport and *(8/f)* $^{1/2}$increases with the relative depth *R/D*;

-Regime 2: a bed load appears and increases with the relative depth; the friction factor is constant;

-Regime 3: both bed load and *(8/f)* $^{1/2}$ increases with the relative depth *R/D*.

In this present study model of Regime 1 and Regime 3 has been developed by using experimental data sets. Fig. 4.2 shows relation between the measured bed load intensity to dimensionless shear stress. In regime 1 shows no load conditions, so load intensity is tend to zero. In case of regime 3 it's clearly visible that load intensity increases as dimensionless shear stress.

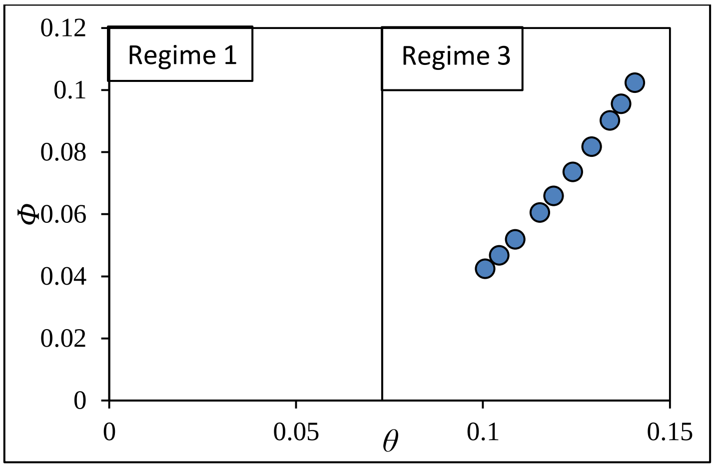

Fig. 4.2 Measured bed load intensity in relation to dimensionless shear stress

4.2. Empirical formulation for Friction Factor with both no load and intense load condition

4.2.1. Friction factor by Keulegan (1938)

For a two-dimensional turbulent flow for the condition of Roughness Reynolds number, R_e^* is greater than 70, Keulegan (1938) had expressed u/u^* by integrating the formerly given logarithmic velocity distribution by Karman-Prandtl in Eq. 4.2.

$$\frac{u(z)}{u*} = \frac{1}{k} ln \frac{z}{z_0} \quad (4.2)$$

Where u is channel velocity, u^* is the shear velocity, k is Von Karman coefficient, z is the height above the bed and z_0 is the height where velocity is zero according to the law of the wall.

A good approximation has been obtained by taking over entire flow depth assuming logarithmic profile (Cardoso et al. 1989, Song et al. 1995, Nikora and Smart 1997, Smart 1999) and that $R >> z_o$ (where velocity tends to zero), Smart et al. (2002) reported that until the relative depth *R/D* was higher than 1, $(8/f)^{1/2}$ can be expressed as in Eq. 4.3 (for k=0.4):

$$\sqrt{\frac{8}{f}} = \frac{U}{u^*} = \frac{1}{k}\left[\left(\frac{R}{R-z_o}\right) ln\left(\frac{R}{z_o}\right) - 1\right] \approx 5.75\, log\frac{0.368R}{z_o} \quad (4.3)$$

Nikuradse (1933) first suggested calculating z_o as a fraction of the bed roughness, K_s by taking z_o=*D*/30. Using this classification, $(8/f)^{1/2}$ can be written as:

$$\sqrt{\frac{8}{f}} = E + 5.75\, log\left(\frac{R}{D}\right) \quad (4.4)$$

E is a constant which is depend upon the channel cross sections. Keulegan (1938) presented that *E* may marginally differ with the natures of the canal between 6 for wide rectangular channels and 6.25 for trapezoidal open channel.

4.2.2 Friction factor by Cao (1985)

Cao (1985) observed that in a 0.6-m wide flume a 10% of error matches to minimum *R/D* values of 2, 2.5 and 9 for 44 mm, 22 mm and 11 mm grain diameters, respectively. There had been only a few attempts made, essentially on steep slopes, to amend flow resistance laws by adding slope as a parameter. Moreover, this was usually done by a general curve fitting processes without any physical consideration of the effects of slope, even if a discussion regarding slope-induced additional resistance through surface instabilities and waves at high Froude numbers still exists. From this point of view, introducing a parameter like slope which is responsible for the incipient motion of sediments is a new approach. The semi-logarithmic model proposed by Cao (1985) as

$$\sqrt{\frac{8}{f}} = 3.75 + 5.91\, log\left(\frac{R}{D}\right) \quad (4.5)$$

4.2.3 Friction factor by Manning-Strickler (1923)

The Manning-Strickler equation given is:

$$u = K_s R^{\frac{2}{3}} S_0^{\frac{1}{2}} \tag{4.6}$$

Where K_s is the grain resistance Manning-Strickler coefficient. From the definition of K_s which is $K_s = \frac{21.1}{D^{1/6}}$

(4.7)

It becomes:

$$\sqrt{\frac{8}{f}} = 6.74 \left(\frac{R}{D}\right)^{\frac{1}{6}} \tag{4.8}$$

4.2.4. Friction factor for relative depth (2<R/D<8.6) by Recking (2006)

Recking (2006) observed that for 2<*R/D*<8.6, the resistance equation could not be fully derivative for flows without sediment transport from the law of the wall. In its place an empirical law was suggested, essentially affecting the slope of the logarithmic law (9.5). For significantly higher relative depths, the Keulegan law was effective for flows without sediment transport which fall within the small-scale roughness series. Thus, the bed roughness was nearly taken as the grain diameter. The semi-logarithmic model proposed by Recking (2006) as

$$\sqrt{\frac{8}{f}} = 2.5 + 9.5\, log \left(\frac{R}{D}\right) \tag{4.9}$$

4.2.5. Friction factor for relative depth (R/D<17) by Recking (2006)

Recking (2006) saw that when flow increases, the value of friction factor, *f* decreases with respect to relative depth *R/D*, whatever the slope will be. He took all high sediment transport rate data set of 32 values for flow conditions were selected and known as sheet flow regime, for high relative depth. In his study first, it can be observed that all data chosen as high sediment rate regime were lined up with the sheet flow regime data, which he suggested that regime 3 and which was similar as sheet flow regime. The model concerned with the intermediate-scale roughness (*R/D*<17) and must be modelled with the slope coefficient of 9.5.The semi-logarithmic model proposed by Recking (2006) as

$$\sqrt{\frac{8}{f}} = -1 + 9.5\, log\left(\frac{R}{D}\right) \tag{4.10}$$

4.2.6 Friction factor by Julien (2002)

Julien (2002) proposed a model for bed load sediment transport rate, q_b> 0.1 and which was falling under regime 3,

$$\sqrt{\frac{8}{f}} = 5.75 \log\left(\frac{2R}{D}\right) \tag{4.11}$$

4.3. Bed load Intensity with Intense load condition

There were two common methods were there towards the theory of bed load transport. First and most popular method was the usage of the variables such as stream power, shear stress, discharge, and velocity. In this method basic assumption was that until or unless the critical variable exceeds the flow conditions there was no bed load transport occurred and by exceeding the critical variable of the flow condition the bed load transport rate increases too. There were several studies based on this concept, both for natural channels and for flumes. A numbers of transport rate models are presented there in this section. Dependability for the usage of these equations was on the assumptions of the existing models. The second method was probabilistic method (*Einstein*, 1950). A new perception to this method had been offered into bed load transport processes. However, the level of complexity made by this application to natural channels were intricate and challenging (*Yang*, 1996).

The basic assumption made was sediment particles on the bed start to transport when incipient motion happened. The tractive force or bed shear stress acting on the bed was responsible factor for the transport of bed particles in a stream. *Shields* (1936) first gave two major parameter to determine the bed load intensity which is θ and θ_c .The movement of bed particles indicate that the Shield's parameter θ was exceeds the critical value it was called θ_c (for Regime 2), which is:

$$\theta = \frac{\tau}{(\gamma_s - \gamma)D} \tag{4.12}$$

$$\theta_c = \frac{\tau_c}{(\gamma_s - \gamma)D} \tag{4.13}$$

Where τbed shear stress, *D* Particle diameter and $\tau_c$critical bed shear stress.

$$\theta_c = \frac{0.273}{1+1.2d*} + 0.046(1 - 0.576e^{-0.05d*}) \tag{4.14}$$

Where $d_*$is

$$d_* = D\left[\frac{s-1}{\nu^2}\right]^{.33} \tag{4.15}$$

Shields (1936) was the inventor, he described the critical shear stress was depend upon the different particles sizes, *D* and it can be in various or uniform shaped, tends on the threshold of motion by a uniform flow and expressed as:

$$\frac{q_b \gamma s}{q \gamma S} = 10\ \frac{\tau_0 - \tau_c}{(\gamma_s - \gamma_0)D} \tag{4.16}$$

The solid discharge was expressed in a non-dimensional form (Einstein 1942) defined by:

$$\Phi = \frac{q_b}{\sqrt{\left(\frac{\rho_s}{\rho} - 1\right) g D^3}} \tag{4.17}$$

As per literature review, there are many researchers gives different equation considering different factors for bed load intensity. The bed load intensity equation are summarised in table 4.1. In this table shows the different mathematical model of different researchers to calculate the bed load intensity for intense load condition which is related to present study.

Table 4.1 Different approaches for determining Bed load Intensity for Intense load condition used in the present work

Sl. No	Approaches	Developed Equations
1.	Ashmore (1988)	$\Phi = 3.11(\theta - 0.045)^{1.37}$
2.	Wong and Parker(2006a)	$\Phi = 4.93(\theta - 0.047)^{1.6}$
3.	Wong and Parker (2006b)	$\Phi = 3.97(\theta - 0.0495)^{3/2}$
4.	Graf and Suszka (1987)	$\Phi = 10.4\ \theta^{2.5} \quad when\ \theta \geq 0.068$
5.	Recking (2006)	$\Phi = 14\ \theta^{2.45} when\ 0.08 < \theta < 0.25$

4.4. Application of Hydraulic Software for computation of Friction Factor and Bed load Intensity

4.4.1. Conveyance Estimation System, CES

In the field of hydraulics researchers while working on simple channels, in particular, have often taken recourse to a three-pronged strategy. In addition to the physical experiments on laboratory flumes by mimicking river flows or natural flows within the man-made environment and the analysis of fluid dynamics governing the flow in any natural or artificial channels theoretically, a final approach namely the 'Computational Fluid Dynamics' (CFD) has lately been developed in 1960's and pursued in the field of hydraulic research with the advent of modern high-speed digital computers. As per Anderson, Jr. (1995), CFD does not replace experimental or theoretical studies, rather it nicely and synergistically complements the other two approaches. Physical representations of any fluid dynamics problem are either in differential form or in integral form of complex mathematics. So CFD is essence of nothing but a numerical tool that applies to solving differential and partial differential equations of fluid dynamics problem on high-speed computers through various algorithms. In last quarter century, development in the field of computers and hardware have taken place and thus improving the application of CFD to environmental flows resulting in emergence of a number of research codes as well as commercial packages for ready to use by different users. The present research concerning straight trapezoidal channels with gravel bed had adopted various applications of numerical tools to the problems as a complementary study to the experimental research. In this study, CES has been used to compute the boundary shear stress without changing the properties of present research work.

Fig 4.3 illustrates the three-pronged strategy of model studying.

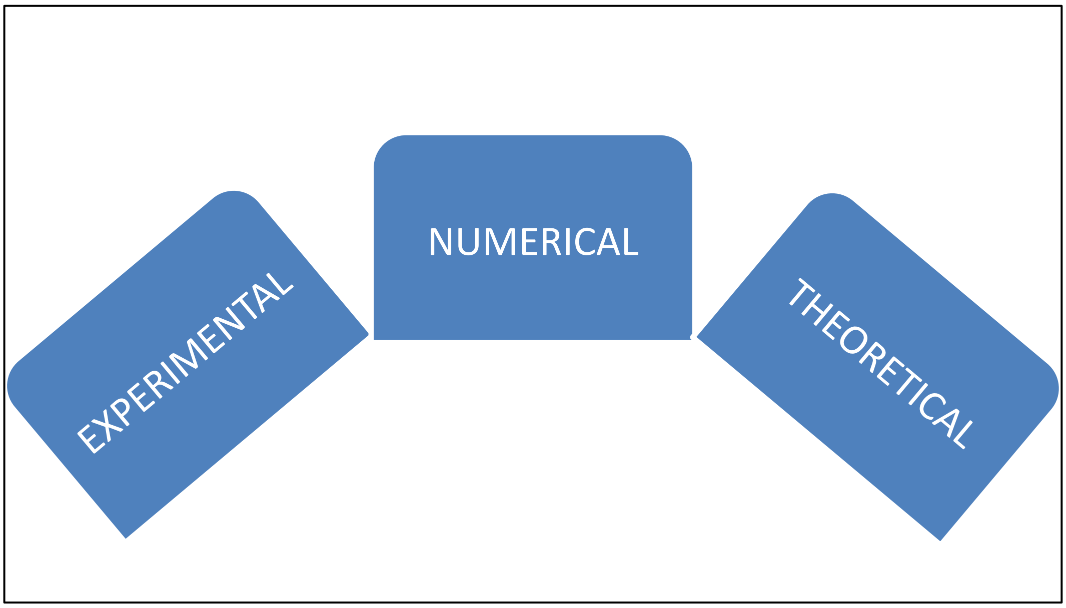

Fig. 4.3 Three Dimensions of Research in River Hydraulics

Basically, CES is based on the depthintegration of the RANS equations for flow in the streamwise direction. The basic form of the depth-averaged momentum equation for application to channel flow is (Shiono & Knight, 1988):

$$\underbrace{\rho g H \frac{dh}{dx}}_{(\mathrm{I})} \underbrace{- \beta' \tau_b}_{(\mathrm{II})} + \underbrace{\frac{\partial}{\partial y}\left(H \bar{\tau}_{yx}\right)}_{(\mathrm{III})} = \underbrace{\frac{\partial}{\partial y}\left[H\left(\rho \overline{UV}\right)_d\right]}_{(\mathrm{IV})} \qquad (4.18)$$

Where, x is stream wise direction parallel to the bed (m), y = lateral distance across section (m), U_d = depth-averaged stream wise velocity (m/s), V_d = depth-averaged lateral velocity (m/s), τ_{yx} = Reynolds stress (N/m^2), τ_b = bed shear stress (N/ m^2), β' = coefficient for the influence of lateral bed slope on the bed shear stress.

4.4.1.1. Outline of Steps for Modelling through CES

The user has to use the steps as outlined below to run the software tool to obtain the results of simulation through CES. First, the roughness file named *.RAD File has to be created for the physical domain where the flow has to be simulated. For this, the user needs to choose various roughness components comprising of vegetation, ground material and irregularity for all the three zones of the channel namely ;bed, bank by selecting from the catalogue available for various morpho types of vegetation, substrates for ground material and irregularity types inside the component 'Roughness Advisor' of CES.

- At this stage, if there is some doubt about the actual value of roughness of the real vegetation, irregularity & substrates etc. the user can assign the lower and upper values for the assigned value so that CES accordingly computes the uncertainty band for the result outputs.

- After saving the RAD file, the Conveyance generator component needs to be activated for creating *.GEN file for where all general data for the physical domain such as name of reach, sinuosity, cross section details measurement (through lateral offsets and heights of various points on the cross section from bed to top of water surface) etc. are to be entered.

- Then all zones of the reach e.g. bed, bank need to be assigned the roughness values as assigned previously through RAD file.

- By exercising the options available in advanced options tab in Conveyance Generator for various parameters e.g. no. of depth intervals, minimum depth used in calculation, value of lateral eddy viscosity in the main channel, no. of vertical segments used in computation, relaxation parameter for convergence criteria, maximum no. of iterations and wall height multiplier etc. the user can vary the results of simulation so as to get the best possible outcome. Also, there is a separate option of adopting Colebrook-White solver for experimental flumes where the temperature during the experiment has to be mentioned.

4.4.2. ANSYS - FLUENT 15

Computational Fluid Dynamics (CFD) is a numerical analysis tool which is computer based analysis as suggested by its name. It facilitate the engineer with the numerical tool which is require to solve the real time problem without going into much experimental detailing as well as huge capital inclusive equipment. CFD based simulations by researchers are identified in various fields of engineering as numerical hydraulic models can significantly reduce costs associated with the experimental models. The application of CFD is to analyse fluid flow indetail by solving a system of non-linear governing equations over the region of interest by applying specified boundary conditions as well as initial condition. The step has been taken into account for numerical analysis on a straight trapezoidal channel flow. The work will help to simulate the different flow variables in such type of flow geometry. Using CFD which was integral part of the project had been only technique of simulation in this project. Simulation in CFD relies on numerical accuracies, modelling precision and computation cost. In general, CFD is a technique to simulate and predict phenomena in applications such as fluid flow, heat transfer, mass transfer, and chemical reactions. The process of numerical simulations involved while solving any equation involve Pre-Processing Solver and Post-Processing.

There are number of CFD programs available that possess capabilities for modelling multiphase flow. Some common programs include ANSYS and COMSOL, which are both multiphysics modelling software packages and FLUENT, which is a fluid-flow-specific software package. CFD is a popular tool for solving transport phenomenon because of its ability to give results for problems where no statistical tool (i.e. correlations) or experimental data exist and also to produce results of real-time situations.

