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Aims and Scope

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Contents

Flow Characteristics Of A Triangular Sharp Crested Weir

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A B S T R A C T

Triangular weir is a simple form of weir best matched for low discharge and is free from aeration difficulties. It is mostly used in various branches of engineering like hydraulics, environmental, chemical and irrigation for the purpose of discharge measurement. Earlier studies conducted on triangular weir indicate that the discharge coefficient related to head or head to weir height ratio covering a limited range of head and vertex angles. Further, no generalized equation proposed to compute either discharge coefficient or discharge for any head and vertex angle. In this study, a total of 160 experimental runs were taken for five weir vertex angles (from 30° to 90°) at apex elevations of16,18,and 20cm. Using the general formula for triangular weir dimensionless discharge and dimensionless head has been defined that helps in merging all the data points of five angles to one single curve. A generalized equation between dimensionless discharge and dimensionless head has been obtained. The maximum error obtained in the discharge computed from this equation is ±7%. This equation also validates the data of previous study (Wahaj, 1999).

Keywords: Weir vertex angle, Discharge coefficient, Dimensionless discharge, Dimensionless head, generalized equation.

1. Introduction

A weir is constructed across a river (or stream) in order to increase the level of water on the upstream side and to allow the excess water to flow over its entire length to the downstream side. The triangular weir is used widely for computing the flow of liquids in flumes and open channels.

Thomson (1858) recommended $C_d = 0.593$ and 0.617 for the 90° and 127° notch angles respectively. Barr (1910) concluded that the coefficient was increased by roughness and projections on the upstream face of the weir. Barr also concluded that the coefficient was independent of channel width if the width was at least eight times the head. Strickland (1910) quoted formula for 90°-notch weirs on the basis of Barr's experiments as

$$C_d = 0.566 + \frac{0.0157}{\sqrt{h}} \qquad (1)$$

Cone (1916) proposed a general formula for all notch angles between 28° and 109° given by:

$$C_d = \frac{0.576}{h^j} + \frac{0.00584}{Sh^j} \quad (2)$$

Greve (1932) gave a formula for C_d as,

$$C_d = \frac{0.585}{((\tan\theta/2)^{0.004}) \times h^{0.03}} \quad (3)$$

Lenz (1943) derived a general formula for the coefficient of discharge,

$$C_d = 0.560 + \frac{M}{R^n \times W^m} \quad (4)$$

Govinda and Muralidhar (1963-64) established a correlation between the discharge coefficient and the wetted perimeter of the notch portion.

For ¼-90° V-notch,

$$\frac{1}{C_d} = 1.28 + \frac{8.326}{P} \quad (5)$$

$$\frac{1}{C_d} = 1.70 + \frac{0.0414}{R} \quad (6)$$

For 1/2-90° V-notch,

$$\frac{1}{C_d} = 1.715 - \frac{0.449}{P} \quad (7)$$

$$\frac{1}{C_d} = 1.715 - \frac{0.04467}{R} \quad (8)$$

For 90° V-notch,

$$\frac{1}{C_d} = 1.733 - \frac{0.4945}{P} \quad (0<P<28cm) \quad (9)$$

$$C_d = 0.5619 + 0.000652P \quad (P>28cm) \quad (10)$$

Numachi Et.al (1940) gave a general equation for the coefficient C_d,

$$C_d = 0.574 + \frac{0.0055}{h} + \left(0.055 + \frac{0.278}{0.886+p}\right) [\frac{h}{B} - (0.1 - \frac{0.0098}{0.079+p})]^2 \quad (13)$$

King (1954) developed the following formulas:

$$(90° \; notch) C = \frac{0.589}{h^{0.03}} \tag{14}$$

$$(60° \text{ notch}) C = 0.595 \, h^{0.01} \tag{15}$$

$$(22.5° \text{ notch}) C = \frac{0.586}{h^{0.07}} \tag{16}$$

Kindsvater-Shen (1980) developed a formula for the discharge over a triangular weir with angles notch angles between 20o and 100°, given by

$$Q = C_e \, {}^{8}/_{15} \sqrt{2g} \; tan \, \theta/2 \; (h_e)^{5/2} \tag{17}$$

Capetillo et.al (2013) developed a discharge coefficient equation for sharp crested triangular weirs on the basis of free vortex theory as described by Bagheri and Heidarpour (2010); The equation is as follows:

$$Q = 2 \, V_b R_b \tan(\theta/2) \; [kY + R_b \ln \frac{R_b}{k\,Y + R_b}] \tag{18}$$

From the literature surveyed above, it is clear that the coefficient of discharge is given for individual angles and most of the investigators related C_d with h and some of them related it with the wetted perimeter, Reynold's number, and Weber number. In the present study, an attempt has been made to compute discharge covering a wide range of notch angles and heads. This study aims at developing a generalized equation for the discharge through a triangular weir and to establish a relationship between the discharge coefficient, notch angle, *h/p* and *p/B*.

1.1 Governing equation:

The discharge through a triangular weir is given by:

$$Q = \frac{8}{15} C_d \sqrt{2g} \; tan \frac{\theta}{2} \; h^{5/2} \tag{19}$$

Where Q is the discharge (m^3/s); Θ is the notch angle; h is the head above the crest (m); C_d is the discharge coefficient (dimensionless).

2. Experimental Set-Up

The experiments were conducted in a horizontal, rectangular (75 cm wide and 53 cm deep), prismatic glass walled channel having cement plastered bottom (Photo 1). Weirs were made of G.I. sheets. Weirs were installed at a distance of 8.5 m from the upstream end of the channel. A sharp-crested triangular weir was installed at a desired angle, Θ and apex height p. Discharge was controlled by means of a control valve. The flow was allowed in the set-up to fill the upstream channel up to the apex level of the triangular weir. The apex level was recorded with a point gauge of accuracy 0.1 mm. Five such readings were taken and averaged to obtain a precise value of gauge reading. Head over the apex was obtained by subtracting the apex level from averaged free surface reading. The discharge was changed by means of the control valve and a number of runs were taken to cover a wide range of *h/p*. The entire procedure was repeated for other weirs having different apex angles. Table - 1 gives an account of the different parameters of the triangular weir taken into consideration in this study. A total of 109 experimental runs were taken.

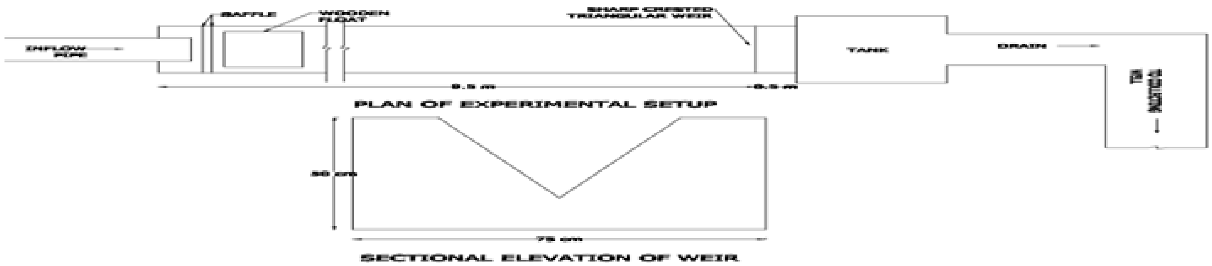

Fig. 1: Experimental Setup

Photo 1: Front view of the channel

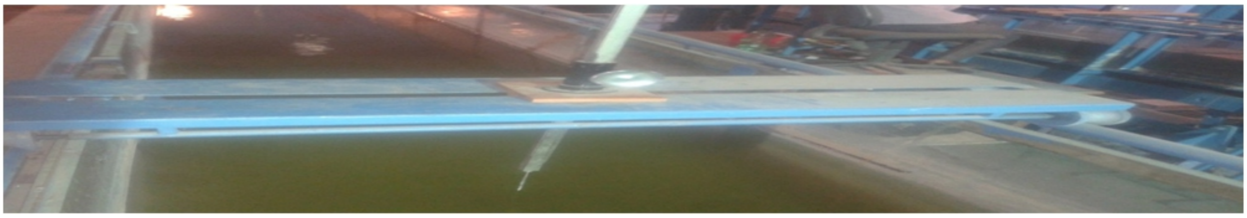

Photo 2: Point Gauge

Table : Range of parameters for the triangular weir

θ *(degrees)*	*p* (cm)	*h* (m)	Q_o *(m3/s)*	F_r	No. of runs
30	16	0.2135-0.2894	0.0081-0.0168	0.00083-0.00143	6
	20	0.177-0.2605	0.0051-0.013	0.00197-0.00526	5
45	16	0.1847-0.2768	0.008-0.022	0.003022-0.008784	7
	20	0.1718-0.2466	0.0076-0.0184	0.004399-0.009335	6
60	16	0.1563-0.2663	0.0078-0.0293	0.017668-0.043014	15
	20	0.148-0.2355	0.007-0.022	0.013796-0.031285	8
75	16	0.142-0.2527	0.0065-0.036	0.015741-0.05541	14
	20	0.1305-0.2469	0.0075-0.033	0.012354-0.039578	21
90	16	0.1385-0.2218	0.01-0.036	0.024628-0.062062	13
	20	0.108-0.2233	0.0065-0.0356	0.02045-0.050483	14

3. Analysis Of Data And Results

3.1 Variation Of Discharge With Head:

To study the influence of an increase in the notch angle of a triangular weir on the discharge coefficient, curves have been plotted between Cd and h/p for different angles (say 30, 45, 60, 75, and 90), keeping the weir vertex heights as constant. Three different weir vertex heights (16, 20 cm) have been used.