4.4.2.1. Development of Model from ANSYS

In the present work, an effort has been made to investigate the velocity profiles and boundary shear stress distribution for different depths of a simple trapezoidal channel having sedimentation as no load condition by using a computational fluid dynamics (CFD) modelling tool, named as FLUENT. The CFD model developed for a real open-channel was first validated by comparing the velocity distribution and boundary shear stress distribution obtained by the numerical simulation with the actual measurement carried out by experimentation in the same channel using Pitot tube for velocity distribution and Preston tube for boundary shear stress distribution. The CFD model has been the used to analyse the effects of flow due to different types of sedimentation. The simulated flow field in each case is compared with corresponding laboratory measurements of velocity distribution and boundary shear stress distribution. Computational Fluid Dynamics (CFD) is a mathematical tool which is used to model open channel ranging from in-bank to over-bank flows. Different models are used to solve Navier-Stokes equations which are the governing equation for any fluid flow. Finite volume method is applied to discretize the governing equations. The accuracy of computational results mainly depends on the mesh quality and the model used to simulate the flow. Fig 4.4 shows the grid generation over the experimental channel in the pre-processing stage of ANSYS-FLUENT.

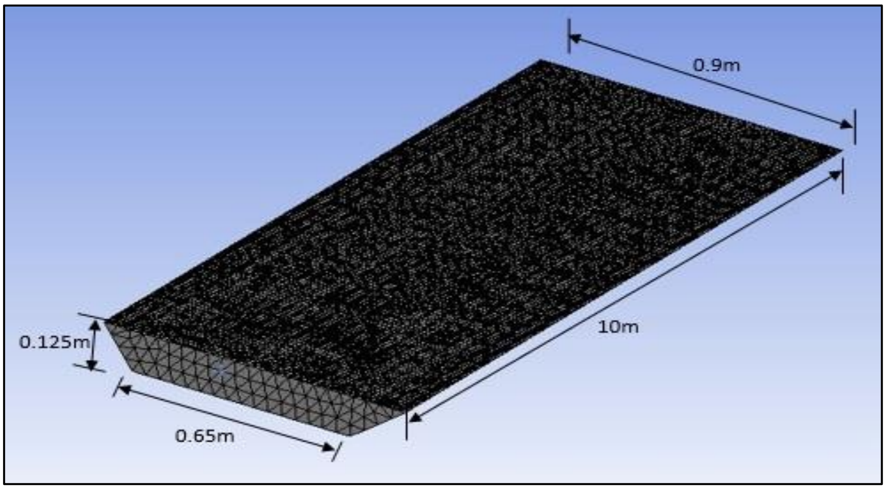

Fig. 4.4 Experimental channel Meshed create using ANSYS

4.5. Advantages of Numerical Modelling hydraulic software

Experimental approach has some drawbacks such as laborious data collection and data can be collected for a limited number of points due to instrument operation limitations even though exact results and clear understanding of flow phenomena are seem to be its merit the three-dimensional flow behaviour or some complex turbulent structure which is the nature of any open channel flow cannot effectively capture through experiments on the model which is usually not at the full-scale model. So in these circumstances, complimentary to experimental is computational approach that can be adopted to overcome some of these issues. While compared with experimental studies, computational approach is repeatable and can simulate at the full-scale model. It can generate the flow taking all the data points into consideration and moreover can take greatest technical challenge i.e. prediction of turbulence. The complex turbulent structures like vortices, Reynolds stresses, secondary flow cells, etc. can be identified by numerical modelling effectively, which are quite essential for the study of energy outflow in open channel flows. An accounting of the various advantages of its simulation and numerical problem ability had indulged many researchers' interest in working on open channel flow with this tool.

CHAPTER 5

RESULTS AND DISCUSSIONS

RESULTS AND DISCUSSIONS

5.1 General

Chapter 3 represents the different experimental procedures including measurement techniques including flow depth (stage), velocity, slope of the channel, boundary shear stress, bed load sediment transport rate etc. Theoretical considerations for the evaluation of different hydraulic parameters of the gravel bed open channel are discussed in Chapter 4. This chapter presents the results of the various parameters calculated from the experimental runs and discussions.

5.2. Longitudinal Velocity Distribution from experimental data

As discussed in Chapter 3 (Experimental Program), velocity at various longitudinal distances along the path perpendicular to flow direction were measured by Pitot tube of diameter 4.77mm for each experimental run in the gravel bed open channel under both no load and intense load flow conditions. In this section, longitudinal velocity distributions were discussed for no load conditions. Velocities were measured at every 0.2 *h* intervals where *h* is the flow depth. These measured values of velocities were used to plot velocity profiles and the values were used for comparing with results from ANSYS. Detailed experimentations of total five depth shave been performed and contour maps were prepared over the flow section. From Figs. 5.1.a to 5.1.e shows the contour maps for different flow depths in trapezoidal channel with gravel bed. At lower depth minimum velocity occurs at the bed and maximum velocity at the free surface when the depths increases the threads of maximum velocity occurs at the free surface of the wall. But for higher flow depth the maximum velocity do not occur at the middle of the free surface but occur at the side of the free surface. This may be due to more uniformly in high flow depths in such channels.

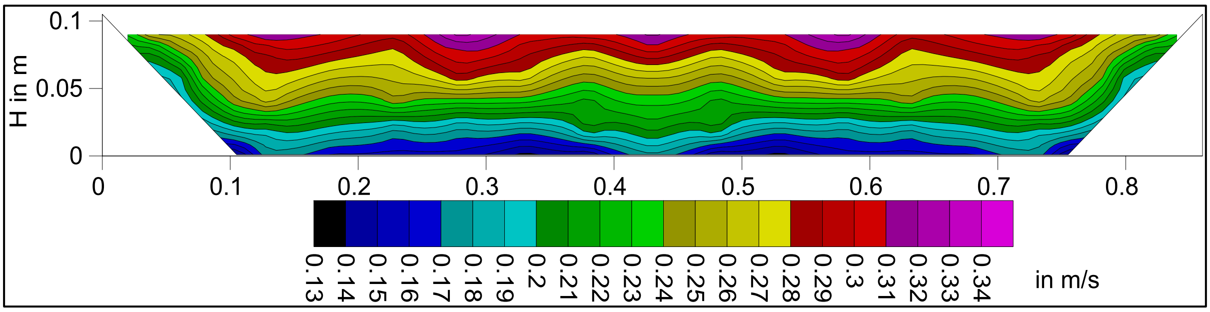

Fig. 5.1 a. longitudinal velocity contour of flow depth 0.07 m

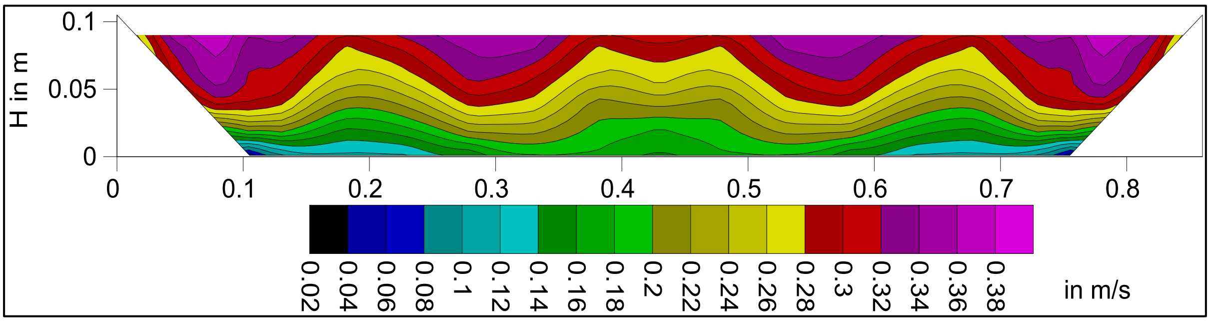

Fig. 5.1 b. longitudinal velocity contour of flow depth 0.08 m

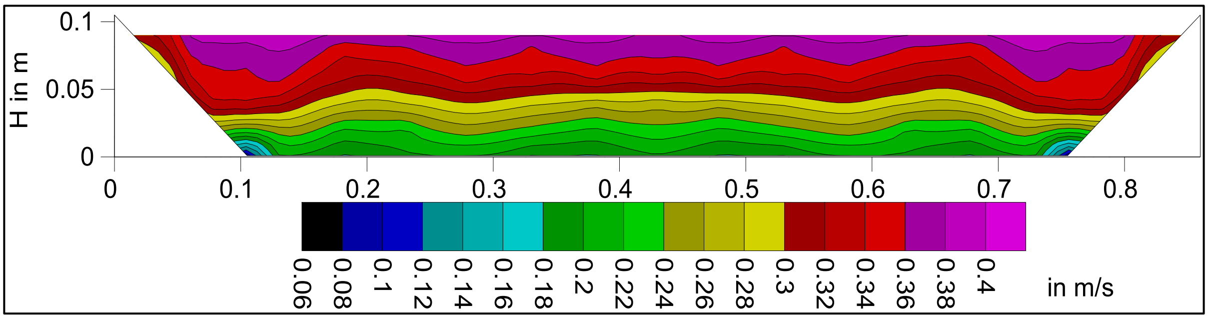

Fig. 5.1 c. longitudinal velocity contour of flow depth 0.086 m

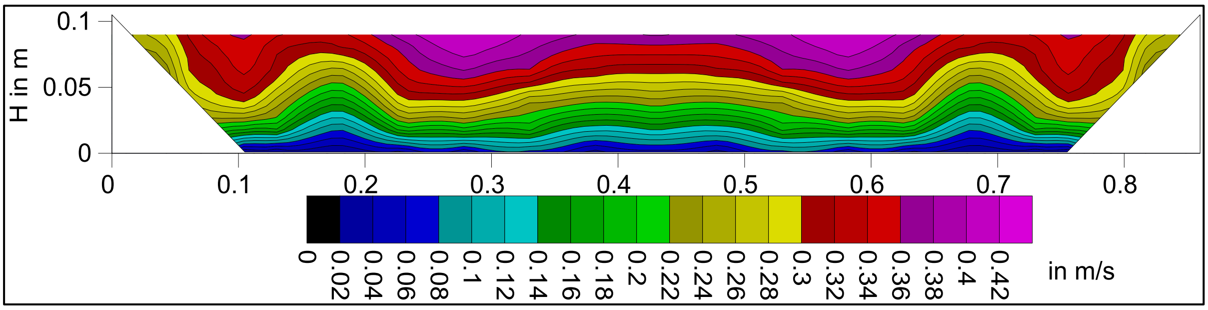

Fig. 5.1.d. longitudinal velocity contour of flow depth 0.0916 m

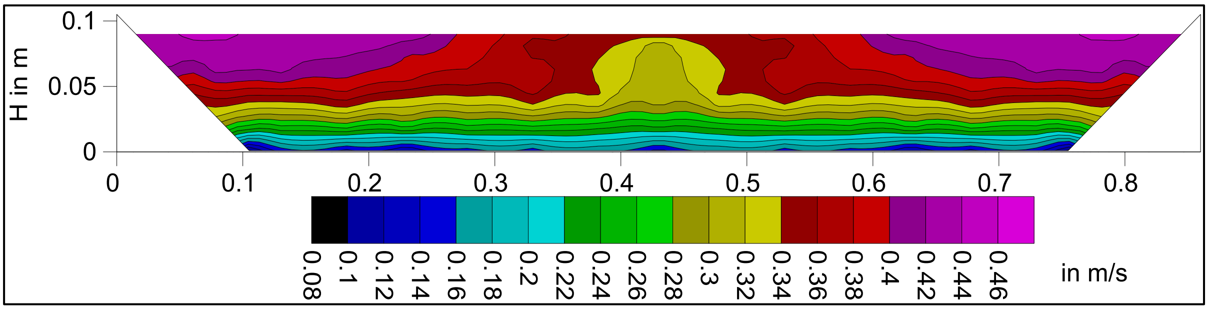

Fig. 5.1.e. longitudinal velocity contour of flow depth 0.10 m

5.3. Application of Numerical Analysis

5.3.1. Velocity distributionobtained from ANSYS

Generally, experimental and theoretical analyses are the main tools for finding out the solution of open channel flow problems to meet the needs of field requirements. In recent times CFD techniques are being used extensively to solve the flow problems. In this study, a few simulations were carried out by using the commercial code namely ANSYS to simulate the present experimental investigation. Total five flow depth has to be considered for no load conditions. 13.5 mm gravels were used in beds of open channel flow to predict the velocity distribution along the channel bed. Here k-ε model is used for turbulence modelling. The k-ε equations are discretized in both space and time. In this study the algorithms adopted to solve the coupling between pressure and velocity field is PISO which is the pressure implicit splitting operators use in Fluent (Issa 1986).

Fig. 5.2a. to Fig. 5.2e shows the velocity distribution has developed from ANSYS for no load conditions. In this present study, it has found that ANSYS results were well matching with the data collected from the experiment. Thus we can conclude from this simulation that we can use the data ANSYS instead of doing experiments for the case of no load transport conditions.

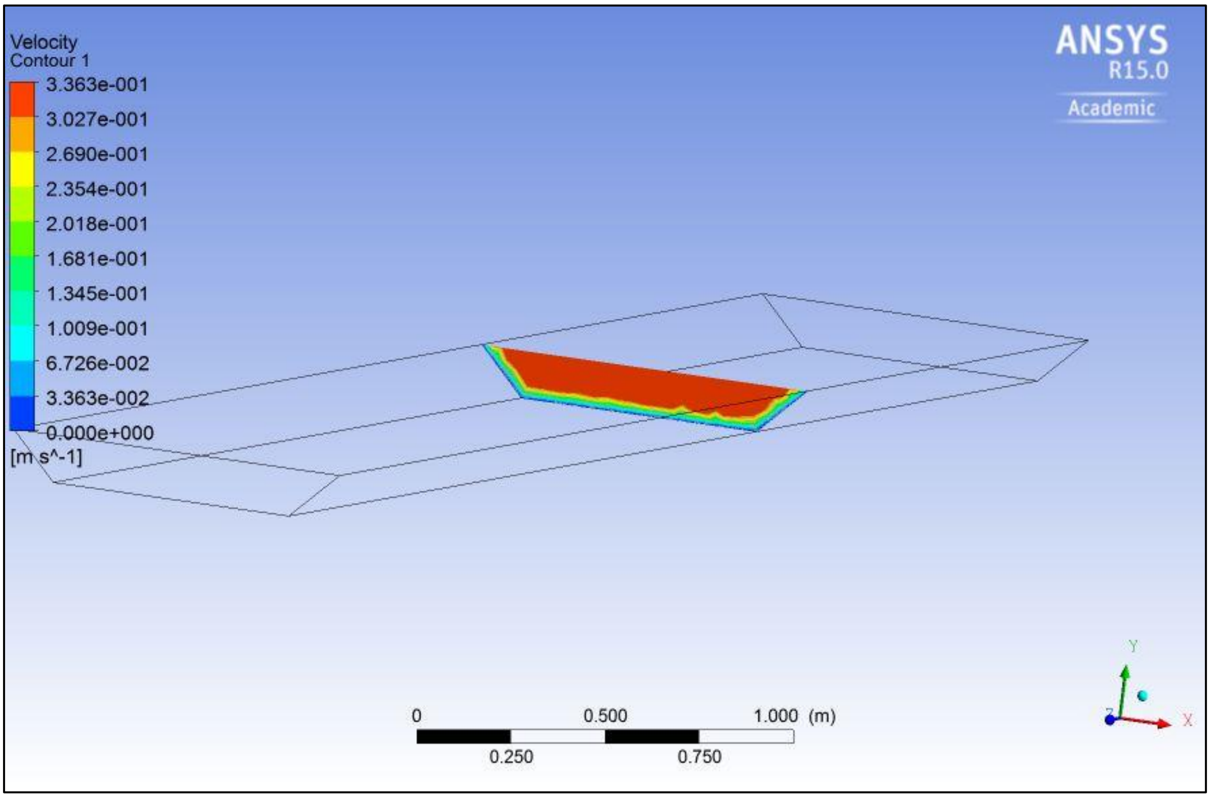

Fig. 5.2.a longitudinal velocity contour of flow depth 0.07 m using ANSYS

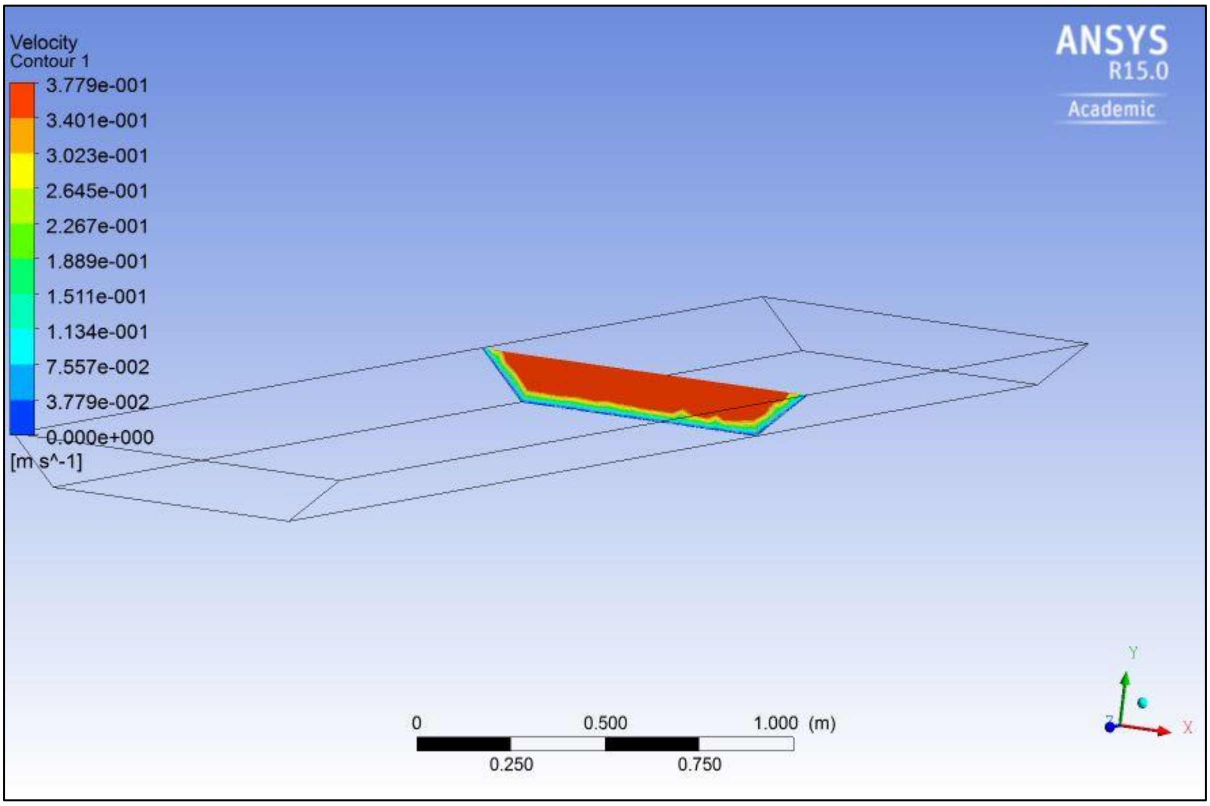

Fig. 5.2.b longitudinal velocity contour of flow depth 0.08 m using ANSYS

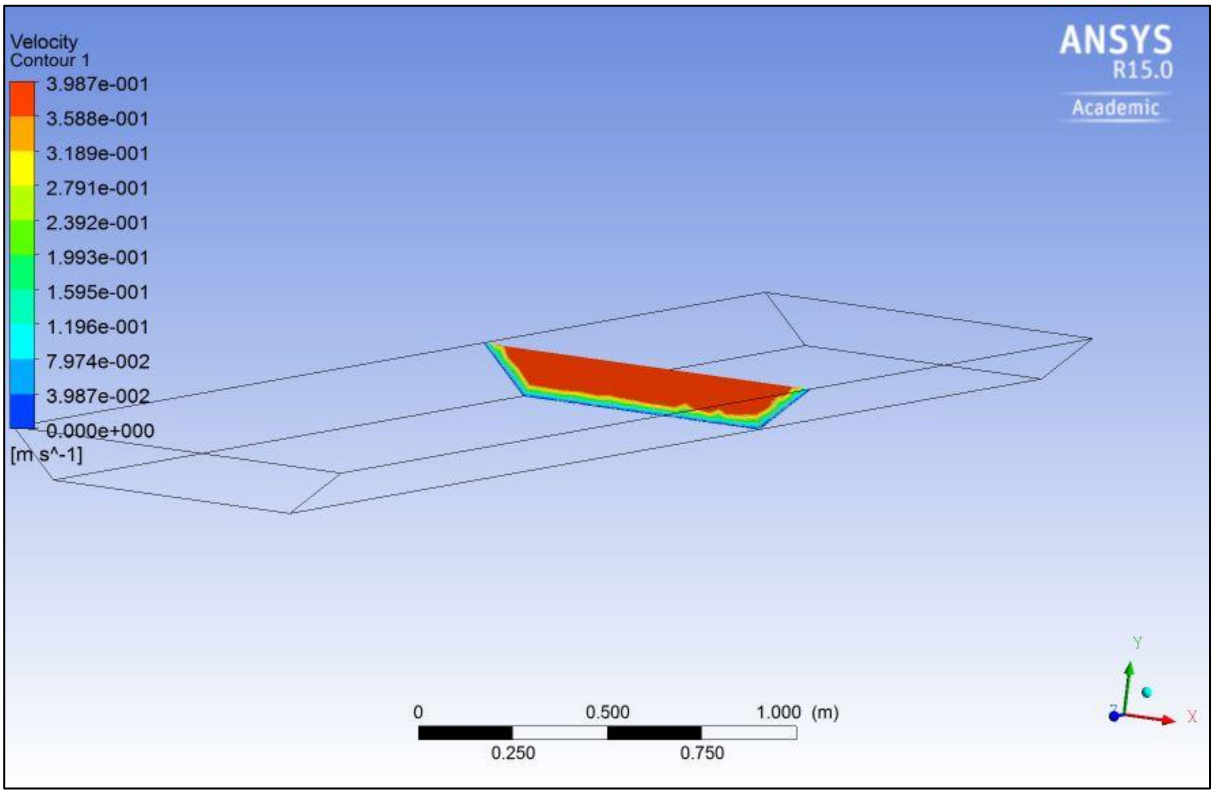

Fig. 5.2.c longitudinal velocity contour of flow depth 0.086 m using ANSYS

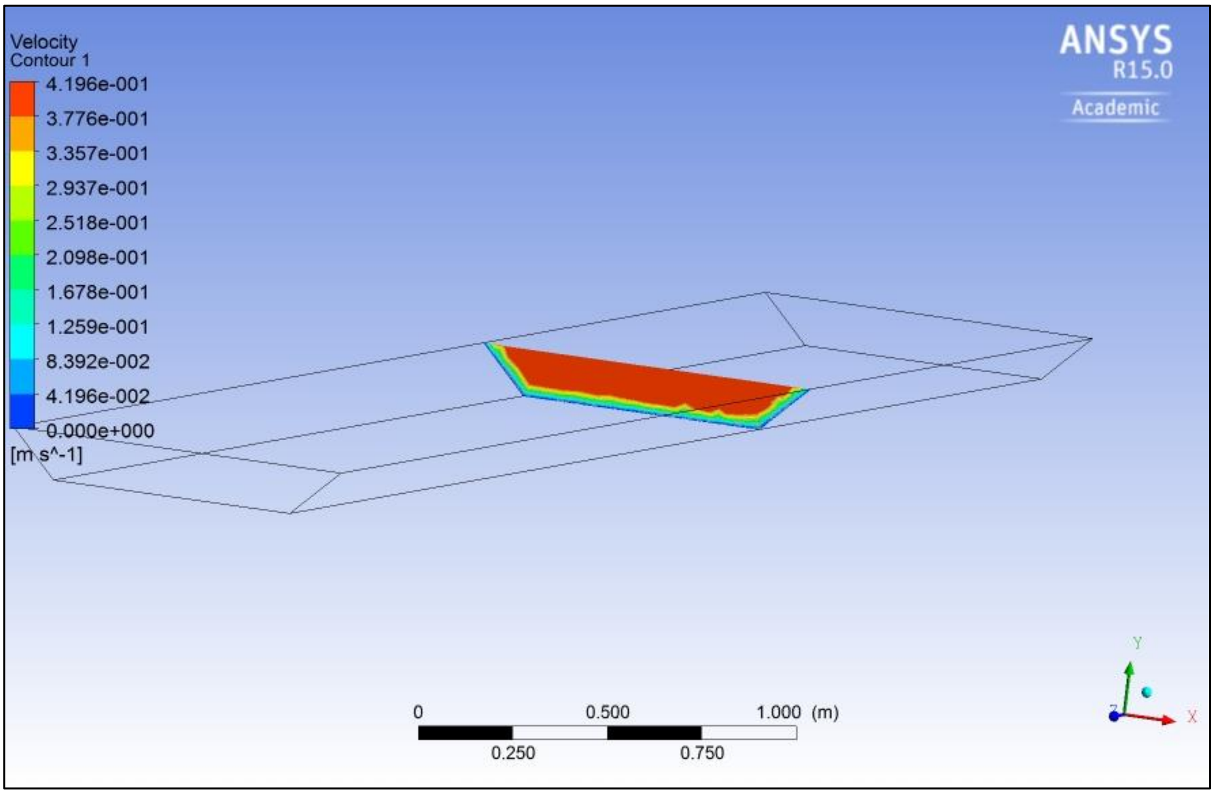

Fig. 5.2.d longitudinal velocity contour of flow depth 0.0916 m using ANSYS

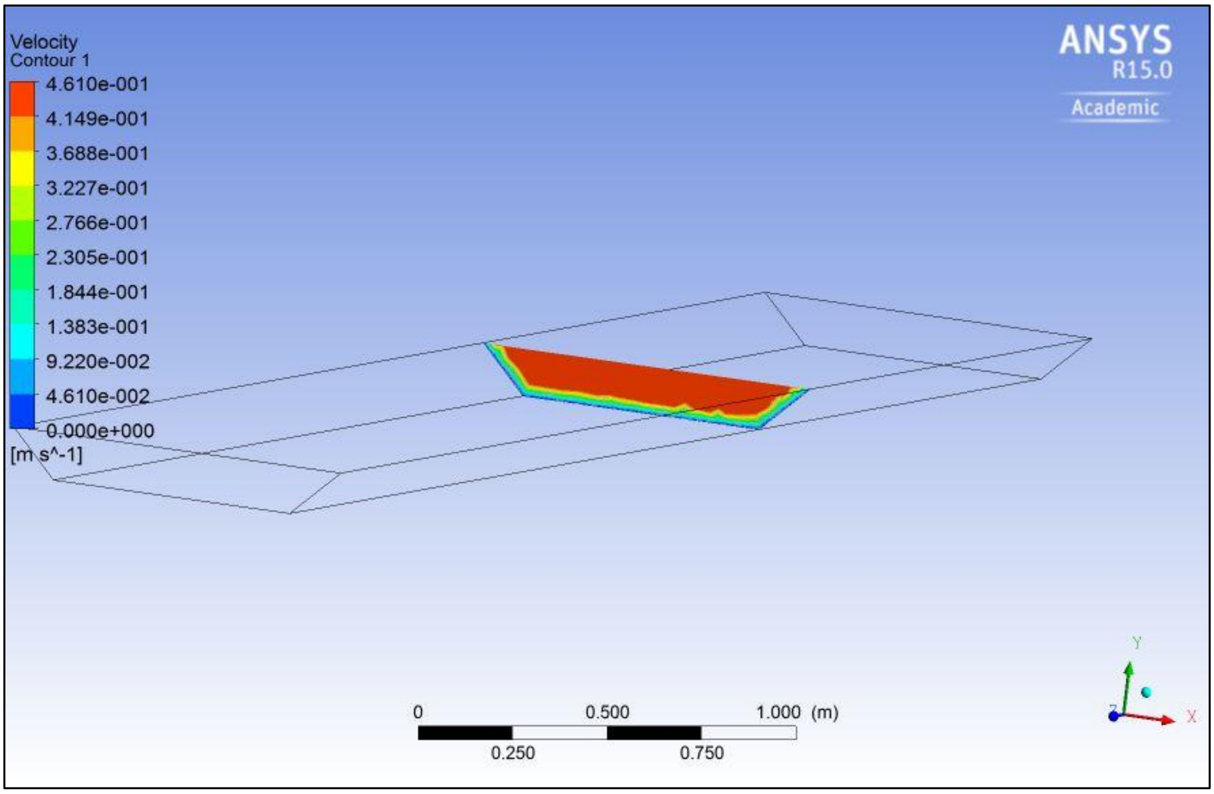

Fig. 5.2.e longitudinal velocity contour of flow depth 0.10 m using ANSYS

5.3.2. ComparisonofBoundary Shear Stress distribution results for no load condition

Boundary shear stress was measured from point to point along the wetted perimeter of the channel by using Preston tube of diameter 4.77 mm along with the various longitudinal distances. A total five flow depths were considered for studying the boundary shear stress distribution of the experimental channel. In this section boundary, shear stress has been measured only for no load gravel bed condition. The measured point boundary shear stresses (τ) are plotted across the flow domain for no load flow condition in Figs. 5.3.a to 5.3.e. From these figure it can be concluded that for both experimental and numerical results maximum boundary shear occur at the interface between channel bed and the side wall of the trapezoidal channel. For higher roughness i.e. larger gravel bed size 13.5 mm the boundary shear stress tends to decrease. Boundary shear at both ends are fond to be same for all flow depths, however at the middle of the channel, the boundary shear stress increases with increase in flow depths. All the numerical software found to give good results as compared to experimental results. But ANSYS found to provide more accurate as compared to CES because ANSYS takes the 3D nature of flow that occurs in a rough gravel bed channel whereas CES cannot. CES is a quasi softwaretake the 1D effect only. It is based on depth-averaged of RANS equation. For both the software initial and boundary condition and roughness value has as same as the experimental data.

Fig. 5.3a to Fig. 5.3e shows the comparison of boundary shear stress distribution of 13.5 mm gravel bed experimental data with CES and ANSYS data. From these figures, we observed that CES over predict the boundary shear whereas ANSYS under predict the experimental data. Thus we can conclude that ANSYS provided better result of boundary shear stress distribution than CES.

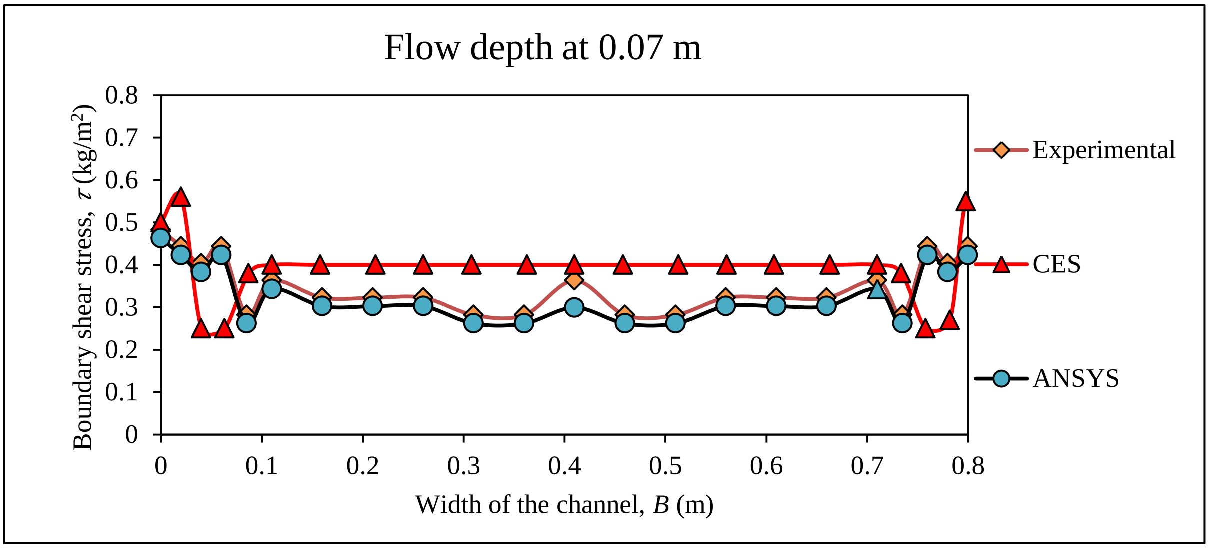

Fig. 5.3.a. Comparison of Boundary shear stress distribution for flow depth 0.07m

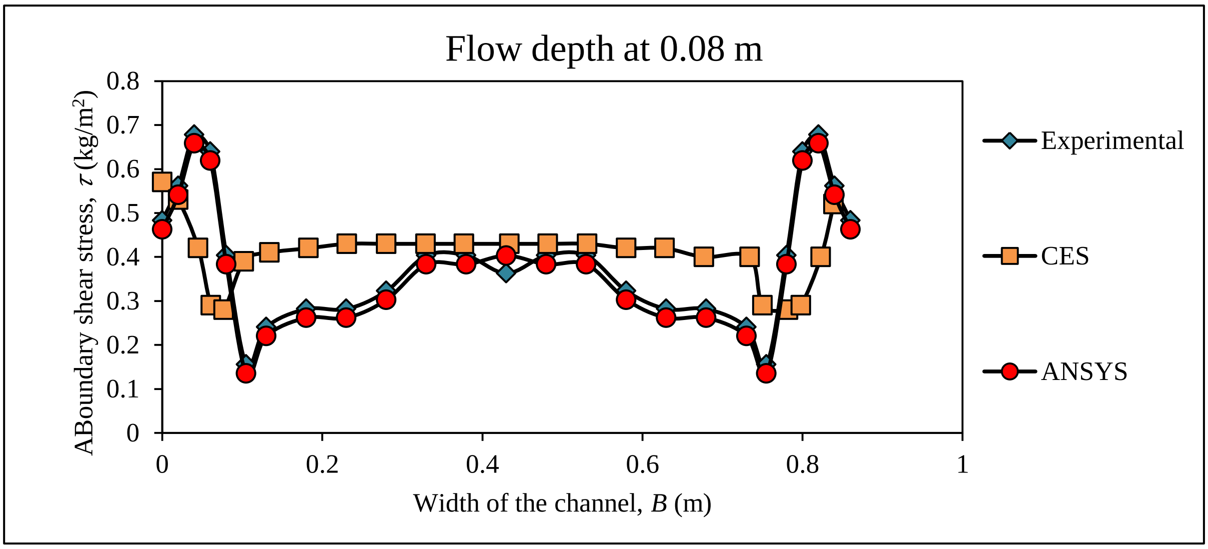

Fig. 5.3.b. Comparison of Boundary shear stress distribution for flow depth 0.08m