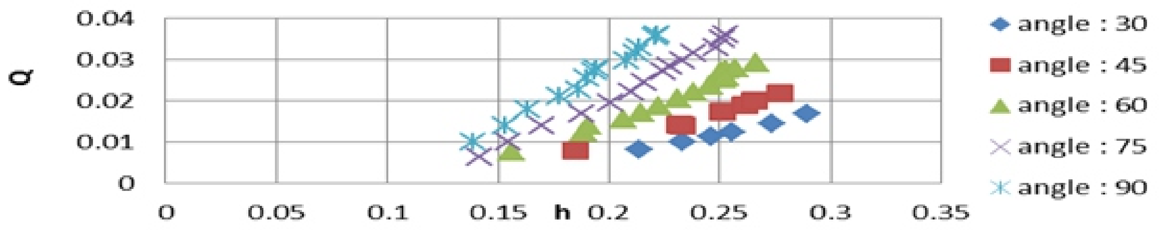

Fig. 7: Q v/s h (p = 16 cm)

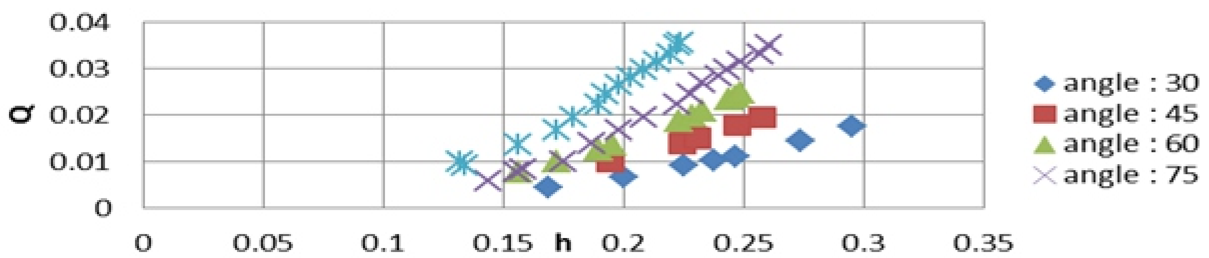

Fig. 8: Q v/s h (p = 20 cm)

These graphs show that there is always an increasing trend for the discharge 'Q' with the head above the weir crest 'h' for increasing values of notch angles. The variation of discharge 'Q' with head 'h' shows an increasing trend with the increase in notch angle. However the RMSE values are small enough to obtain a generalized equation with this method.

3.2 Variation Of Discharge Coefficient With Notch Angle:

To study the variation of discharge with the notch angle for three different weir vertex heights, graphs have plotted showing the variation of Q with h for angles 30°, 45°, 60°, 75°, 90° and for p = 16, 20 cm.

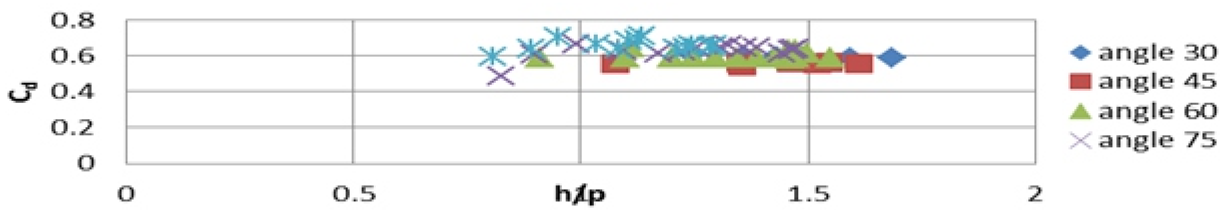

Fig. 10: C_d v/s h/p (p = 16 cm)

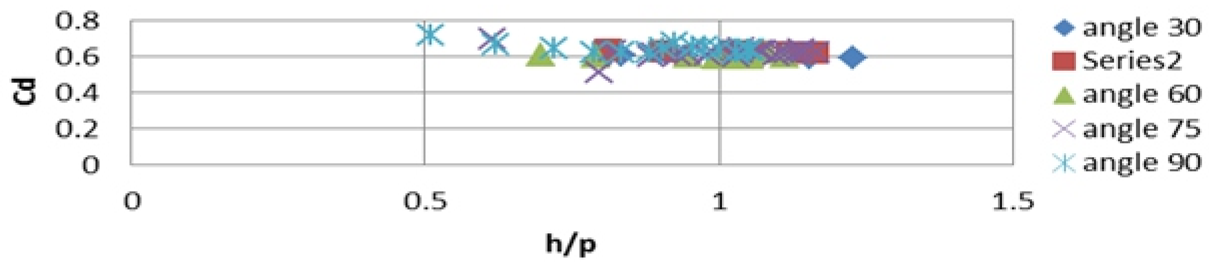

Fig. 11: Cd v/s h/p (p = 20 cm)

The above graphs show that the value of C_d decreases as the angle is changed from 30° to 45°. If the value of notch angle is further increased from 60° to 90°, the values of C_d starts increasing. This variation is noticeable in the lower range of h/p (i.e., 0.5 to 1). In the higher range of h/p, the variation in C_d is insignificant.

Rewriting Eq.4.1 , both sides are changed to dimensionless quantities and can be expressed as:

$$Q_n = C_d(H_n)^{5/2}$$ Where,

$$Q_n = \frac{Q}{\frac{8}{15}p^{5/2}\sqrt{2g}\tan\frac{\theta}{2}}$$

$H_n = h/p$

The data obtained from the experimental work is converted in the form of above mentioned dimensionless discharge Q_n and dimensionless head H_n and graphs are plotted between Q_n and H_n as shown in figure 5.23:

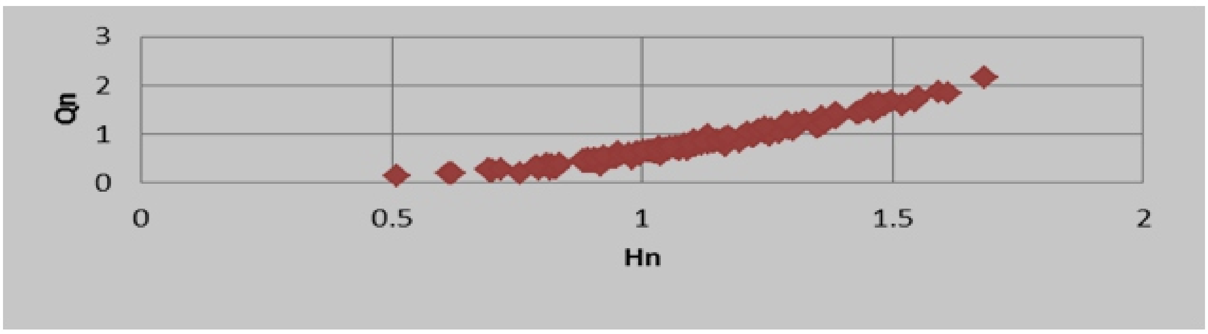

Fig. 13: Q_n v/s H_n

It is clear from the above figure that the entire data for all notches having different apex heights merge to a single curve. The generalized discharge equation so obtained is given by:

$$Q_n = 0.6016(H_n)^{2.4759} \tag{2}$$

Using above generalized equation, dimensionless discharges were computed for the entire data set obtained in experimental programme and % error in discharge was computed using following relation:

$$e = \frac{(Q_{nc} - Q_{no})}{Q_{no}} \times 100$$

Average % error can be computed as:

$$e_{avg} = \frac{1}{N} \sum_{i=1}^{N} e_i$$

Fig 14 shows the agreement diagram between Q_{no} and Q_{nc}. It is evident from this figure that most of the data points lies within an error band of $\pm 6\%$.

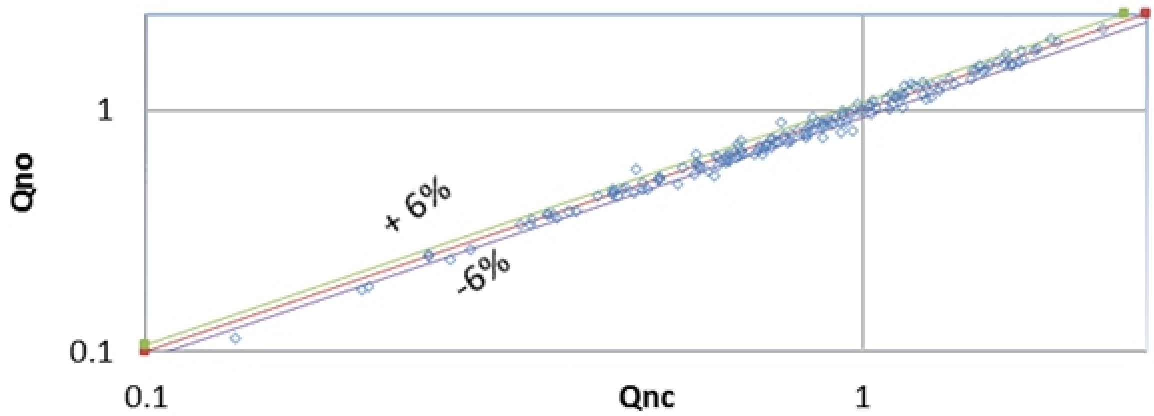

Fig. 14: Q_{no} v/s Q_{nc}

Validation of the generalized discharge equation:

For the purpose of validation, the data of Wahaj (1999) for angles 28°, 45°, 60° and 90° collected for the weir height 15 cm has been taken. Since the generalized equation developed in this study covers a range of heads varying from 14 to 30 cm, the data of Wahaj (1999) lying in this range has been chosen for validation.