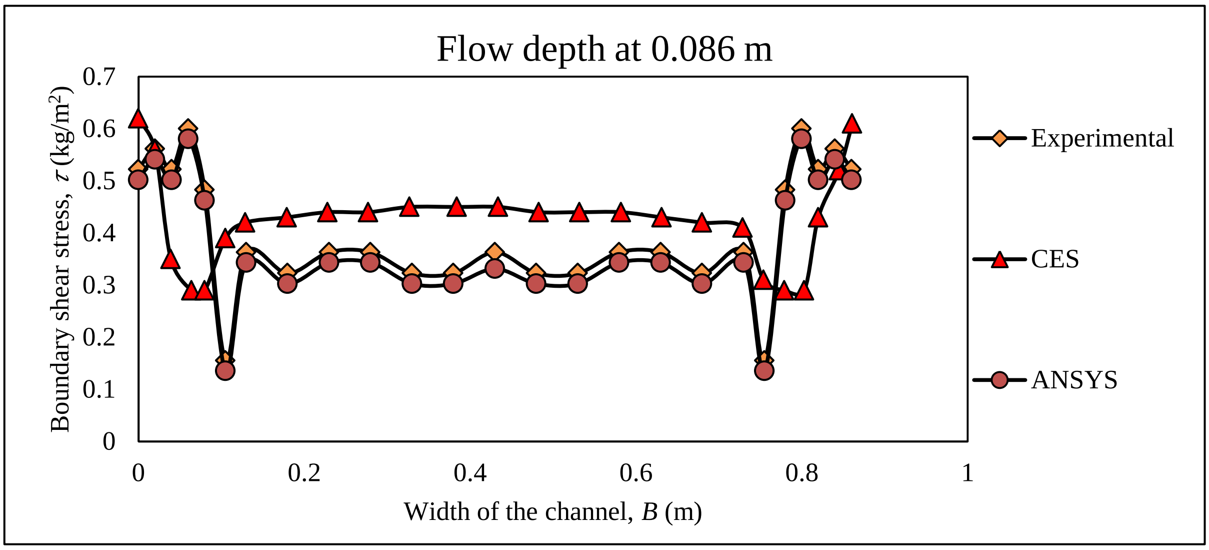

Fig. 5.3.c. Comparison of Boundary shear stress distribution for flow depth 0.086m

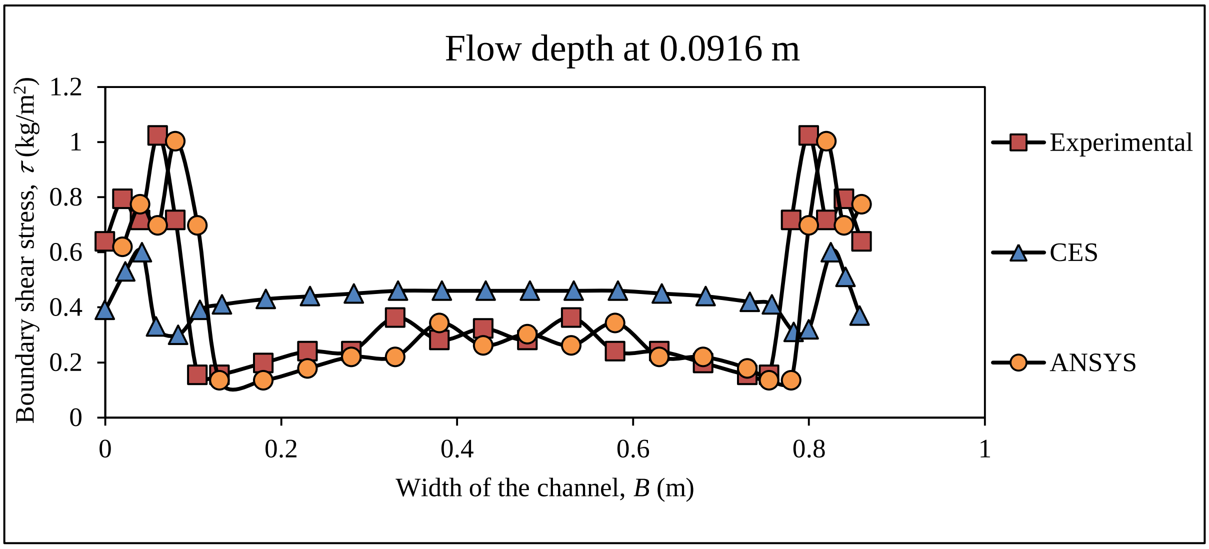

Fig. 5.3.d. Comparison of Boundary shear stress distribution for flow depth 0.096m

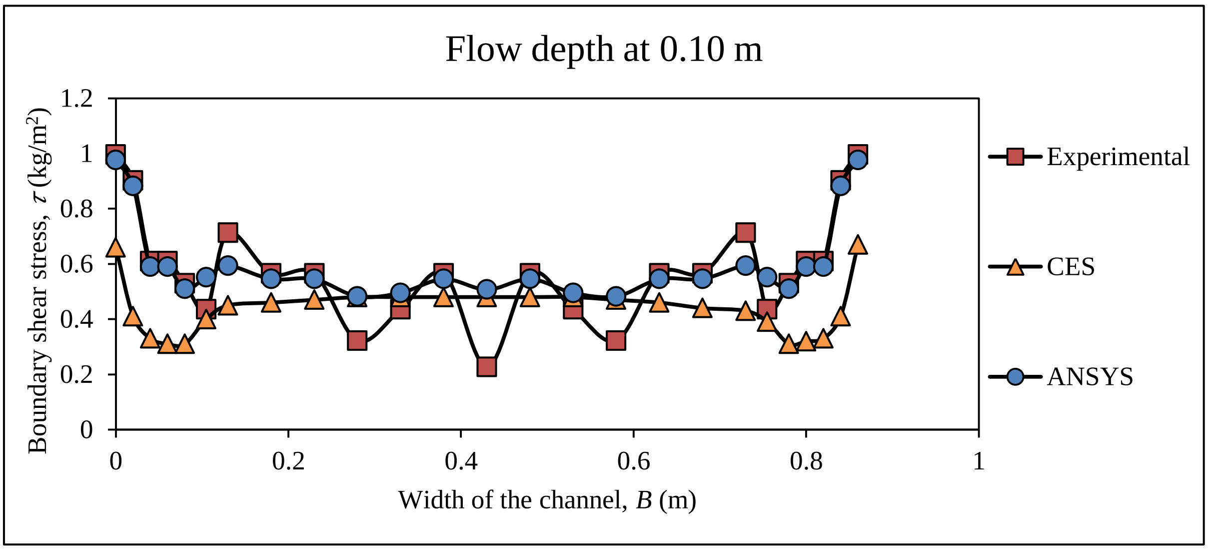

Fig. 5.3.e. Comparison of Boundary shear stress distribution for flow depth 0.10 m

5.4 Expression for resistance

5.4.1. Relative Depth for both no load and intense load conditions

For all the experimental runs fewer than two gravel bed conditions, the value of the diameter of the gravels (*D*) are 0.0135m and 0.0065m. By increasing flow depth (*H*), the value of hydraulic radius (*R*) increases and thus the relative depth (*R/D*) increases. In the present study, values of relative depth were taken from 1.54 to 5.98 as the Hydraulic radius increased from 0.02 to 0.08 m for no load condition and for intense load condition relative depth were taken from 2.07 to 12.41 as the Hydraulic radius are chosen from 0.014 to 0.08 m.

5.4.2. Stage-Discharge Variation of gravel bed Open Channel for no load condition

Discharge is an important parameter used in this present study. Accurate measurement of discharge is most important since it is frequently used in calculation of a number of hydraulic parameters. Measurement of discharge is always difficult for an open channel flow at the start of each experimental run. If a relationship between discharge and stage of flow which is otherwise called as depth is developed, then it will save considerable time in computing discharge instead of measuring it. From the developed relationship, one can compute the discharge if head or depth of water is measured which is relatively easier than measuring the discharge. As discussed in Chapter 3, 17 runs each at 17 different depths of flow were carried out in the hydraulic flume with sediment which is used in the experiment study under no load flow condition. Relationship between these discharges and their individual stages (head/depth) were studied. The developed relationship is found to be in the power form with high value of coefficient of determination ($R^2 = 0.99$). The graphical relationship is presented in Fig. 5.4. The relationship is found as to be

$$h = 0.70\ Q^{.56} \tag{5.1}$$

Where, h = depth of flow in the channel, m and Q = discharge, m^3/s

. In the present experimental study, Eq. 5.1 was used to estimate the discharge flowing in the channel under uniform flow conditions with gravel, using different values of depths of flow, *h* in the channel. For checking the accuracy, some of the values of the discharges so computed by Eq. 5.1, were compared with the measured values and the results are found to be very much satisfactory with very negligible difference in discharge values.

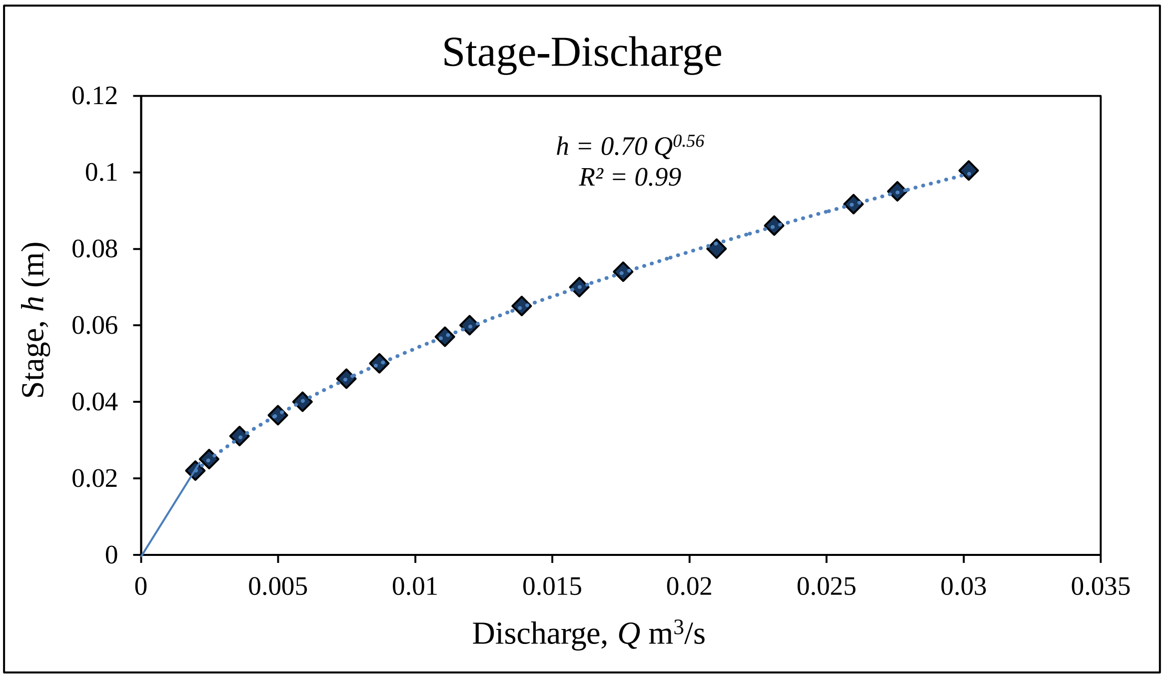

Fig. 5.4. Stage-discharge relationship for no load condition

5.4.3. Experimental details for no load condition

In the present study, the relevant variables and parameters are discharge, depth of flow, width of channel section, cross-sectional area of flow, wetted perimeter, hydraulic radius, velocity, shear velocity and relative depth. The detailed geometric and hydraulic parameters are measured experimentally for 13.5 mm gravel bed channel. The values so measured and evaluated are mentioned in Table 5.1. For all the experimental runs, diameter of gravel was kept constant as 0.0135m. Moreover, a constant bed slope of the channel (0.25%) was used for all the runs. Total 17stage-discharge experimental runs for no load flow conditions were taken for this study.

In this study, the discharges varied from 0.002 to 0.0302 m^3/s and corresponding depth of flow varies from 0.022 m to 0.10 m. Values of average velocity has been varied from 0.13 to 0.40 m/sec (Table 5.1). Mean channel velocity of gravel bed channel was found to increase with increasing discharge and depth of flow.

Table 5.1 Experimental data sets for no load condition

Discharge Q (m^3/s)	Flow depth h (m)	Cross section area A (m^2)	Wetted perimeter P (m)	Hydraulic radius $R=A/P$	Mean channel velocity u (m/s)	Relative Depth(R/D)	$u/u*$
0.0302	0.1004	0.075	0.93	0.08	0.40	5.97	9.01
0.0276	0.095	0.070	0.91	0.077	0.38	5.70	8.95
0.026	0.0916	0.068	0.90	0.074	0.38	5.53	8.93
0.0231	0.086	0.063	0.89	0.070	0.36	5.24	8.73
0.021	0.08	0.058	0.87	0.066	0.36	4.93	8.91
0.0176	0.074	0.053	0.85	0.062	0.32	4.61	8.38
0.016	0.07	0.050	0.84	0.059	0.31	4.40	8.29
0.0139	0.065	0.047	0.83	0.055	0.29	4.12	8.06
0.012	0.06	0.043	0.82	0.051	0.28	3.84	7.91
0.0111	0.057	0.040	0.81	0.049	0.27	3.67	7.88
0.0087	0.05	0.035	0.79	0.044	0.24	3.27	7.57
0.0075	0.046	0.032	0.78	0.04	0.23	3.04	7.42
0.0059	0.04	0.028	0.76	0.036	0.21	2.67	7.13
0.005	0.0365	0.025	0.75	0.033	0.20	2.46	7.07
0.0036	0.031	0.021	0.73	0.028	0.17	2.12	6.48
0.0025	0.025	0.017	0.72	0.023	0.14	1.73	6.23
0.002	0.022	0.015	0.71	0.020	0.13	1.58	5.94

5.4.4. Friction Factor for No load condition

As discussed in chapter 4, a linear regression model is developed to predict friction factor, *f* for gravel bed uniform flow condition. Friction law proposed by Keulegan (1938) for flows over fixed beds was restriction to flows without bed load. The effect of friction coefficient *f*, due to the bed load is observed to decrease with increase of relative depth (*R/D*)

Later Recking (2006) stated that the friction law developed by Keulegan (1938) was valid only for *R/D*>8.6. The modified model by Recking (2006) was claimed to be valid for 2<*R/D*<8.6.

The present model for modelling friction factor for gravel bed channel is developed for the range of *R/D* value less than 8.6. In total, 71 data sets from the experimental observation, including that of Recking (2006); have been utilized in developing the new model. The data sets of Recking (2006) are tabulated in appendix.

Figure 5.5 illustrates a semi-logarithmic relationship between *R/D* and friction factor in terms of $(8/f)^{1/2}$ for uniform flow case. The linear regression curve shows a higher value of correlation coefficient, indicating a satisfactory relation between the *R/D* and $(8/f)^{1/2}$.

The linear regression expression for the determination of friction factor is given as

$$\sqrt{\frac{8}{f}} = 2.95\ \ln\left(\frac{R}{D}\right) + 3.89 \qquad (R^2 = 0.84) \qquad (5.2)$$

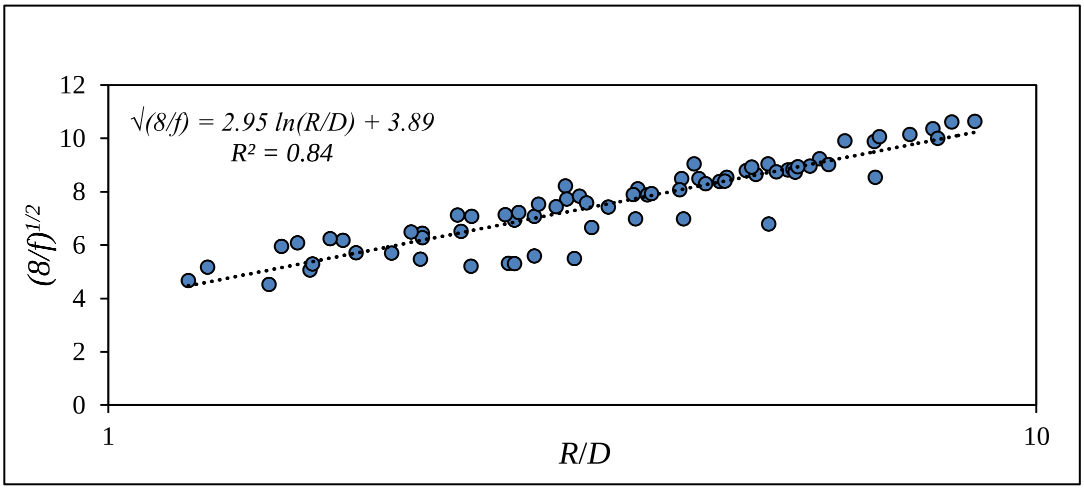

Fig. 5.5. Friction factor variation for no load condition

The expression in the previous section is validated with the data sets of other researchers such as Paintal (1971), Cao (1985), Graf and Suzuka (1987), Recking (2006) etc. along with the present experimental observations. The number of data sets being 145 in total which are given in appendix. The performances of models suggested by previous investigators have also been analyzed for the above datasets.

Fig. 5.6shows the performance of different models with equation 5.2 is predicting the value of friction factor, *f* for no-load condition for all the available data sets. The present model is observed to give better predictions.

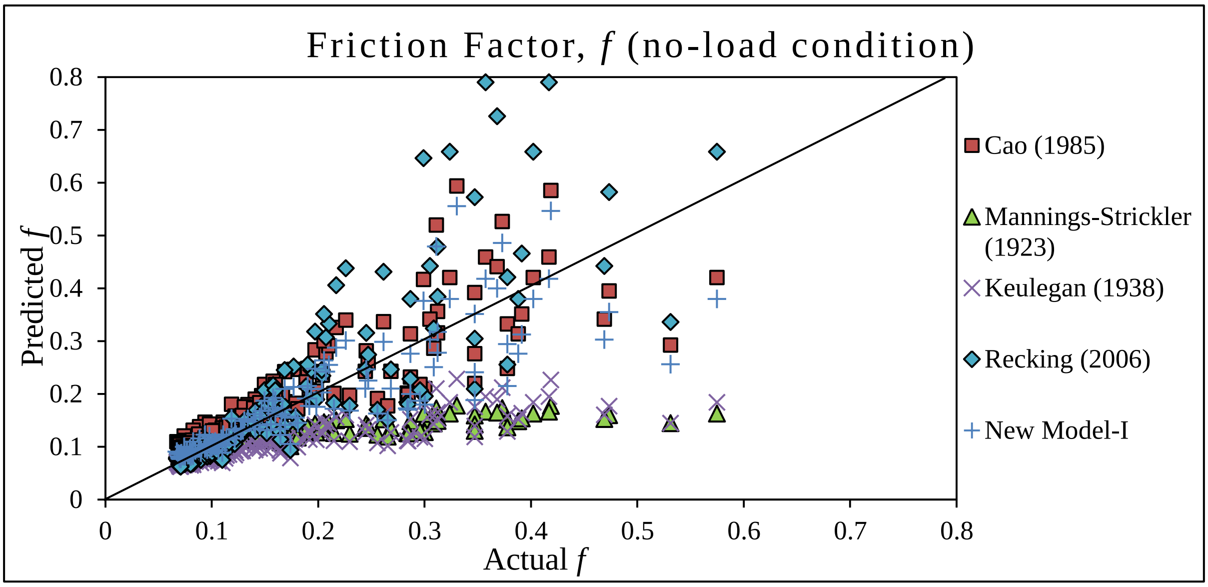

Fig.5.6. Validation of friction factor model for no load condition with other author data

5.4.4.1. Error Analysis

Fig. 5.7 (a-e) illustrates the error analysis of different data sets uniquely by using four most popular models for estimating friction factor, *f* (chapter 4). The developed expression by the other authors has been incorporated in this analysis. The developed expression in equation 5.2 is observed to provide with lower value of MAPE and RMSE for each of the datasets (1971) and Graf & Suzuka (1987) which are quite close to the present model but gives expression as better solution on finding the friction factor for no load conditions.

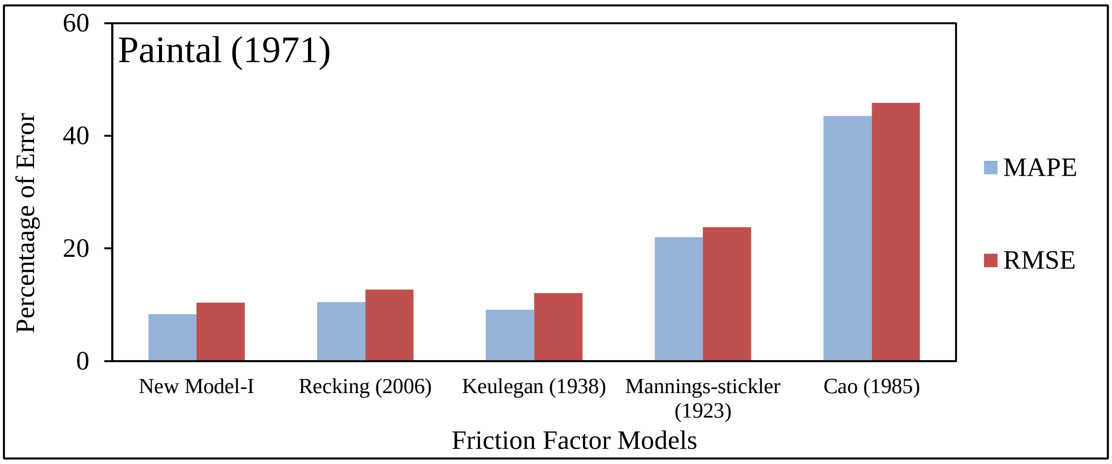

Fig. 5.7.a Error Analysis of friction factor model for no load condition with Paintal (1971) data

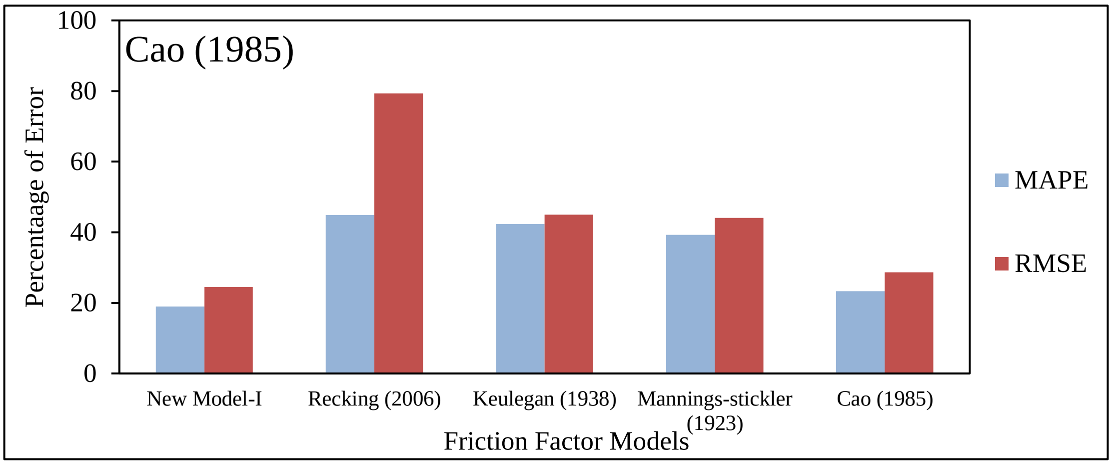

Fig. 5.7.b Error Analysis of friction factor model for no load condition with Cao (1985)data

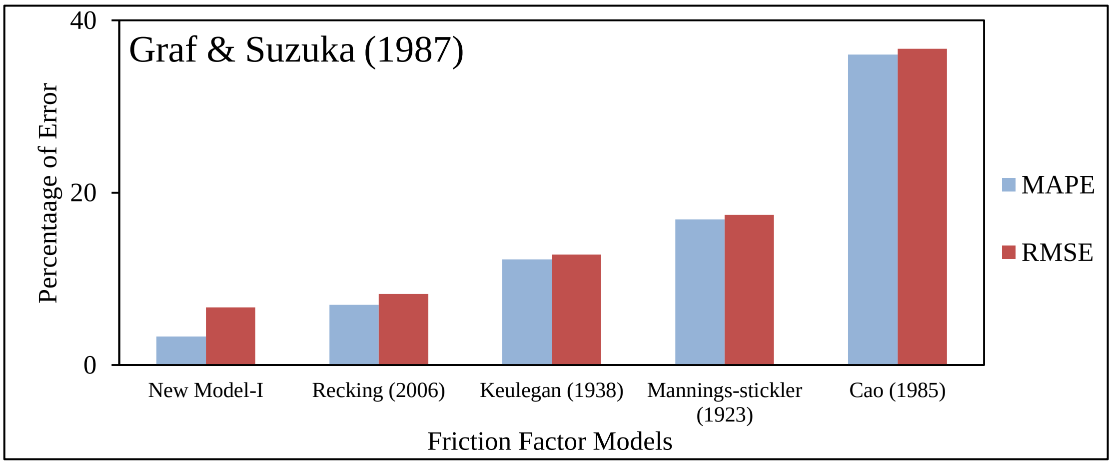

Fig. 5.7.c Error Analysis of friction factor model for no load condition with Graf & Suzuka (1987) data

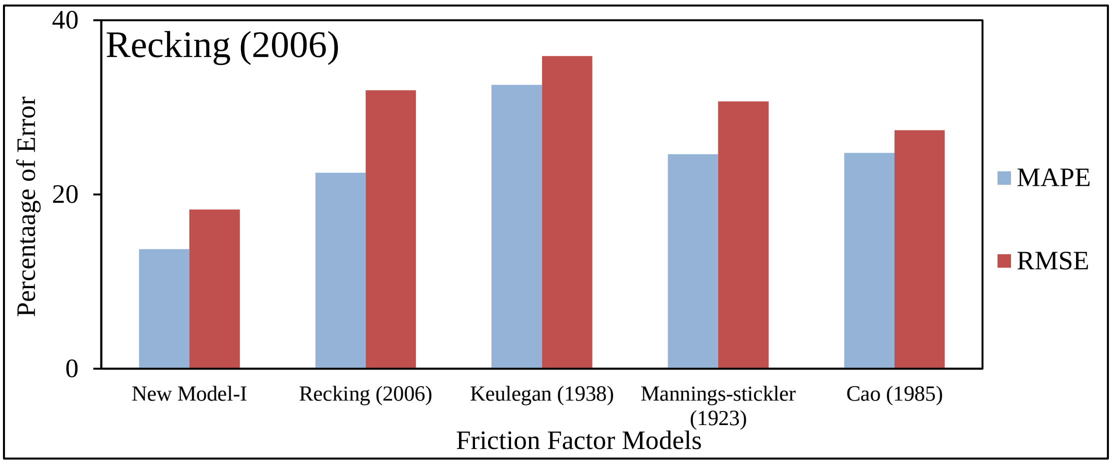

Fig. 5.7.d Error Analysis of friction factor model for no load condition with Recking (2006) data

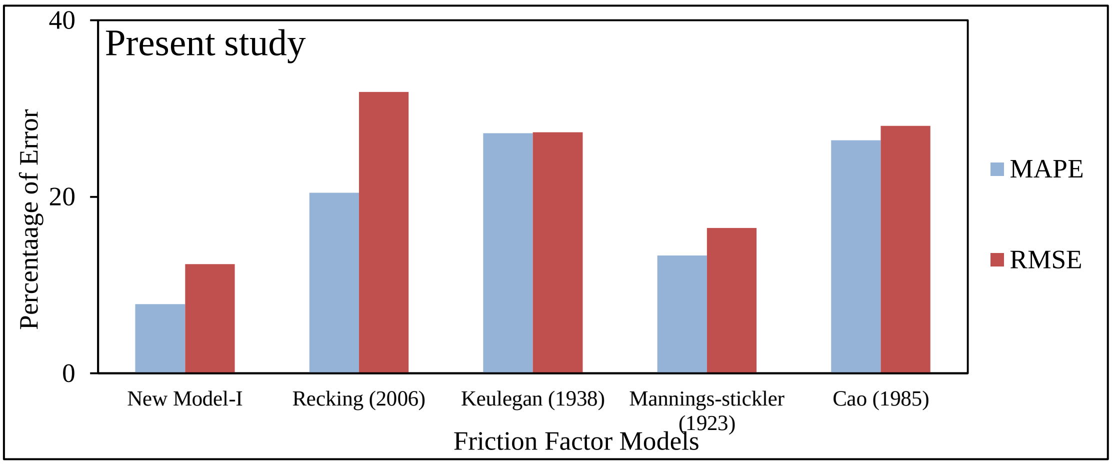

Fig. 5.7.e Error Analysis of friction factor model for no load condition with present study data

5.4.5. Stage-Discharge variation of gravel bed Open Channel for intense load condition

As total 18runs, each at 18 different depths of flow was carried out in the trapezoidal hydraulic flume with sediment which is used in the present experiment study under intense load flow condition. Relationship between these discharges and their individual stages were calculated. The relationship is found to be in the power function with coefficient of determination of (R^2 = 0.99). The graphical relationship is presented in Fig. 5.8. The relationship is found as:

$$h = 0.58\, Q^{.51} \tag{5.3}$$

Where, h = stage or head or depth of flow in the channel, m and Q = discharge, m^3/s.

In the present experimental study, Eq. 5.3 was used to estimate the discharge flowing in the channel under uniform conditions with gravel, using different values of depths of flow, *h* in the channel. With the help of this stage-discharge data friction factor has been calculated for intense load conditions. As in this study, 6.5 mm gravel has been used thus relative depth in this section was found more than load conditions. In this study flow depths were remaining constant as well as for the no load conditions this channel velocity was almost same. The sediment transport rate had also been measured with the help of measurement technique which was discussed in chapter-3.

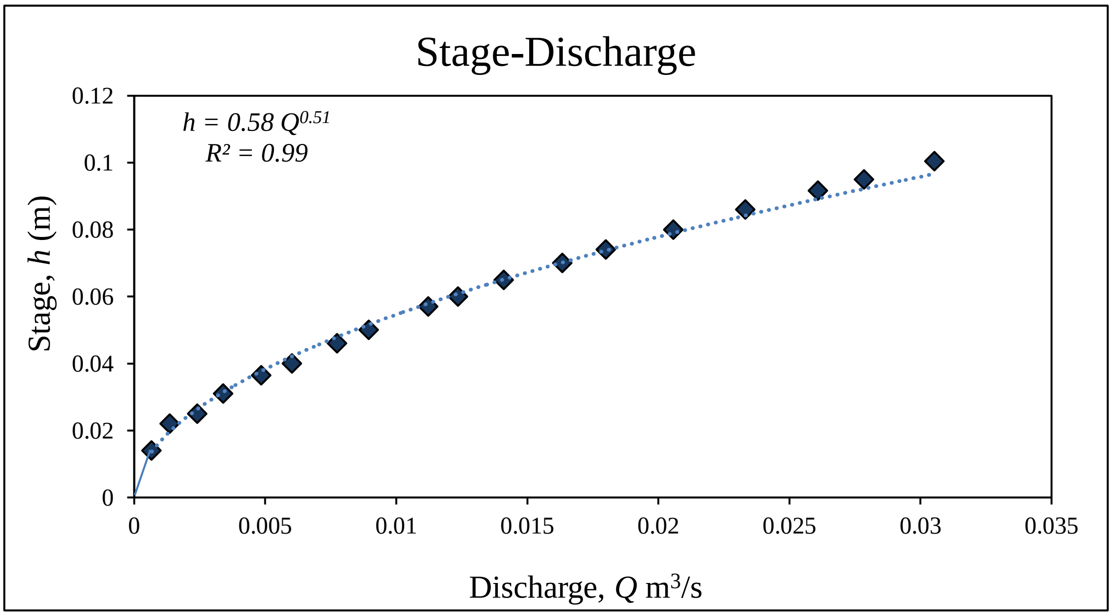

Fig.5.8. Stage-discharge relationship for intense load condition

5.4.6. Experimental results for intense load condition

The measured values or evaluated values of geometric and hydraulic parameters are mentioned in Table 5.2. For all the experimental runs, diameter of gravel was kept constant as 0.0065m. Moreover, a constant bed slope of the channel (0.25%) maintains the subcritical flow conditions were used for all the runs. Total 18 experimental runs for uniform flow conditions were taken for the study.

In this study, the discharges has been varied from 0.001 to 0.0305 m^3/s and the corresponding depth of flow varies from 0.014 m to 0.22 m. Mean channel velocity of gravel bed channel was found to increase with increasing discharge and depth of flow.

Table 5.2. Experimental data sets for intense load condition

Discharge Q (m^3/s)	Flow depth h (m)	Cross section area A (m^2)	Wetted perimeter P (m)	Hydraulic radius $R=A/P$	Mean channel velocity u (m/s)	Relative Depth(R/D)	$u/u*$
0.030	0.1004	0.075	0.93	0.080	0.405	12.41	9.11
0.028	0.095	0.071	0.91	0.077	0.39	11.85	9.06
0.026	0.0916	0.068	0.90	0.074	0.38	11.49	8.97
0.023	0.086	0.063	0.89	0.070	0.36	10.90	8.83
0.020	0.08	0.058	0.87	0.066	0.35	10.25	8.71
0.018	0.074	0.0531	0.85	0.062	0.33	9.59	8.59
0.016	0.07	0.050	0.84	0.059	0.32	9.14	8.49
0.014	0.065	0.046	0.83	0.055	0.30	8.57	8.21
0.012	0.06	0.042	0.81	0.051	0.29	7.99	8.13
0.011	0.057	0.040	0.81	0.049	0.27	7.64	7.98
0.009	0.05	0.035	0.79	0.044	0.25	6.80	7.77
0.007	0.046	0.032	0.78	0.041	0.24	6.31	7.62
0.006	0.04	0.027	0.76	0.036	0.22	5.56	7.33
0.005	0.037	0.025	0.75	0.033	0.19	5.11	6.77
0.003	0.031	0.021	0.73	0.029	0.16	4.40	6.07
0.002	0.025	0.017	0.72	0.023	0.14	3.60	5.95
0.001	0.022	0.014	0.71	0.021	0.09	3.19	4.09
0.0006	0.014	0.009	0.68	0.013	0.07	2.07	3.94

5.4.7. Friction Factor for Intense load condition

As discussed in chapter 4, a linear regression model is developed to predict friction factor, *f* for gravel bed under uniform flow condition with intense load. Recking (2006) developed a model for predicting friction factor, *f* for intense load. This model was supposedly valid for relative depth (*R/D*) ranging from 2 to 16.9.

The present model is also developed for the range of *R/D* value less than 16.9 with values even lower than 2. The Shield's parameter, θ which is of assistance in finding out the bed load intensity, Φ is also a contributing factor for carrying out the new set of experimentations. Recking used θ values in the range of 0.10 to 27, while the present set of experimental data sets have θ values less than 0.15. In total, 90 data sets from the experimental observation, including that of Recking (2006) have been utilized in developing the new model.

Figure 5.9 illustrates a semi-logarithmic relationship between *R/D* and friction factor in terms of $(8/f)^{1/2}$ for uniform flow case for intense load condition. The linear regression curve shows a higher value of correlation coefficient, indicating a satisfactory relation between the two.