For this purpose, the data of Wahaj (1999) has been converted to non-dimensional parameters Q_n and H_n, and then Q_{nc} has been calculated from the generalized equation obtained using Wahaj's data.

The Q_{no} obtained from Wahaj's data and Q_{nc} obtained from the generalized equation has been compared. The agreement diagram between Q_{no} and Q_{nc} has been shown in Fig. 5.25. A perusal of Fig. 5.25 indicates that most of the data points lie within an error band $\pm 6\%$.

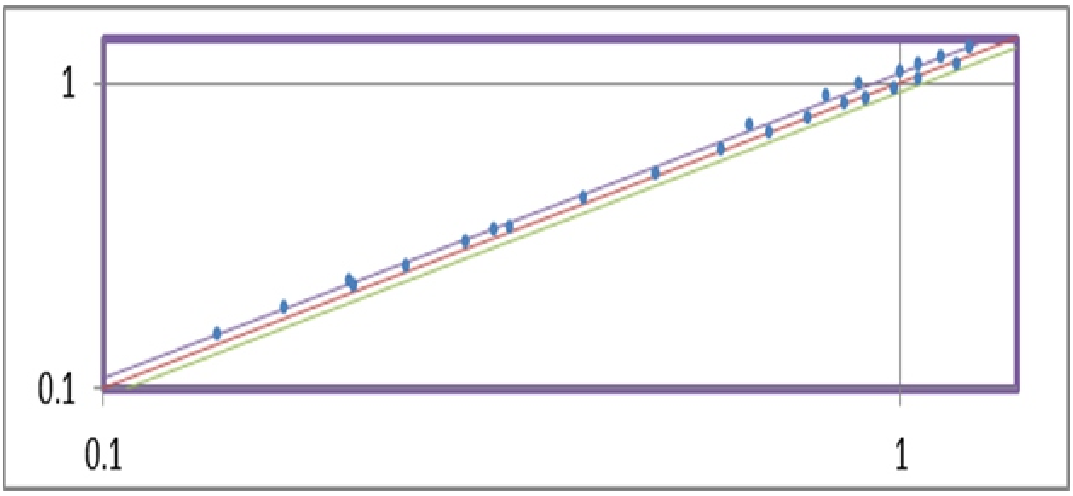

Fig5: Qno v/s Qnc (for the data of Wahaj (1999))

Conclusions

- A generalized equation for the discharge over a triangular weir has been obtained. This equation can be used for angles from 30° to 90° and for weir heights 16, 20 cm.
- The variation of C_d with h/p shows a decreasing trend for angles 30°, 45° and 60°. Whereas there is a increasing trend for angles 75° and 90°.
- There is a slight variation in the value of C_d with an increase in the value of p/B. Thus the effect of different weir heights on the discharge coefficient of a triangular weir is insignificant.
- The discharge coefficient for a triangular weir decreases as the notch angle is increased from 30° to 45°. With further increase in the notch angle (i.e., from 60° to 90°), the value of C_d starts increasing. There is not much variation in C_d with the change in weir height 'p'. So the effect of 'p' on C_d is negligible for this range of weir heights (from 16cm, 20cm).
- The generalized equation obtained in present study successfully validates the data of Wahaj (1999).

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Sensitivity Analysis Of Parameters Governing Bed-Material Load Transport

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A B S T R A C T

Bed-material load transport process comprises of different parameters of different nature such as hydraulic, bed characteristics, geometric parameters of the channel and the nature of sediment particles. Hence it is important to know the influences of these parameters on transport process. Based on the input significant techniques through ANN model, it has been found that hydraulic parameters are influencing more the bed-material load transport. The most influencing parameter is flow discharge and least is the sediment gradation coefficient. This shows the importance of determining the hydraulic parameters more correctly. The variability of with different parameters is also important to have a proper understanding of the bed-material load transport process. Based on the ANN model, it has been discussed in the present work.

Keywords: alluvial channel; bed-material load; bed shear stress; input significance; sensitivity analysis; neural network

1. Introduction

Establishing sediment transport in alluvial channels is of vital importance to both sedimentological and engineering studies. Transport of sediment particles is commonly divided into suspended load and bed load. By definition, suspended load is material that is buoyed by the water motion and bed load is the material that moves near the bed by traction and saltation. By definition, the bed-material load is the sum of bed load and suspended load. The bed-material load transport in alluvial rivers is the principle link between river hydraulics and river form (Parker, 1978; Leopold, 1994; Gomez, 2006) and is responsible for building and maintaining the channel geometry (Almedeij and Diplas, 2003; Goodwin, 2004). It provides the major process linkage between the hydraulic and material conditions that govern river-channel morphology and knowledge of transport is required not only to elucidate the causes and consequences of changes in fluvial form but also to make informed management decisions that affect a river's function. There are two general approaches to the determination of total load. The first is to compute bed load and suspended load separately and then add them. A number of sediment transport models designed to describe bed load and suspended load have been formulated e.g. Meyer-Peter &

Müller (1948), Bagnold (1966), Einstein (1950), Yalin (1963), Chang et al., (1967). Some of the many other transport models which can be mentioned in this context are: Engelund & Hansen (1967), Ackers-White (1973), Engelund & Fredsøe (1976) and van Rijn (1984). The second is to determine bed material load directly such as Velikanov (1954), Acaroglu (1968), Graf (1971), Karim and Kennedy (1990), Yang (1996), Yang and Lim (2003). Depending on the conditions under which data are collected, the same formula could have different ratings of accuracy and also do not agree with each other and observed data and also it is extremely difficult for an engineer to select appropriate one for a given river (Vanoni, 1971; Yang, 1988). As bed-material load transport is an immensely complex process and the expression of the transport process through a deterministic mathematical framework may not be possible in the foreseeable future. In parallel with research into sediment transport has been the emergence of new modeling paradigms such as data mining (DM). This has opened up new opportunities for modeling processes about which the level of available knowledge is too limited to put the relevant information in a mathematical framework. DM is presently being utilized in almost all branches of science as an alternative and complementary to the more traditional physically-based modeling system. Use of artificial neural networks (ANN) remains in the forefront of this complementary modeling practice. ANN modeling has been used in a wide variety of applications. Neural network techniques have been used to study several hydrologic and hydraulic phenomena including water quality, stream flows, rainfall, runoff, sediment transport, and to infill missing data (Tatang et al, 1997; Haas, 1998;Govindaraju, 2000). ANNs have also been successfully applied to the sediment discharge prediction by Nagy et. al. (2002) and Cigizoglu (2002). Caamaño et. al. (2006) has used ANN technique to derive the bed load sediment transport formula. Doğan et. al. (2007) has applied ANN model to predict total load. The primary application of ANNs involves the development of predictive models to forecast future values of a particular response variable from a given set of independent variables. Recent studies have provided a variety of methods for quantifying and interpreting the contributions of the variables in the neural networks (Olden and Jackson, 2002; Gevrey et al., 2003). As said earlier, a proper understanding of the parameters governing the transport process is of paramount importance in the planning and design of hydraulic structures in alluvial settings.