The linear regression model developed for the determination of friction factor, *f* is given as

$$\sqrt{\frac{8}{f}} = 3.55\ ln\left(\frac{R}{D}\right) + .29 \qquad (R^2 = 0.92) \qquad (5.4)$$

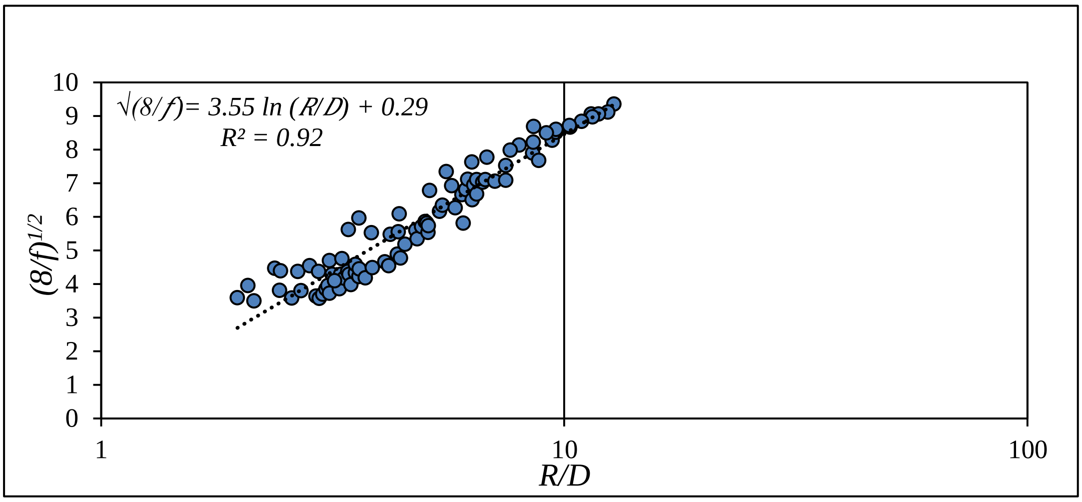

Fig.5.9. Friction factor variation for intense load condition

The equation 5.4establishedabove is verified with the data sets of other investigators such as Bogardi and Yen (1939), Casey (1935), Gilbert (1914), Graf&Suzuka (1987), Hopang-Yung (1939), Mavis et. al. (1937), Meyer-Peter and Muller (1938), Paintal (1971), Recking (2006), Rickenman (1990), Smart and Jaeggi (1983) etc. along with the experimental observations by the author. The total number of data sets being 666 which are given in the appendix section. The functionality of the models suggested by previous researchers has been evaluated for all of the available datasets. Fig. 5.10 shows the working of different models in predicting the values of friction factor, *f* for intense load conditions for all the available data sets. The present model on whole is observed to provide with better predictions.

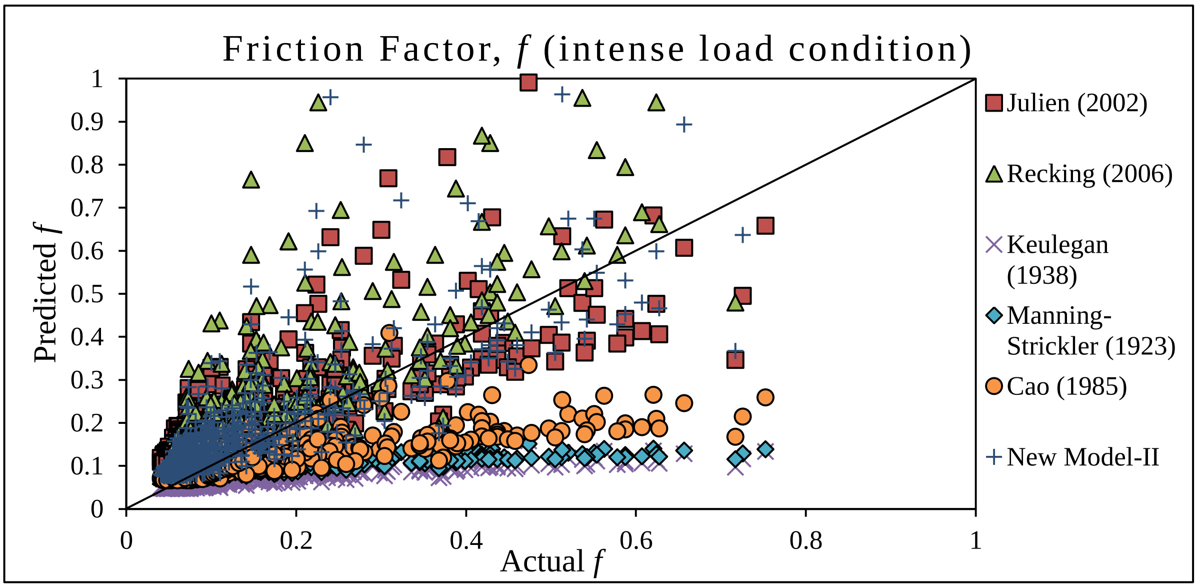

Fig.5.10. Validation of friction factor model for intense load condition with other author data

5.4.7.1. Error Analysis of Friction factor Model for Intense load Flow condition

Error analysis for intense load conditions is carried out by using 5 friction factor models for such cases. The models have been illustrates in chapter 3 for the different range of *R/D* and θ. The present expression, developed by the author in equation 5.4 has also been incorporated in the error analysis.

The error analysis of the different models is carried out for twelve data sets of intense load conditions given in fig. 5.11 (a to l). In almost all the sets of data, the developed expression is observed to provide with lower values of MAPE as well as RMSE in comparison to the other models.

The model suggested by the Keulegan (1938) provides significantly lower values of MAPE as well as RMSE for the data sets of Casey (1935), Mavis et al. (1937), Hopang-Yung (1939)but concurrently gives higher values for the other data sets. The above observations is probably because the gravel size used by all the three investigators was less than 6 mm and even the shield's parameter values less than 0.05.

Therefore, in general, the new developed expression can be accepted to give satisfactory prediction of friction factor for intense load conditions.

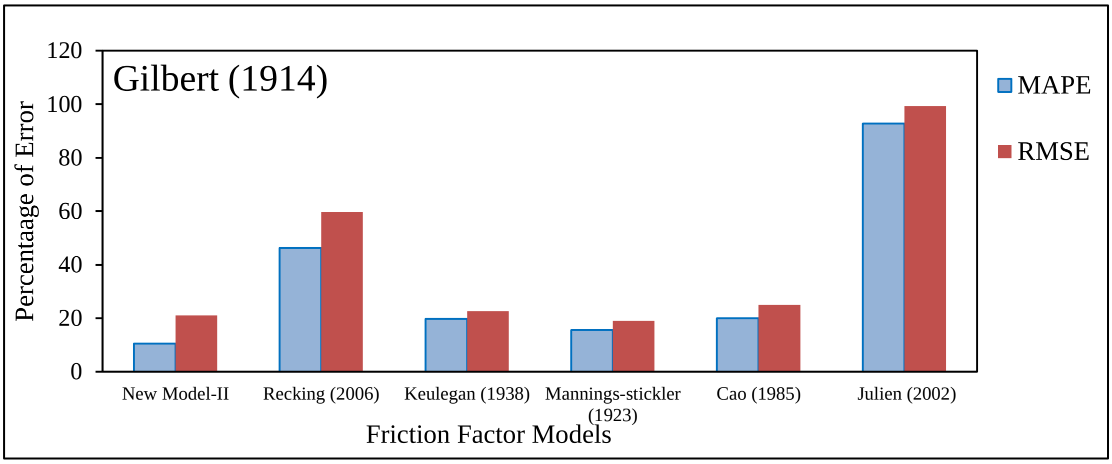

Fig. 5.11.a Error Analysis of friction factor model for intense load condition with Gilbert (1914) data

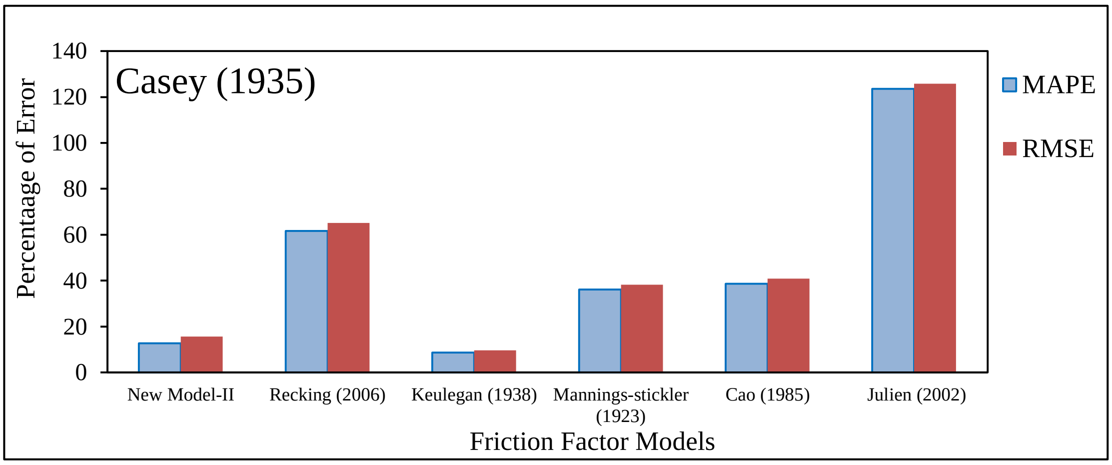

Fig. 5.11.b Error Analysis of friction factor model for intense load condition with Casey (1935) data

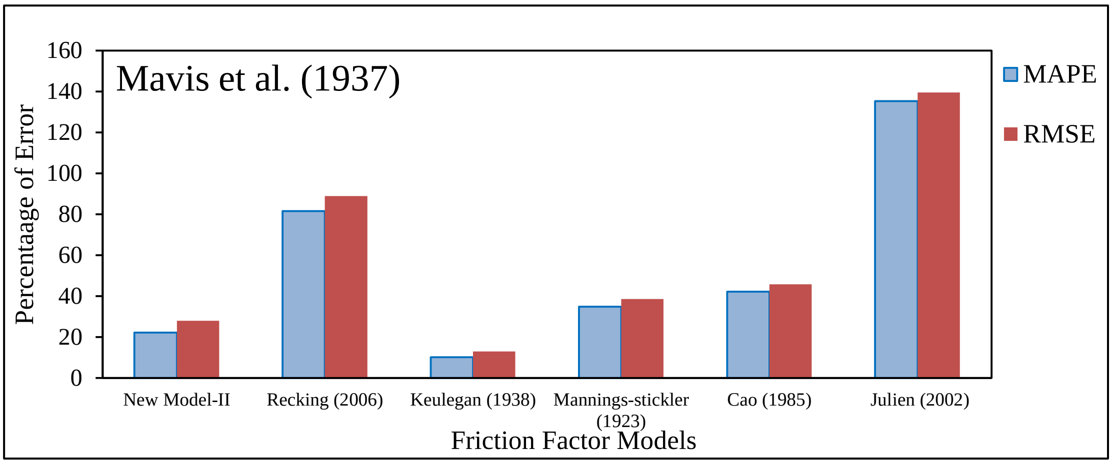

Fig. 5.11.c Error Analysis of friction factor model for intense load condition with Mavis et al. (1937) data

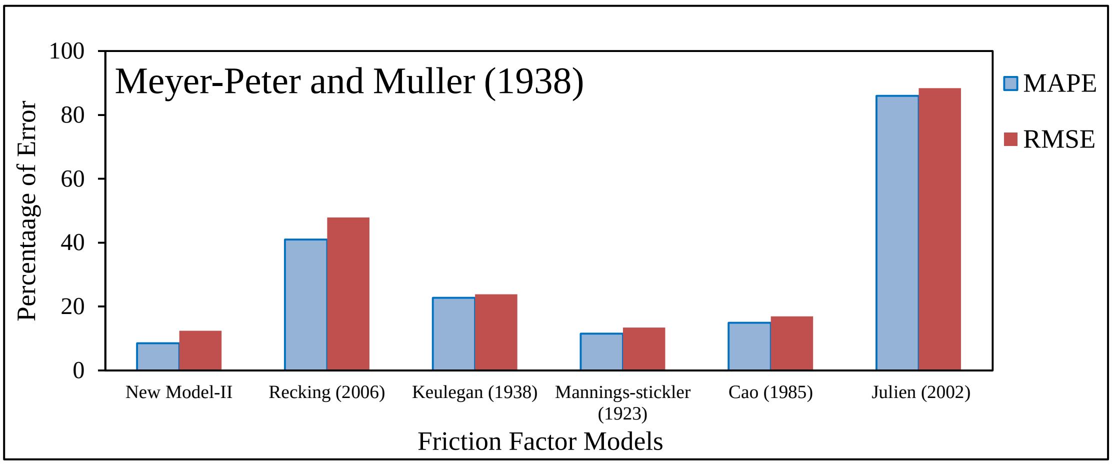

Fig. 5.11.d Error Analysis of friction factor model for intense load condition with Meyer-Peter and Muller (1938) data

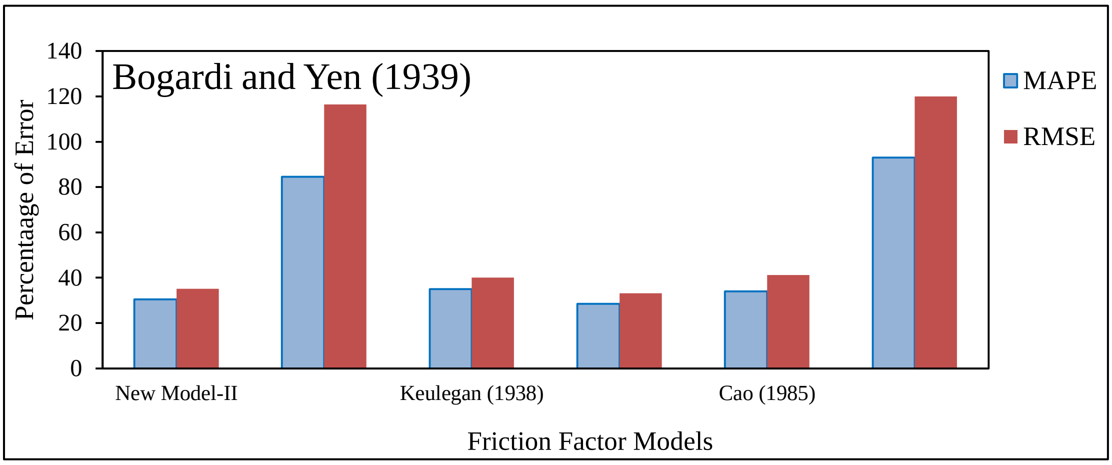

Fig. 5.11.e Error Analysis of friction factor model for intense load condition with Bogardi and Yen (1939) data

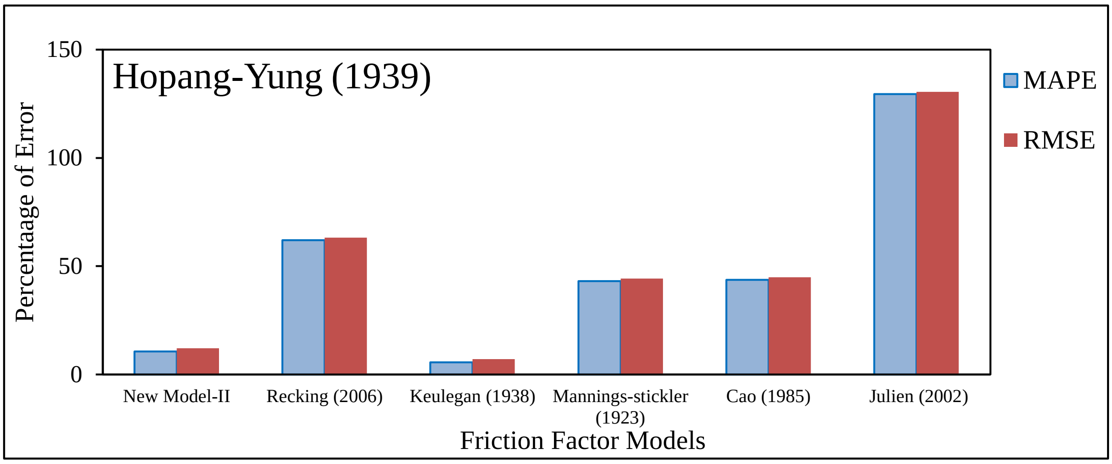

Fig. 5.11.f Error Analysis of friction factor model for intense load condition with Hopang-Yung (1939) data

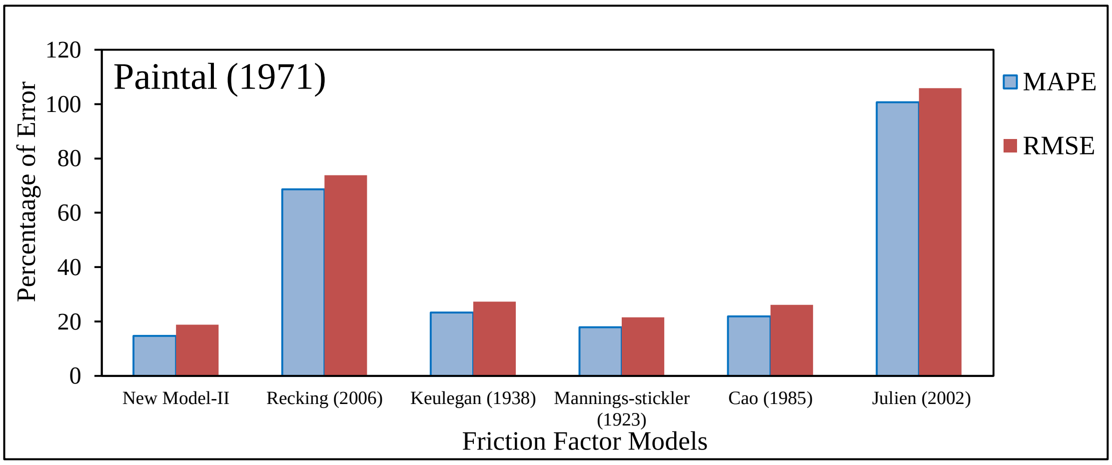

Fig. 5.11.g Error Analysis of friction factor model for intense load condition with Paintal (1971) data