1.1 Data Structure

Brownlie (1981) has compiled by far one of the most comprehensive documentation of existing flume and field data in this area. Present work is based on the data compiled by Brownlie (1981). Sediment load is the material being transported. The sediment load can be divided into wash load and bed-material load. The wash load is not considered to be dependent on the local hydraulics of the flow. The bed-material load is the material of sizes which are found in appreciable quantities on the bed, The bed-material load can be conceptually divided into the bed load (that portion of the load that moves near the bed) and the suspended load (that portion of the load that moves in suspension), although the division is not precise. Table 1 shows the source of the data used (1767 in numbers) in the study and their ranges. It consists of both field and flume type data. Data source is not cited in the present manuscript, as it is already exist in Brownlie (1981). Variables to characterize the bed-material load transport are as follows:

- Channel geometry – b (width of the channel), y (flow depth) and BF (bed form of the channel)
- Dynamic properties – Q (Channel discharge), S_f (friction/energy slope), τ_b (bed shear stress) and $_ct$
  (critical shear stress or shields shear stress)
- Sediment properties – d (mean size of sediment), G_s (gradation coefficient of the sediment particles) and G_s (specific gravity)
- Fluid properties – ν (viscosity)

So it can be said that bed-material load transport (C) is a function of all the above parameters or in other

$$C = f(b, y, \mathrm{BF}, Q, S_f, \tau_b, \tau_c, d, \sigma, G_s, \nu) \tag{1}$$

Table 1: Descriptive Statistics of the Observations Used in the Analysis

	Q	b	y	Sf	d	τ_b	τ_c	σ	G_s	ν	C
	(m^3/s)	(m)	(m)		(m)	(N/m^2)	(N/m^2)			m^2/s	ppm by mass
Mean	16.02	8.30	0.31	4.51E-03	1.25E-03	4.50	0.88	1.40E+00	2.65E+00	1.01E-06	3284.94
Standard Deviation	68.12	28.41	0.64	4.84E-03	2.54E-03	5.26	2.05	4.05E-01	3.36E-02	1.24E-07	5991.90
Minimum	5.95E-04	0.13	0.01	4.51E-05	1.90E-04	0.26	0.12	1.00E+00	2.25E+00	6.49E-07	3.71
Maximum	486.82	162.43	4.30	2.75E-02	2.70E-02	51.09	21.80	3.85E+00	2.68E+00	1.58E-06	49938.00
Count	1767	1767	1767	1767	1767	1767	1767	1767	1767	1767	1767

2. Input Significance Testing

A variety of methods are available for the estimation of the contribution of predictor variables in relationship to the output. For example, Partial Derivative method provides a profile of the output variations for small changes of each input variable and classification of the relative contributions of each variable to the network output. 'Stepwise' method is based on the classical stepwise approach that consists of adding or rejecting step by step one input variable and noting the effect on the output results (Gevrey et al., 2003). 'Profile' method proposed by Lek et al. (1996) studies each input variable successively when the others are blocked at fixed values. In the neural network, the connection weights between neurons are the linkages between the input and the output of the network, and therefore are the determined by applying Levenberg-Marquardt neural networks. The final architecture of neural net used in the analysis is 11-16-1. The transfer function used in the hidden layer is tan sigmoid and at the output layer is tan sigmoid. The maximum epochs has been set to 5000. The idea behind choosing sigmoid functions as transfer functions is that it bears a greater resemblance to the biological neurons. In case of sigmoid functions, the output of the neurons varies continuously but not linearly with the input. Results of neural modeling are shown in Figure 1

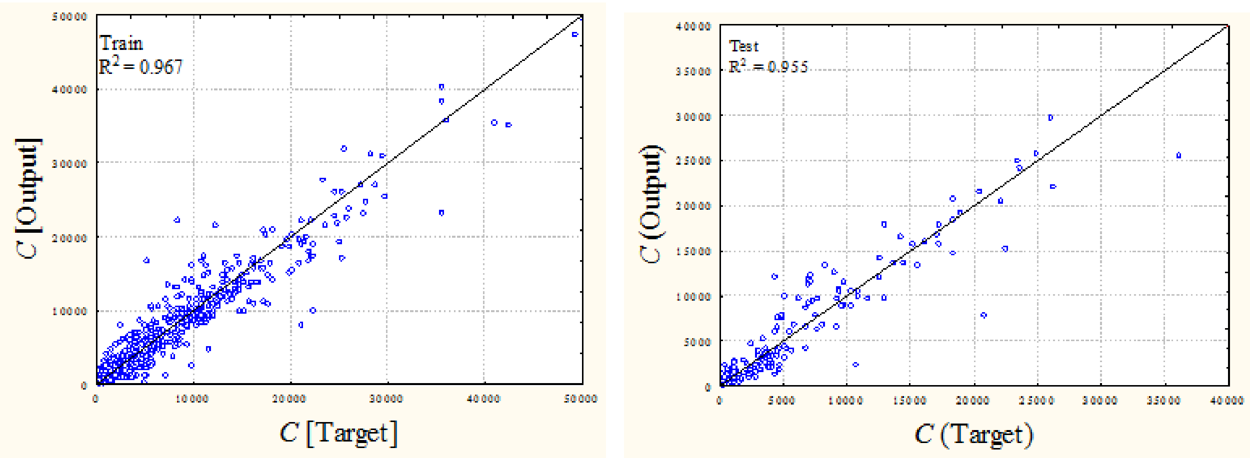

Figure 1: Results of ANN model (Training and Testing)

It can be clearly seen from Figure 1 that the linear coefficient of correlation is very high between observed data and values predicted through neural nets and it is 0.967 and 0.955 in training and testing. This shows the learning and generalization performance of the network is good. Garson algorithm or 'Weights' method includes partitioning the connection weights to determine the relative importance of the various inputs. In the present work, two such methods have been implemented to know the importance of the input variables on output.

Connection weights - Calculates the product of the raw input-hidden and hidden-output connection weights between each input neuron and output neuron and sums the products across all hidden neurons (Olden and Jackson, 2002). In the neural network, the connection weights between neurons are the links between the inputs and the outputs, and therefore are the links between the problem and the solution. The relative contributions of the independent variables to the predictive output of the neural network depend primarily on the magnitude and direction of the connection weights. Input variables with larger connection weights represent greater intensities of signal transfer, and therefore are more important in the prediction process compared to variables with smaller weights.

Garson's algorithm - Garson (1991) proposed a method for partitioning the neural network connection weights in order to determine the relative importance of each input variable in the network. The methodology for this algorithm is as follows:

(a) For each hidden neuron h, divide the absolute value of the input-hidden layer connection weight by the sum of the absolute value of the input-hidden layer connection weight of all input neurons, i.e.
For h =1 to nh, For i=1 to ni ,

$$A_{ih} = \frac{|W_{ih}|}{\sum_{i=1}^{ni} |W_{ih}|} \tag{3}$$

(b) For each input neuron i, divide the sum of the Aih for each hidden neuron by the sum for each hidden neuron of the sum for each input neuron of Aih, multiply by 100. The relative importance of all output weights attributable to the given input variable is then obtained.

For i: 1 to ni

$$RI\,(\%)_i = \frac{\sum_{n=1}^{nh} A_{ih}}{\sum_{n=1}^{nh}\sum_{i=1}^{ni} A_{ih}}\, X\,100 \tag{4}$$

Connection weight and Garson's algorithm are based on the weight matrices of the neural network.

Table 2: Input Significance test of the Variables

Variables	Connection Weights	Garson's algorithm	Ranking
Q	3.188	0.169	1
y	3.105	0.165	2
d	2.703	0.144	3
τ_b	2.214	0.118	4
τ_c	2.095	0.111	5
S_f	1.957	0.104	6
BF	1.191	0.063	7
b	0.946	0.05	8
v	0.729	0.039	9
G_s	0.469	0.025	10
σ	0.21	0.011	11

Calculations of all approaches are tabulated in the Table 2. As it can be seen from the Table 2, as expected dynamic variables are having much more influence on the bed-material load transport than any other group parameters. In dynamic parameters, flow discharge and flow depth are greater on the process. Each bed-material load transport equation available in the literature has its own validity and range of applicability of its data set (Sinnakaudan et al., 2006). Calculations shown in the Table 2 indicate that any error in determining flow discharge and flow depth may result in under or over estimation of bed-material load transport. Table 2 also shows that determining the bed particle size is also important.

Conclusion

Bed-material load transport models are often very complex and subject to semi-empirical or empirical equilibrium transport equations that relate sediment fluxes to physical properties such as velocity, depth and characteristic sediment particle sizes. In this work, ANN model for predicting bed-material load transport has been developed using several published flume and field datasets. The main conclusions of the study are listed below:

1. The performance of the ANN model can be said satisfactory if taken into the consideration of variability of the different datasets and nature of the experimental condition.

2. In engineering applications, errors in determining governing parameters affect the accuracy of the prediction of bed-material load transport. Experimentation has inherent errors which can not be neglected. Present analysis based on the input significant test through ANN model shows that in order to get the accurate prediction of bed-material load transport, there should be more accurate determination of hydraulic parameters.

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Flow Modelling Of Boundary Shear Stress Distribution For A Converging Compound Channel Using ANSYS

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ABSTRACT

The computation of boundary shear stress distribution in a converging compound channel flow plays a significant role in various engineering problems such as modelling and designing of channels, determination of total energy losses in a channel or river, sedimentation phenomena, analysis of bed load transfer and in the study of shear stress distribution. The objective of this flow modelling for converging compound channel is to find out the shear stress distribution at the walls and on the bed of converging compound channel. Various experiments are conducted in laboratory under different discharge and relative depths. These are further analysed and compared with the results of modelling in 3-Dimensional modelling software ANSYS 15.0 whose prime purpose is to obtain contours of longitudinal, lateral and resultant body shear stress for converging compound channel of 90 using K-Omega model.

Keywords: boundary shear stress, converging compound channel, relative depth, ANSYS 15

1. Introduction

Rivers have always played an important role in the development of ancient civilization. From being used as a source of water, for obtaining food, for transport, as a defensive measure, to being the largest source of hydropower to drive machinery, Rivers have proved to be of utmost importance in our lives. In spite of this, the research growth in perceiving the hydraulic characteristics of Natural River flood plains is not satisfactory. Recent hazards and disasters clearly states that the aftermath resulting from flood can turn out to be catastrophic in nature. Therefore the flood protection methods need to be studied precisely and ahead of time in order to forecast the water levels that are the outcome of any flood discharge. Numerical analysis of compound channel flow has not been explored in the past as compared to the number of experiments that have been conducted on compound channel. Fortunately the advancement in sophisticated CFD techniques has given rise a lot more scope to the research areas in this field now. Not only CFD techniques have proved to be most economical and time saving, they also cover the aspects of flow behavior which are very difficult to determine by experimentation. A number of CFD packages such as Fluent, CFX, Star-CD are now available in market and are used for research water flows. (Wormleaton(2005),Nguyan et al (2007), Zhang et al(2008), Shiono et al(2008)) have made a good use of these software packages for prediction of different aspects of 3D flow fields. In their work, they detected that flow characteristics in a compound channel depend on factors like topography, sinuosity, surface roughness, convergence etc. Despite such progress in CFD, alteration in flow behavior still remains an unresolved phenomenon. Since it's not practically possible to predict the flow behavior from experiments, the researchers have utilized various numerical models in order to predict the flow behavior. This problem arises due to errors that are generated in numerical modeling, such as grid generation, choice of turbulence model, discretizing scheme, specifying the initial and boundary conditions etc.

This paper highlights the flow modelling of boundary shear stress distribution for a converging compound channel which can be further used to analyse flood inundation in a river. The present work investigates effects of boundary shear stress distribution in a converging compound channel by using experimental data of a compound channel with different angles. In research the results and observations have been compared and modelled in ANSYS 15.

2. Experimental Setup And Procedure

Experiments were conducted in non-prismatic compound channels with varying cross section built inside a concrete flume measuring 15m×.9m×0.5m at National Institute of Technology Rourkela Hydraulic laboratory. The width ratio of the channel is $\alpha \leq 1.8$and the aspect ratio is $\delta \geq 5$. The converging angle of the channel is 9°.Converging length of the channel is 1.26m.The inside channel is made up of Perspex sheets. Water will be supplied through a Centrifugal pumps (15 hp) discharging into a RCC overhead tank. In the downstream end there will be a measuring tank followed by a sump which will feed to overhead tank through pumping thus completing recirculation path. Point velocities were taken at the grid points. The grid points in an experimental section are spaced horizontally by 5 cm and vertically by 0.2 multiplied by the depth of water at that point. Fig.1 shows the schematic diagram of experimental setup and dimensions of channel with test section respectively.Fig.2 shows the grid points in an experimental section where point velocities were recorded. Fig.3 shows different experimental sections. Table 1 shows the details of Experimental parameters for Converging Compound Channel. (Naik 2014)

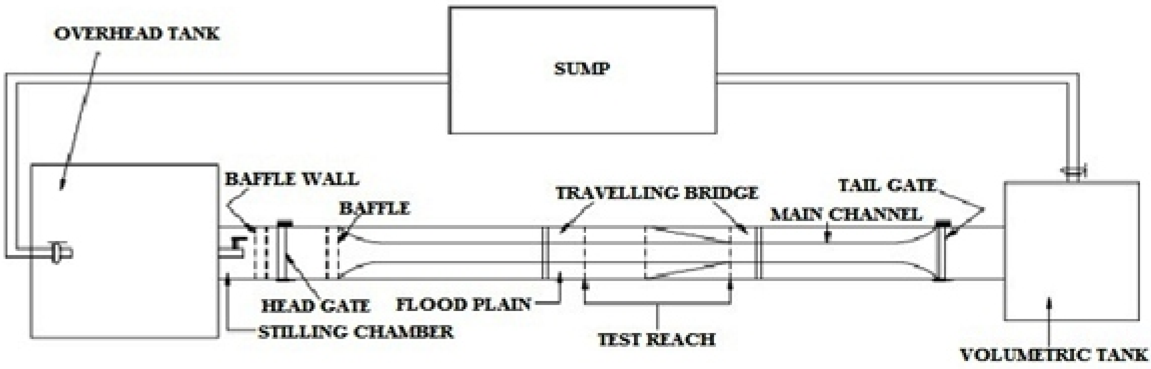


Figure1 Plan view of Experimental Setup

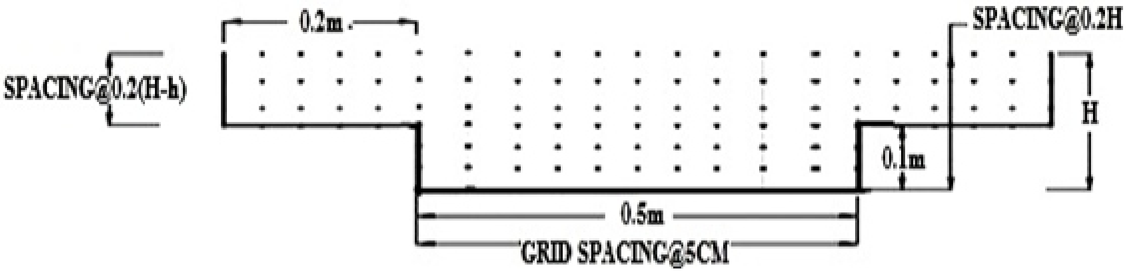


Figure 2 grid points in an experimental section where point velocities were recorded

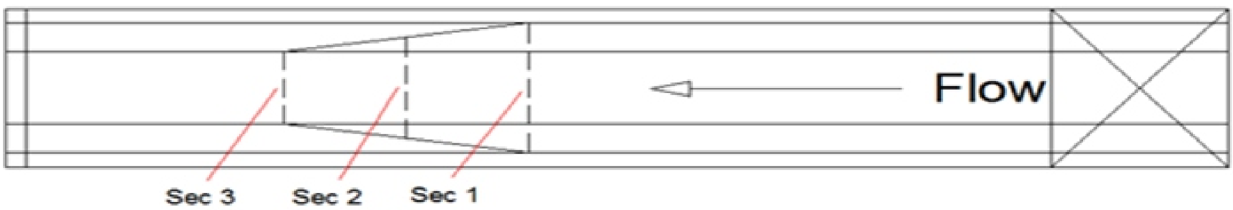


Figure 3 Experimental sections

Table1.Hydraulic parameters for the experimental channel data sets

Sl. No.	Description	Experimental Channel
1	Channel Type	Converging Compound Channel with smooth flood plain
2	Flume Size	15m*0.9m*0.5m
3	Geometry of Main Channel	Rectangular
4	Geometry of flood plain	Converging
5	Channel Width	0.5m
6	Bed Slope of Channel	0.0011
7	Main Channel Height	0.1
8	Relative Depth(Dr)(Total depth(*H*)- depth of main channel(*h*)/ Total depth *(H)*)	0.3
9	Convergent angle	90
10	Converging length of the channel	1.26m
9	Width ratio(á) = Ratio of top width (B) to main channel width (b)	1? *a* ?1.8
10	Aspect Ratio (ä)=Ratio of main channel width(B) to main channel height	

3. Numerical modelling

The paper is based on three-dimensional form of Navier-Stokes equations in which a Computational Fluid Dynamics simulation is used to verify the fluent model. Finite Volume Method also known as FVM is the most common method that is used in CFD modelling. It consists of both structured as well as unstructured grids. The governing equations are discretized in both space and time in free-surface modelling. In the present work, K-Omega model has been used. In this study the algorithms adopted to solve the coupling between pressure and velocity field is PISO, the pressure implicit splitting of operators use in Fluent 15. When the residuals of the discretized transport equation reach a value of 0.001 or when the solution do not change with further iterations, the numerical solution is converged. To promote the convergence of the solution the changing variables are controlled during the calculations. For the simulations with an unsteady solver, the difference in the mass flow rates at the velocity inlet and pressure outlet is monitored to be less than 0.01% in the final solution. Furthermost, a number of extra time steps are added to verify the steadiness of the flow field in the final solution. Fig 4 and 5 shows the geometry of converging compound channel along with its meshing of angle 90.

4. Methodology

The process of the numerical simulation of fluid flow generally involves four different steps and the details are given below.

(a) Problem identification

1. Defining the modelling goals
2. Identifying the domain to model

(b) Pre-Processing

1. Creating a solid model to represent the domain (Geometry Setup)
2. Design and create the mesh (grid)

(c) Solver

1. Set up the physics

- Defining the condition of flow (e.g. turbulent, laminar etc.)
- Specification of appropriate boundary condition and temporal condition.

2. Using different numerical schemes to discretize the governing equations.

3. Controlling the convergence by iterating the equation till accuracy is achieved

4. Compute Solution by Solver Setting.

- Initialization
- Solution Control
- Monitoring Solution

(d) Post processing

1. Visualizing and examining the results
2. X-Y Plots
3. Contour Draw

4.1 Preprocessing

In this initial step all the necessary information which defines the problem is assigned by the user. This consists of geometry, the properties of the computational grid, various models to be used, and the number of Eulerian phases, the time step and the numerical schemes.

4.1.2 Creation Geometry

The first step in CFD analysis is the explanation and creation of computational geometry of the fluid flow region. A consistent frame of reference for coordinate axis was adopted for creation of geometry.