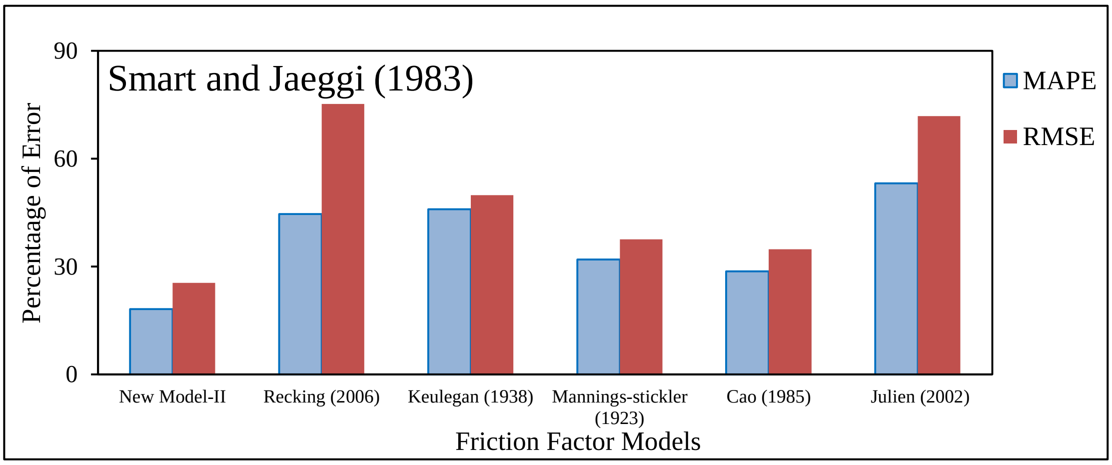

Fig. 5.11.h Error Analysis of friction factor model for intense load condition with Smart and Jaeggi (1983) data

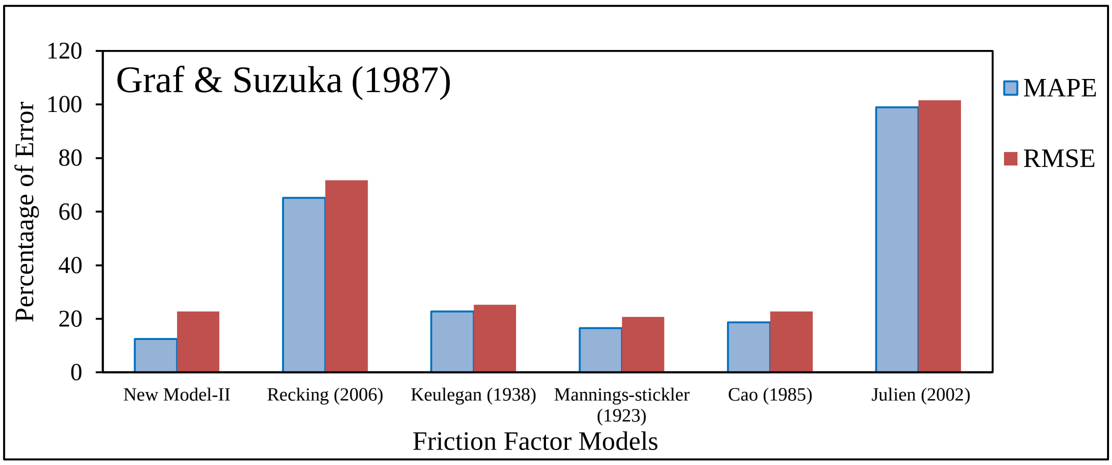

Fig. 5.11.i Error Analysis of friction factor model for intense load condition with Graf&Suzuka (1987) data

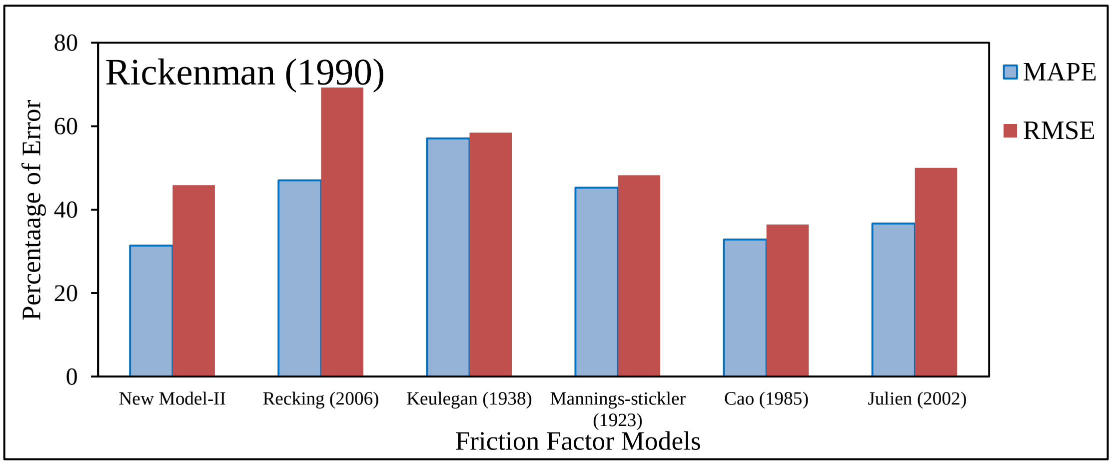

Fig. 5.11.j Error Analysis of friction factor model for intense load condition with Rickenman (1990) data

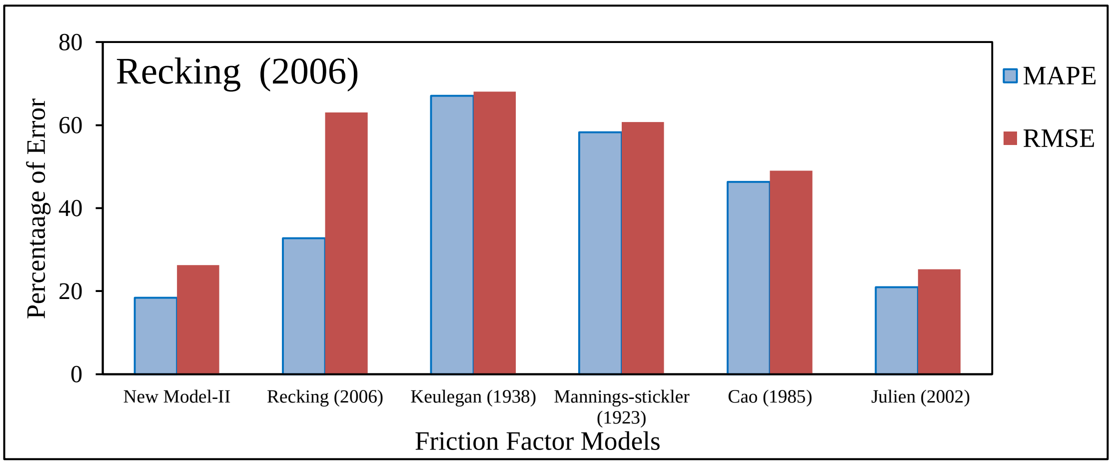

Fig. 5.11.k Error Analysis of friction factor model for intense load condition with Recking (2006) data

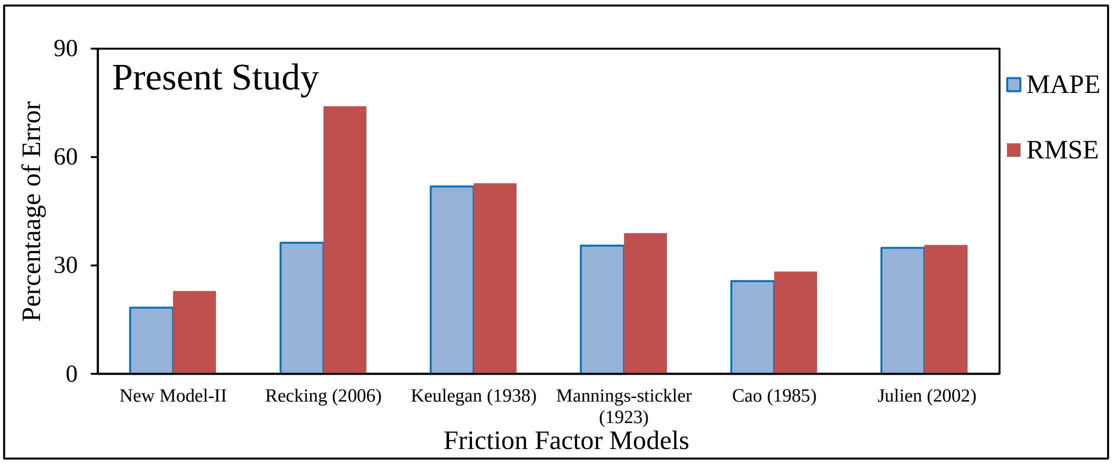

Fig. 5.11.l Error Analysis of friction factor model for intense load condition with Present Study data

5.4.8. Bed load Intensity for Intense load flow condition

5.4.8.1 Experimental results for intense load condition

Table 5.3 Experimental data sets of bed load intensity for intense load condition

Discharge Q (m^3/s)	Flow depth h (m)	Cross section area A (m^2)	Wetted perimeter P (m)	Hydraulic radius $R=A/P$	Mean channel velocity V (m/s)	Relative Depth (R/D)	$u/u*$	Shields parameter θ	Bed load Intensity Φ
0.031	0.10	0.075	0.934	0.081	0.405	12.41	9.11	0.141	0.102
0.029	0.095	0.071	0.919	0.078	0.394	11.85	9.06	0.137	0.095
0.026	0.092	0.068	0.910	0.075	0.384	11.50	8.97	0.135	0.090
0.023	0.086	0.063	0.893	0.071	0.368	10.90	8.84	0.129	0.082
0.021	0.08	0.058	0.876	0.067	0.352	10.25	8.71	0.124	0.074
0.018	0.074	0.054	0.859	0.062	0.336	9.59	8.59	0.119	0.066
0.016	0.07	0.050	0.848	0.059	0.324	9.14	8.49	0.115	0.061
0.014	0.065	0.047	0.834	0.056	0.304	8.57	8.21	0.109	0.052
0.012	0.06	0.043	0.820	0.052	0.290	8.00	8.13	0.104	0.048
0.011	0.057	0.040	0.811	0.050	0.279	7.64	7.98	0.101	0.042

In the present study, two parameters were very important one is shield's parameter, θ and another was intensity of bed load, Φ. To determine the shield's parameter, boundary shear stress had to be calculated. Thus to determine the boundary shear stress energy gradient method had been used. And another part of this present study was bed load intensity which was calculated with the help of bed load sediment transport rate. To measure the bed load sediment transport rate all the techniques already elaborated in chapter-3. All the remaining parameters were same and taken from the stage-discharge data of intense load conditions. For all the experimental runs, diameter of gravel was kept constant as 0.0065m. Moreover a constant bed slope of the channel (0.25%) was used for all the runs. Total 18 experimental runs for uniform flow conditions were taken for the study. Table 5.3 shown the experimental results.

A regression model is developed for bed load intensity as discussed in chapter 4. Graph between bed load intensity and Shield's parameter is plotted which provides with a power function relationship. The bed load intensity model by Recking (2006) for intense load condition was apparently valid for relative depth (*R/D*) ranging from 2 to 16.9, with θ values ranging from 0.10 to 27.The present model is developed for the range of *R/D* values less than 16.9 with data even lower than 2. The Shield's parameter for the present set of experimentation ranges from 0.1 to 0.15. In total, 77 data sets from the experimental

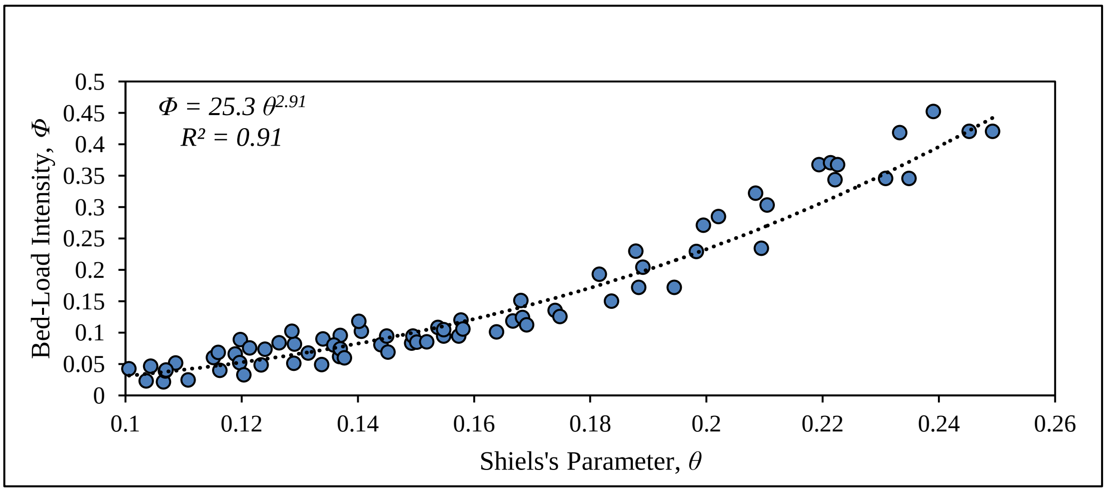

Fig.5.12. Bed load intensity variation for intense load condition

observation, including that of Recking (2006) which has given in appendix section have been utilized in developing the new model.

$$\Phi = 25.3\ \theta^{2.91} \qquad (R^2 = 0.91) \qquad (5.5)$$

Fig. 5.12 illustrates a graphical relationship between Shield's parameter, θ, and Bed Load Intensity, Φ for intense bed load condition. The power law curve shows a higher value of correlation coefficient, indicating a satisfactory relation between the two. The linear regression model developed for the determination of bed load intensity Φ is given as equation 5.5.

The model developed in the previous section is validated with the data sets of other researchers such as Wong Parker(2006a), Wong Parker (2006b), Ashmore (1988), Recking (2006) Graf and Suzuka (1987) etc. along with the present experimental observations. The number of datasets are253 in total. The performance of models suggested by previous investigators have also been analysed for the above datasets. Total datasets of other investigators are given in appendix.

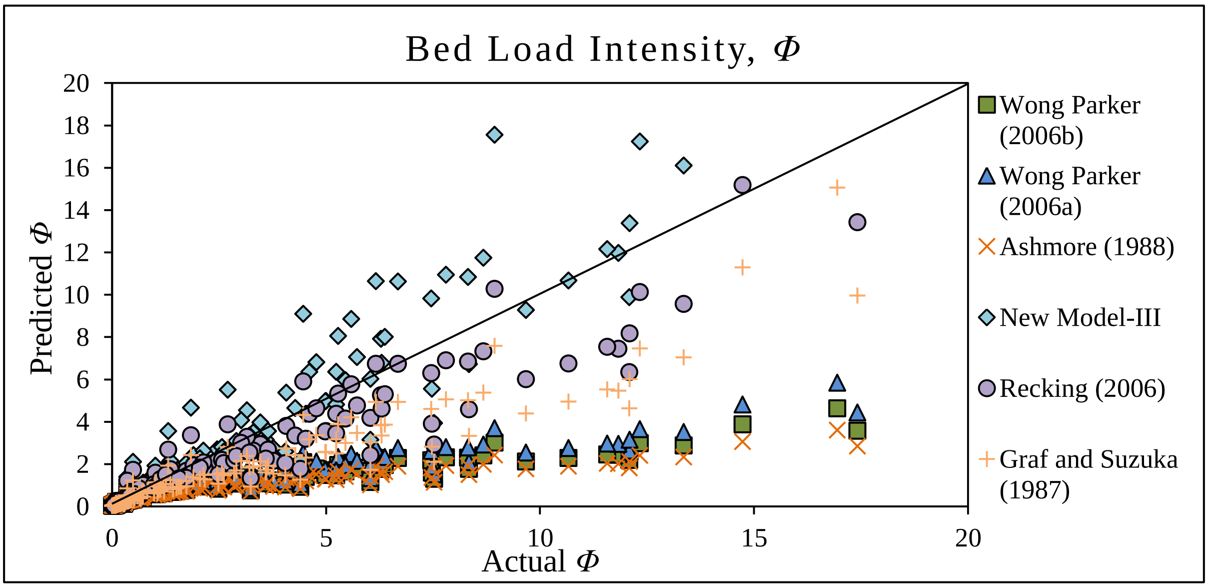

Fig.5.13. Validation of Bed load intensity model for Intense load condition with other author data

Fig. 5.13shows the performance of different models is predicting the bed load intensity, Φ for intense load condition for all the available data sets. The present model is observed to give better predictions.

5.4.8.2. Error Analysis of Bed load Intensity Model for Intense load Flow condition

Bed load intensity is calculated by using different models developed by the researchers and the values of MAPE and RMSE are demonstrated in fig.5.14 (a to f). The models developed are power function of the shield's parameters, θ which depend on the boundary shear stress.

Total six numbers of datasets are used for this analysis including the present experimental results with θ values less than 0.1, which indicates an intense load condition.

In the error analysis figures demonstrated below, it is clearly observed that the developed expression by the other authors in equation 5.5 gives lower values of MAPE and RMSE for all the available data. The values of RMSE and MAPE for the case of Meyer-Peter and Muller (1938), is quite high for all the prediction models, it's because of higher range of gravel size used i.e., 1.4 mm to 28.65 mm for a mild slope of 0.1 to 0.5.