4.1.3 Mesh Generation

Second and very most important step in numerical analysis is setting up the grid associated with the construction of geometry. The Navier-Stokes Equations are non-linear partial differential equations, which consider the whole fluid domain as a continuum. In order to simplify the problem the equations are simplified as simple flows have been directly solved at very low Reynolds numbers. The simplification can be made using what is called discretization. Construction of mesh involves discretizing or subdividing the geometry into the cells or elements at which the variables will be computed numerically.

4.1.4 Solver setting

The numerical scheme that CFD codes adopt is the finite volume method. The differential transport equations are integrated over each computational cell, and the Gauss and Leibnitz theorems are applied in this method. This consists of various models used for analysis, the initial and boundary conditions, the number of Eulerian phases, the properties of the materials, the physical and chemical phenomena involved. At last, the set of algebraic equations is solved by iteration process and the cell-center values of the flow variables are calculated.

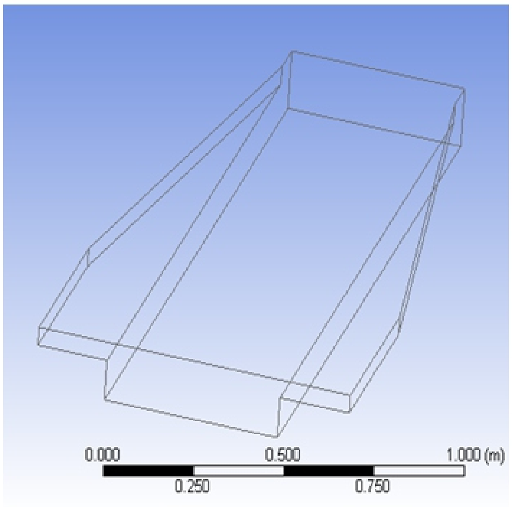


Figure 4 Geometry of Converging Compound Channel

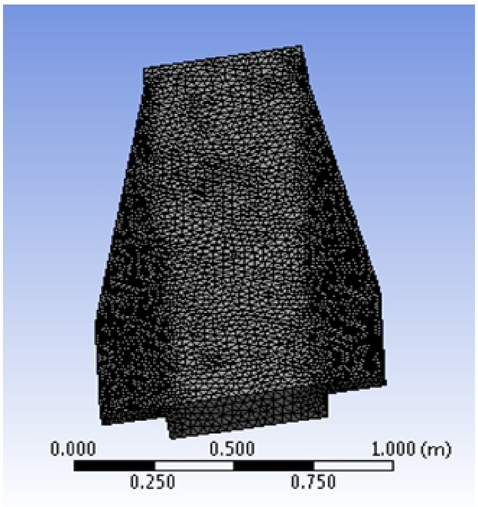


Figure 5 Meshing of converging Compound Channel

5. Results and discussion

Point velocities at grid points of the three different cross sections are collected and the velocity contour of each cross section is plotted using software called Surfer. The CFD model of similar specifications as experimental channel was simulated and again the velocity contour is extracted. Velocity contours from both experimental and numerical analysis are compared. It was found out that the point velocities are close to each other with a standard deviation of 10-12%. Figures 6, 7,8,9,10,11 shows the transverse velocity contours of sec-1, sec-2, sec-3 respectively for relative depths 0.3. Similarly figure 12, 13, 14 shows the both numerical and experimental boundary shear stress of sec-1, sec-2 and sec-3 respectively. From this figures we concluded that boundary shear increases from sec-1 to sec-3 gradually.

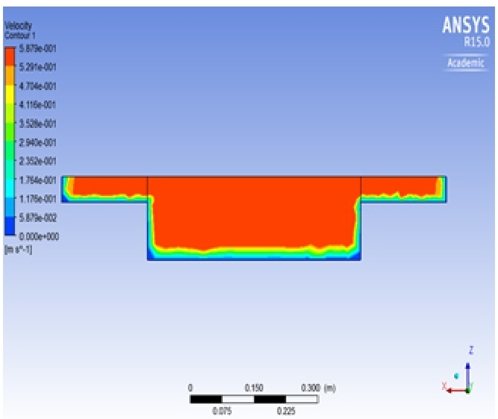


Figure 6 Transverse Velocity Contour of Sec 1 Numerical Analysis

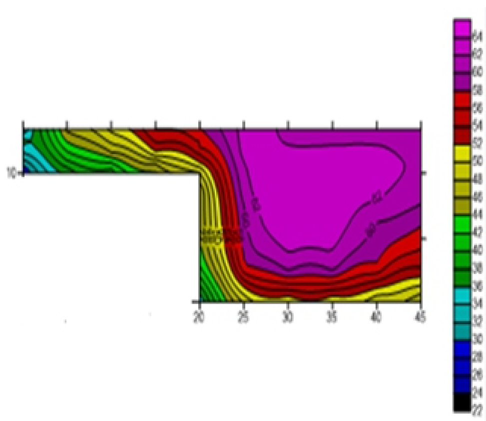
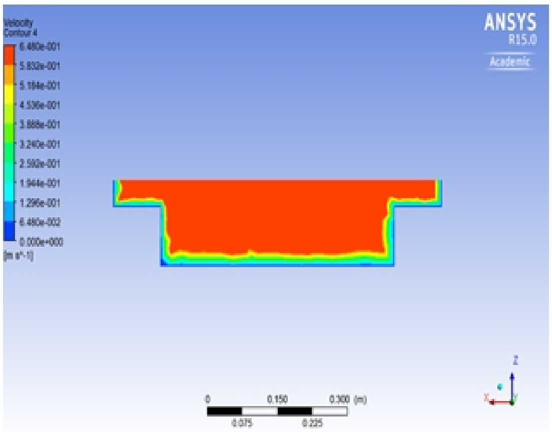


Figure 7 Transverse Velocity Contour of Sec 1 by from Experiment



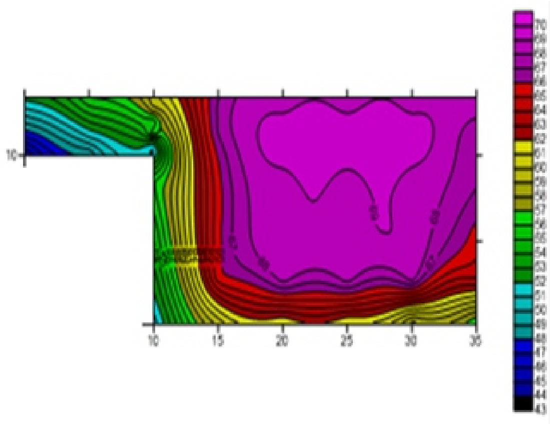
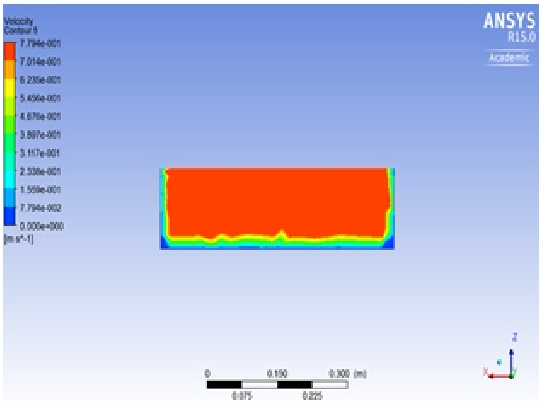