The data sets of Smart and Jaeggi (1983), Rickenman (1990), and Recking (2006) have steep slopes while that of Gilbert and the present experimental data sets have mild bed slope. Hence, this might be an explanation for different ranges of values, where the former have the values in the range of 100 while the later gives better results with values below 35. Therefore it can be concluded, that all the bed load and intensity models are quite suitable for predicting bed load intensity, Φ for lower size of gravel weight with a mild slope, with the present expression provided the better results.

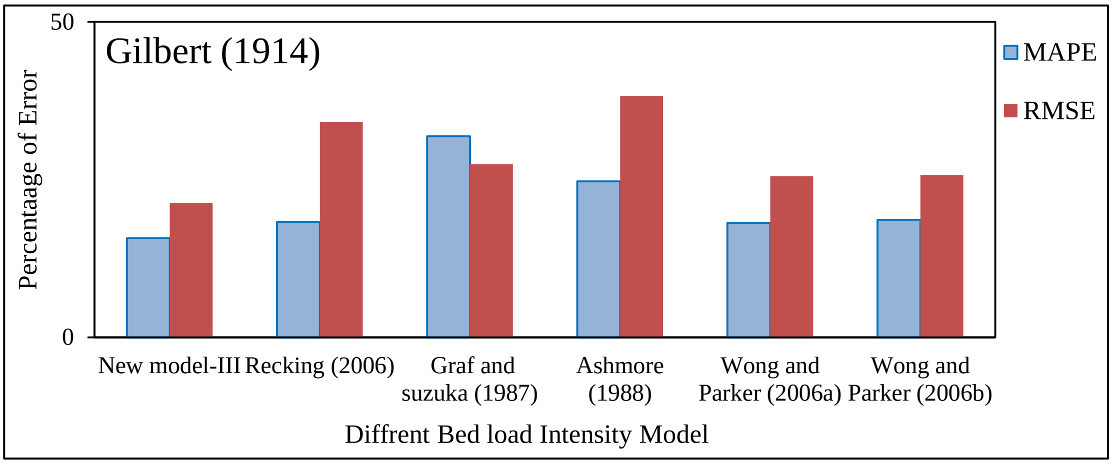

Fig. 5.14.a Error analysis of Bed load intensity model for Intense load condition with Gilbert (1914) data

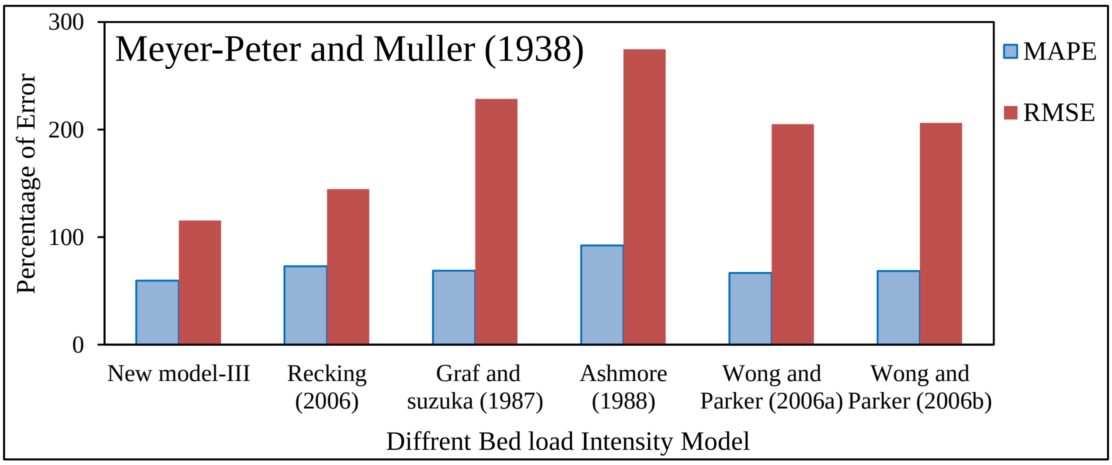

Fig. 5.14.b Error analysis of Bed load intensity model for Intense load condition with Meyer-Peter and Muller (1938) data

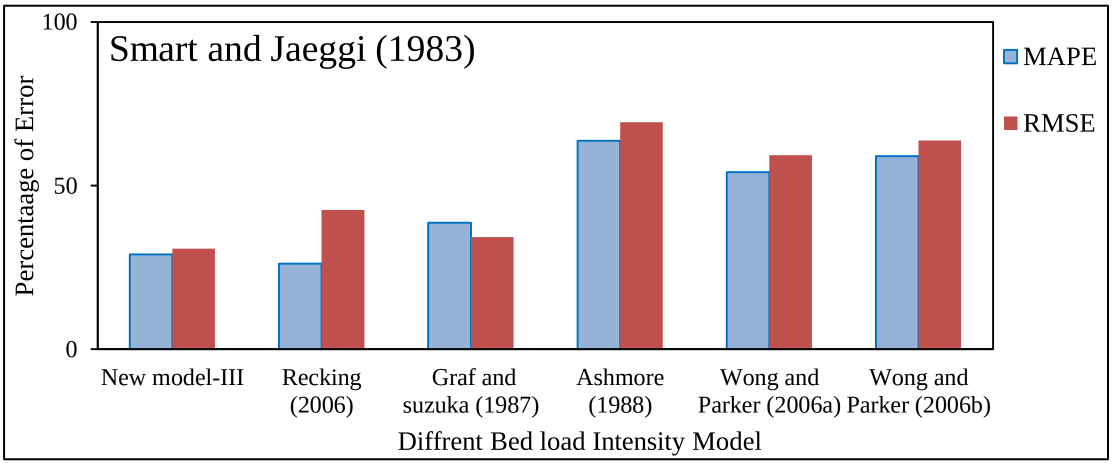

Fig. 5.14.c Error analysis of Bed load intensity model for Intense load condition with Smart and Jaeggi (1983) data

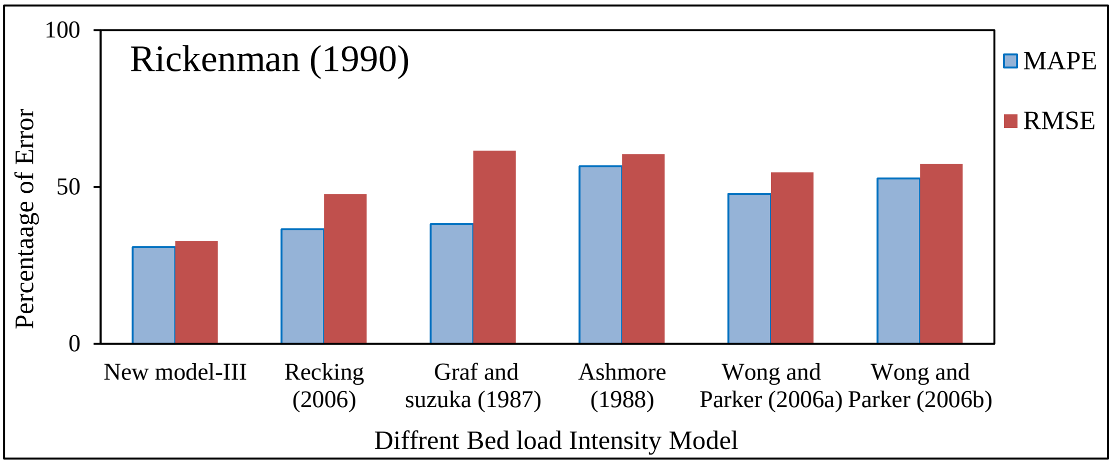

Fig. 5.14.d Error analysis of Bed load intensity model for Intense load condition with Rickenman (1990) data

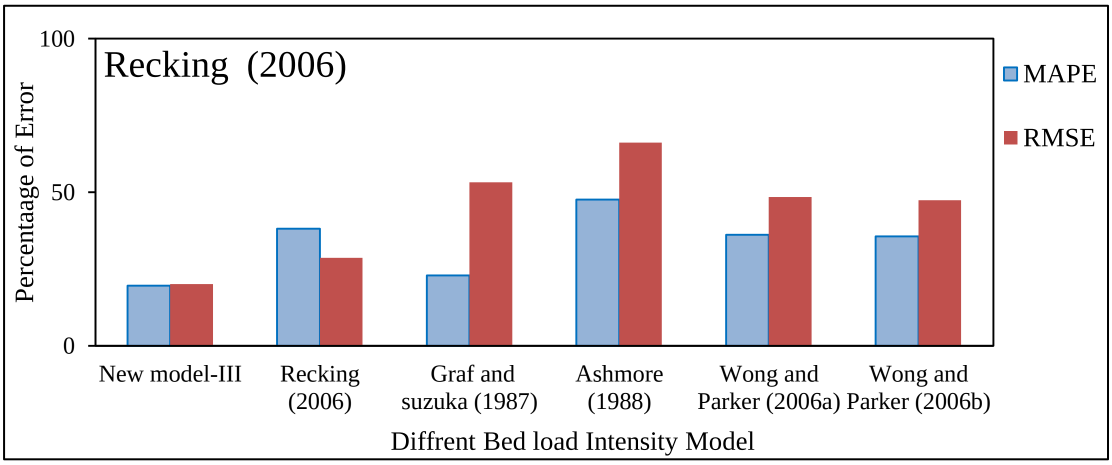

Fig. 5.14.e Error analysis of Bed load intensity model for Intense load condition with Recking (2006) data

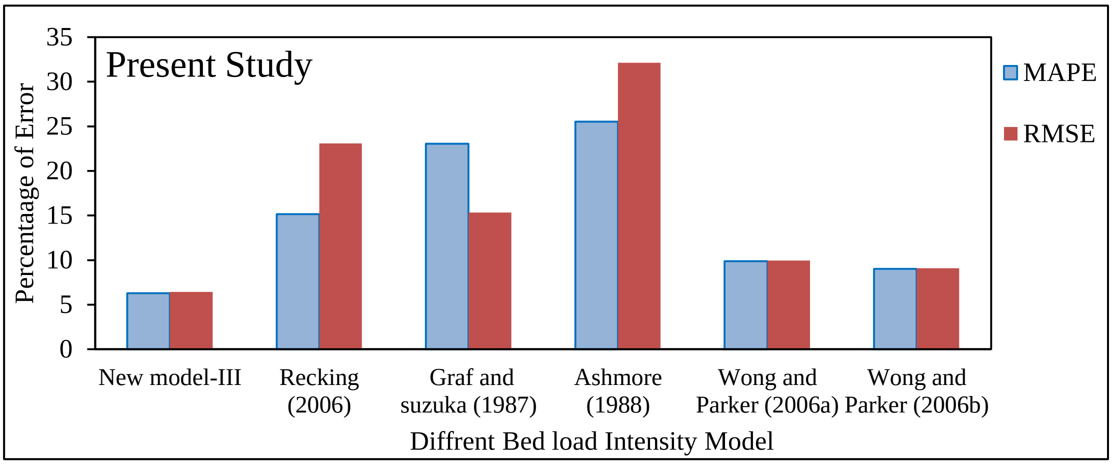

Fig. 5.14.f Error analysis of Bed load intensity model for Intense load condition with Present Study data

CHAPTER 6

CONCLUSIONS

CONCLUSIONS

Experiments were commenced in gravel bed open channels on a tilting hydraulic flume at the Fluid Mechanics Laboratory of National Institute of Technology, Rourkela, Odisha (India) for both no load and intense load conditions for various flow depths. Two types of sediment sizes are considered for the purpose of the present study. Following salient findings are obtained from the present research work.

- The local velocity towards the top surface are found to be increasing with increase in depth of flow, for the no load conditions. However, uniform distribution of boundary shear stress is observed over the bed of the channel, ANSYS-FLUENT predicted better results as compared to CES for all the flow depths.
- The developed model for friction factor in no load conditions provided less than 10% MAPE and RMSE values for almost all data sets. For the data sets of Cao (1985) and Recking (2006), the model provided 20% of MAPE and RMSE values, which were also significantly less values when compared with other models for those datasets.
- The developed model for friction factor in intense load conditions provided less than 15% to 20% MAPE and RMSE values for almost all data sets with respect to other models. For the data sets of Mavis et al. (1937), Rickenman (1990) and Bogardi and Yen (1939), the model provided with 25% to 35% MAPE and RMSE values, which were also significantly less values as compared with respect to other models for those datasets.
- The developed model for predicting bed load intensity gives comparably less values of MAPE and RMSE as compared to other bed load intensity models. The developed model gives less than 20% MAPE and RMSE values for all datasets, except for Meyer-Peter and Muller (1938) where its error is around 50% which is also comparable to other models for

that datasets. Some load intensity models gives acceptable errors for some of the datasets, whereas the developed model gives less and tolerable errors for all the datasets.

Suggestions for Future Research

The present research leaves a wide scope for the future investigators to explore many other aspects of gravel bed open channels. The friction factor and bed load intensity have been determined with limited data of flow discharges and depths. Wide range of flow discharges and depths could not be achieved due to limited pump capacity. It is expected that future investigators should carry on researches to find out the different hydraulic parameters at wide range of discharges and flow depths, channel bed slopes, and various grain size characteristics. Effect of boundary and side wall shear stress on flow hydraulics in gravel bed open channels need to be attended in the future research. Following work can be extended using fine and coarse sand as a transport material and thus modelling for same can be done under same procedure. But, while considering fine material aspect of suspended load has to be considered. The work can be extended for meandering, skewed and non-prismatic channel with gravel bed.

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APPENDIX

Table. 1 Data sets for intense load conditions which was used in this present study

Author	*W* (m)	*D* (mm)	ρ_s (kg/m^3)	*S* (%)	*Q* (m^3/s)	*R/D*	u/u^*	*H* (m)	θ	Run nos.
Gilbert (1914)	0.201, 0.305, 0.402	3.17, 4.94, 7.01	2650	0.6 to 2	0.005 to 0.03	6.38 to 16.79	9.01 to 12.5	0.02 to 0.16	0.05 to 0.18	80
Casey (1935)	0.4	1, 2.46	2650, 2810	0.49 to .51	0.001 to 0.009	8.88 to 16.05	10.49 to 13.09	0.01 to 0.04	0.02 to 0.04	13
Mavis et. al. (1937)	0.819	1.41 to 4.18	2660	0.2 to 1	0.002 to 0.41	6.39 to 16.82	9.69 to 14	0.009 to 0.07	0.02 to 0.09	108
Meyer-Peter and Muller (1938)	0.15, 0.35, 2	1.4 to 28.65	1250, 2680	0.1 to 2	0.001 to 1.64	6.06 to 16.69	9.13 to 11.93	0.01 to 0.45	0.049 to 0.25	31
Bogardi and Yen (1939)	0.3, 0.823	6.8, 10.3, 15.2	2610, 2630, 2640	1 to 2.3	0.01 to 0.06	4.24 to 10.87	6.56 to 12.28	0.03 to 0.14	0.04 to 0.11	44
Hopang-Yung (1939)	0.399	4.36, 6.01, 6.28	2700, 2660, 2660	0.3 to 0.5	0.016 to 0.033	13.81 to 15.16	12.02 to 13.33	0.07 to 0.11	0.027 to 0.042	5
Paintal (1971)	0.914, 0.919	7.95, 22.2	2650	0.4 to 1	0.07 to 0.25	5.94 to 13.91	7.95 to 12.13	0.09 to 0.21	0.02 to 0.04	20
Smart and Jaeggi (1983)	0.2	2, 4.2, 4.3, 10.5	2670, 2680	3 to 20	0.005 to 0.03	3.62 to 15.2	4.58 to 11.61	0.02 to 0.09	0.11 to 1.37	64
Cao (1985)	0.6	11.5, 22.2, 44.3	2650, 2570, 2750	0.5 to 9	0.015to 0.25	1.3 to 15.26	3.26 to 14.02	0.03 to 0.25	0.03 to 0.20	56
Graf&Suzuka (1987)	0.6	12.2, 23.5	2716, 2736	0.5 to 2	0.04 to 0.20	3.84 to 16.67	7.03 to 13.47	0.07 to 0.255	0.03 to 0.08	105
Rickenman (1990)	0.2	10	2680	7 to 20	0.01to 0.03	3.04 to 7.85	4.53 to 8.89	0.03 to 0.08	0.28 to 0.76	46
Recking (2006)	0.1, 0.15	2.3, 4.9, 12.5	2600	2 to 9	0.001 to 0.005	1.97 to 12.79	3.49 to 9.35	0.01 to 0.04	0.08 to 0.27	77

Table.2 Data sets for no load conditions which was used in this present study

Author	W (m)	D (mm)	ρ_s (kg/m^3)	S (%)	Q (m^3/s)	R/D	u/u^*	H (m)	θ	Run nos.
Paintal (1971)	0.914	7.95, 22.2	2650	0.4 to 1	0.02 to 0.22	4.14 to 8.15	8.67 to 10.91	0.05 to 0.18	0.018 to 0.042	18
Cao (1985)	0.6	11.5, 22.2, 44.3	2650, 2570, 2750	0.5 to 9	0.007 to 0.2	0.97 to 7.48	3.73 to 10.6	0.02 to 0.26	0.008 to 0.06	51
Graf&Suzuka (1987)	0.6	12.2	2716	0.7 and 1	0.039 to 0.059	0.77 to 0.98	5.93 to 7.58	9.89 to 10.32	0.02 to 0.04	5
Recking (2006)	0.1, 0.25	2.3, 4.9, 9	2600	1 to 5	0.0003 to 0.05	1.22 to 8.59	4.52 to 10.63	0.008 to 0.035	0.01 to 0.09	54

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