Figure 8 Transverse Velocity Contour of Sec 2 Numerical Analysis

Figure 9 Transverse Velocity Contour of Sec 2 by from Experiment



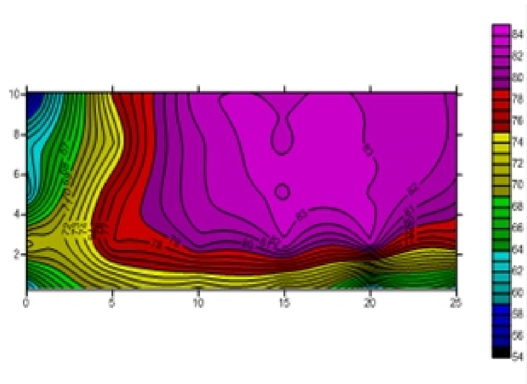


Figure 10 Transverse Velocity Contours of Sec 3 Numerical Analysis

Figure 11 Transverse Velocity Contours of Sec 3 by from Experiment

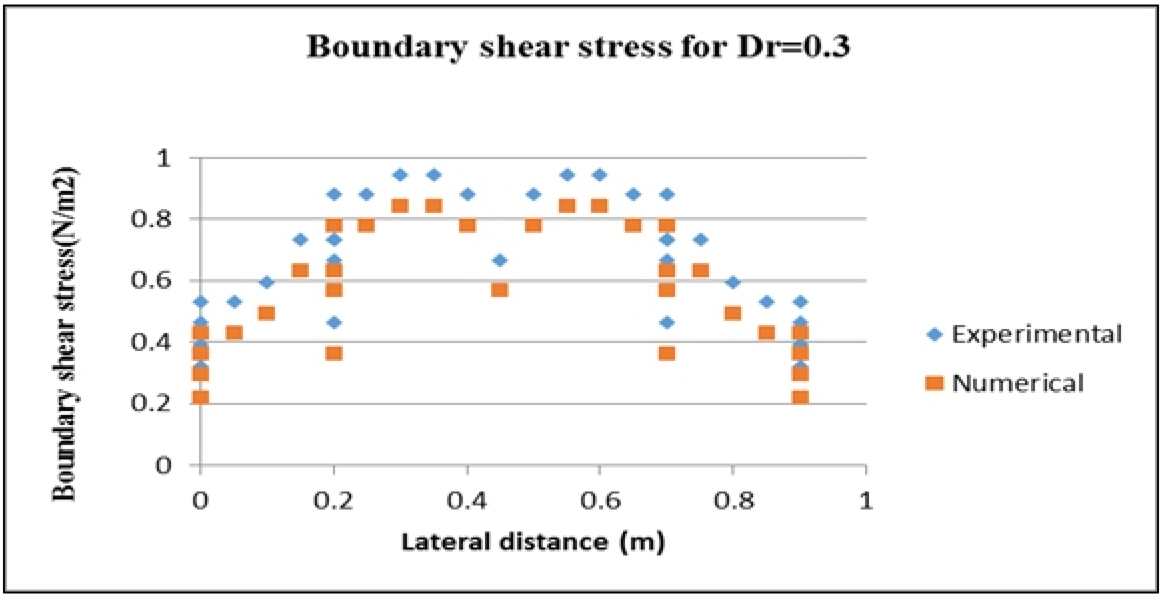


Figure 12 Boundary shear stress of Sec 1

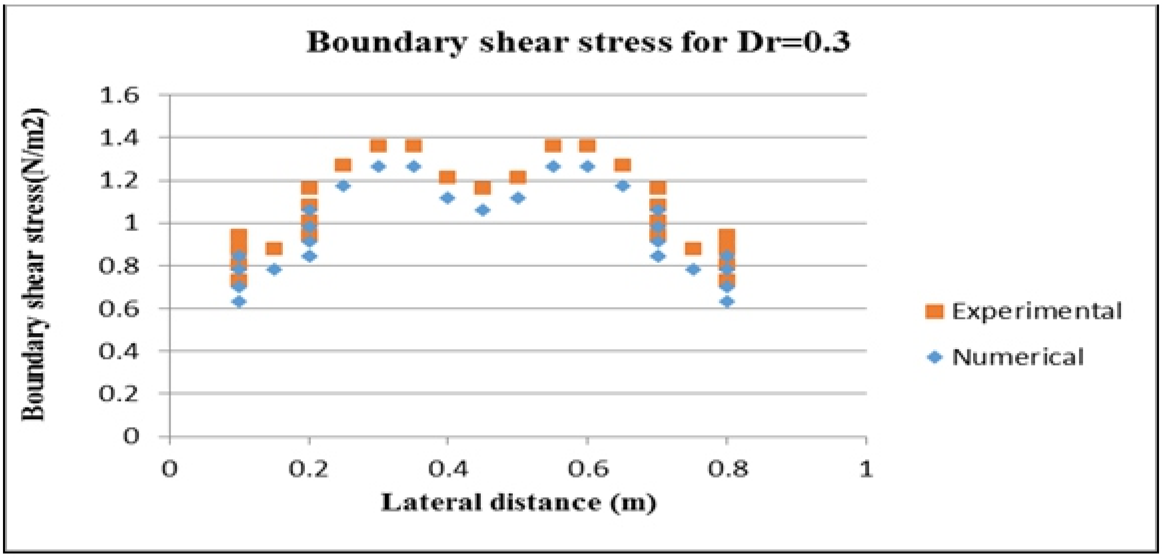


Figure 13 Boundary shear stress of Sec 2

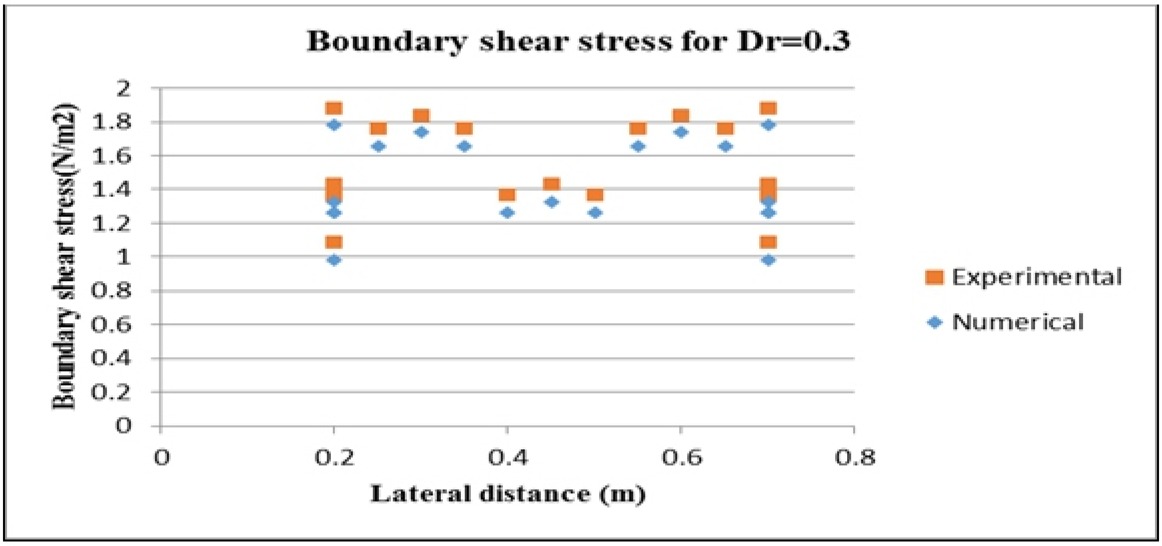


Figure 14 Boundary shear stress of Sec 3

6. Conclusions

In the present study, the experimental and numerical modelling of the flow pattern at a non-prismatic compound channel with a converging flood plain has been carried out. On the basis of the investigations concerning boundary shear stress distribution along the channel bed for relative depth 0.3 in a non-prismatic converging compound channel having 9° converging angle, the point to point observations were drawn and the following conclusions were made:

1. Three dimensional modelling of converging compound have been successfully verified with a mesh refinement studies and validated with experiments.

2. The experimental results that were observed and related to low velocity contours are then compared with the corresponding values obtained from Numerical Analysis for 3 different sections i.e. Sec-1, Sec-2, and Sec- 3 for the converging compound channel of relative depth 0.3. A good agreement between the results of the CFD analysis and Experimental results was seen. The velocity increases along the channel, as the cross-section area decreases in both the cases.

3. From the boundary shear stress distribution, it can be clearly seen that the numerical analysis provides good results and it is slightly underestimating the data when it is compared with the experimental result with a positive error of more or less than 5%.

7. Acknowledgements

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Evaluation Of Unit Stream Power Approach For Predicting Total Load Transport Rate

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A B S T R A C T

Sediment transport is a continuous process in nature. Considerable quantity of sediment of various sizes, shapes and density, get carried away by different modes of movement like bed load, suspended load and wash load depending upon the flow and channel characteristics. Yang (1972) concludes that total sediment concentration or total bed material load is directly related to unit stream power. In recent times, stream power concept has gained significance for sediment transport analysis. Yang C. T has worked on this concept extensively. The present study aims to verify the predictability of Yang's (1973) unit stream power equation for computing the total load transport rate in open channel. This unit stream power equation is tested using flume and river data sets. Various statistical parameters have been calculated for evaluating the performance of the selected formula. Graphical comparisons are done to demonstrate the performance and variations for different data sets. The Yang's (1973) approach was found to predict the sediment discharge reasonably well for the range of flows tested. Comparison between the predicted sediment transport rate and observed values indicates minimum discrepancy ratio in the range of 0.53 to 1.1. Also, the scattering of the predicted sediment transport rate from the line of equality lies within 11.26% to – 46.21%. The best predictability of Yang (1973) approach is observed for M4 mixture of Samaga et al. (1986 a, 1986 b) data set followed by US WWES –A(1936) data set ,Willis et.al (1972) data set, Niobrara river (1955) data and Stein R. A (1965) data set respectively, with discrepancy ratios of 1.1,1.1,0.96, , 0.934,0.87 and 0.8553 respectively for mean sediment diameter of 0.2mm, 0.29mm, 0.95 mm,0.1 mm, 0.258 to 0.359 mm and 0.399 mm .

Keywords: Unit stream power, statistical analysis, total load transport rate

1. Introduction

In alluvial river system, river banks will erode sediment and side channel will undergo modification with time. Effect of sedimentation reduces carrying capacity of river which may lead to flood water damage to surrounding area. The significant factor that has affected the natural sediment transport behavior in an open channel is construction of hydraulic structure like check dams, weir, hydroelectric plants etc. The knowledge of correct prediction of total sediment load carried by the channel is necessary to tackle hydraulic problems like aggradations, degradation, river training etc. According to

Waikhom S. I & Yadav S. M (2014) various contributors to the development of total load sediment transport theories are Laursen(1958), Bagnold (1966),Ackers and White (1973), Garde and Albertson(1961), Garde and Dattatri(1963), Graf and Acaroglu(1968), Toffleti(1968), Graf(1971), Yang C T (1973, 1979, 1984), Shu-Qing Yang (2003), Shen and Hung (1972), Jennifer Duan (2013) etc. The concept of the rate of work done should be related to the rate of energy expenditure was used by Bagnold (1966). It was demonstrated by Yang (1972) that the rate of sediment transport depends on the unit stream power more than any other hydraulic parameter. Unit stream power, defined as the time rate of potential energy expenditure per unit weight of water is shown to be the dominant factor in the determination of total sediment load concentration. Yang's (1973) unit stream power equation for the computation of total sediment concentration includes the incipient motion criteria. The present paper aims to evaluate the applicability of Yang (1973) unit stream power equation for total load transport rate using the incipient motion criteria.

2. Data Collection

The selected flume and field data sets used for testing Yang (1973) approach are collected from Brownlie, W.R. (1981) and Samaga et al. (1986 a, 1986 b) as described below in Table 1.

Table 1 Summary of range of data sets

Data Sets		Discharge, L/s	Flow depth, m	Flow width, m	Energy Slope, m/m	Mean grain size, mm (D_{50})
Stein R. A (1965)		78.15 to 482.37	0.09 to 0.37	1.22	0.61	0.4
Willis et al (1972)		78.15 to 480.239	0.378 to 0.1097	1.219	0.269 to 2.04	0.1
US WWES - A (1936)		6.68 to 62.55	0.03 to 0.2	0.7	1 to 2	0.95
Chyn S. D (1935)		12.29 to 35.96	0.05 to 1	0.61	1.1 to 3	0.59 to 0.84
Samaga et. Al (1986)	M1	5.58 to 14.64	0.06 to 0.10	0.2	5.05 to 6.87	0.35
	M2	7.54 to 14.36	0.07 to 0.10	0.2	4.96 to 6.93	0.29
	M3	6.26 to 13.38	0.06 to 0.09	0.2	5.42 to 6.93	0.27
	M4	8.14 to 13.46	0.06 to 0.09	0.2	4.49 to 6.10	0.2
Foley M (1975)		3.705 to 7.536	0.0305 to 0.0473	0.267	3.81 to 10.63	0.29
Niobrara River Data of Colby and Hembree (1955)		6456.043 to 13789	0.4354 to 0.5884	21.031 to 21.641	1.13 to 1.7992	0.258 to 0.359

3. Unit Stream Power Approaches For Computing Total Load Transport Rate

The required hydraulic parameters collected in the flume and field data sets for computing the total load transport rate of the Yang's unit stream power approach are processed through computer programs developed in the MS Excel spread sheet.

3.1 Chih Ted Yang (1973)

Yang C T (1973) defines unit stream power as the time rate of expenditure of flow potential energy per unit weight of flow, which can be expressed by the product of average velocity V and energy slope S. Chih Ted Yang (1973) suggest that the following relation provides the best correlation between total sediment concentration Ct and unit stream power (VS-VcS).

$$\log C_t = I_1 + J_1 \log\left(\frac{VS}{\omega} - \frac{V_C S}{\omega}\right) \tag{1}$$

Where, VS = unit stream power, $V_c S$ = Critical unit stream power at incipient motion. Further from multiple regression analysis with 463 of flume data from uniform flows, the following equations were obtained for I_1 and J_1 parameters.

$$I_1 = 5.435 - 0.286\ log\left(\frac{\omega d_S}{v}\right) - 0.457\ log\left(\frac{u_*}{\omega}\right) \tag{2}$$

$$J_1 = 1.799 - 0.409\ log\left(\frac{\omega d_s}{v}\right) - 0.314\ log\left(\frac{u_*}{\omega}\right) \tag{3}$$

Where, ω = fall velocity, ds = median sieve diameter of sediment, υ = kinematic viscosity, u* = Shear velocity. Eq. 1, 2, and 3 were proposed by Yang (1973) for the prediction of total sediment concentration Ct in parts per million by weight for the particles in the sand size range with ds = d50 = median sieve diameter of bed material.

4. Data analysis

Yang (1973) sediment transport function is tested against large data sets for both flume data and River data. The discrepancy ratio of calculated value to measured value for each sets of data is consider for comparison of performance. The percentage of data coverage between accepted lower and upper limit of the discrepancy ratio and their statistical properties is taken as the criteria of the goodness of fit. The predicted values are plotted against the observed values for same data set, so that the scattering of the values about the line of perfect agreement can be determined. Sample calculation for the predictability of Yang's 1973 approach using Samaga et al. (1986 a, 1986 b) data set for M1 mixture is as shown in Annexure 1 where comparison between observed and predicted values is made. Similar calculation is also done for the other flume data sets as described in Table 1.

Comparison between the observed and predicted values is shown in fig.1, 2, 3, and 4 for the data sets of Samaga et al (1986), US WWES-A (1936) and Willis et al (1972), Stein R. A (1965) and, Foley M (1975) data sets and data set of Chyn S. D (1973). Fig. 5 also represents the comparison between observed and predicted values of Niobrara River (1955).The solid red line represents line of equality.

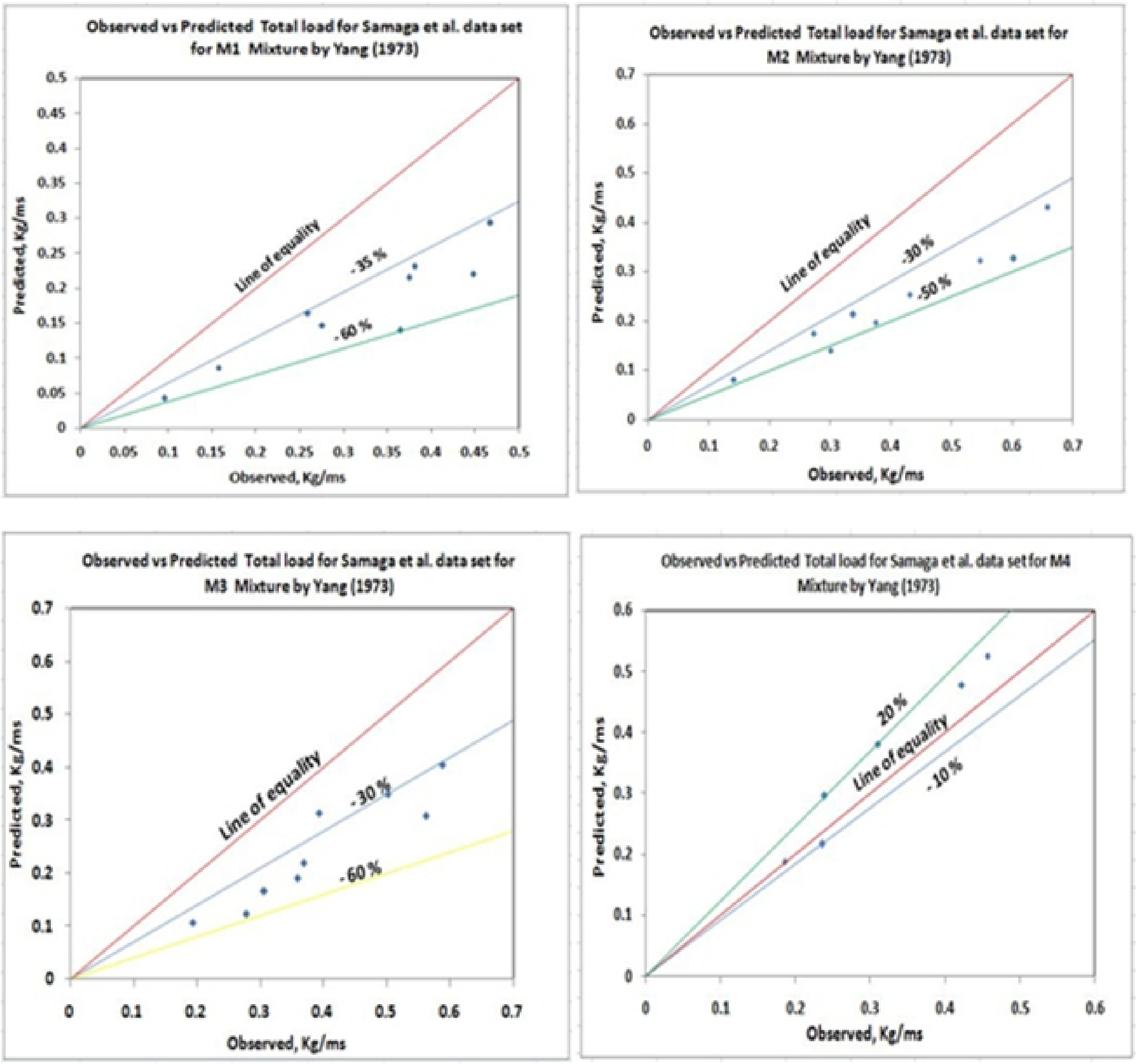

Figure 1 Observed vs. Predicted value for Samaga et al. (1986 a, 1986 b) data set for M1, M2, M3 and M4 mixture

It can be observed from Fig.1 that Yang (1973) sediment transport functions under predict for the mixture M1, M2 and M3 and over predict for M4 mixture of Samaga et al. (1986 a, 1986 b) data set. The percentage error between the observed and predicted transport rate is in the range of -30% to -60% for the M1, M2 and M3 mixture. For M4 mixture, the values are nearer to the line of perfect fit and most of data are lying between error ranging from -10% to 20%.

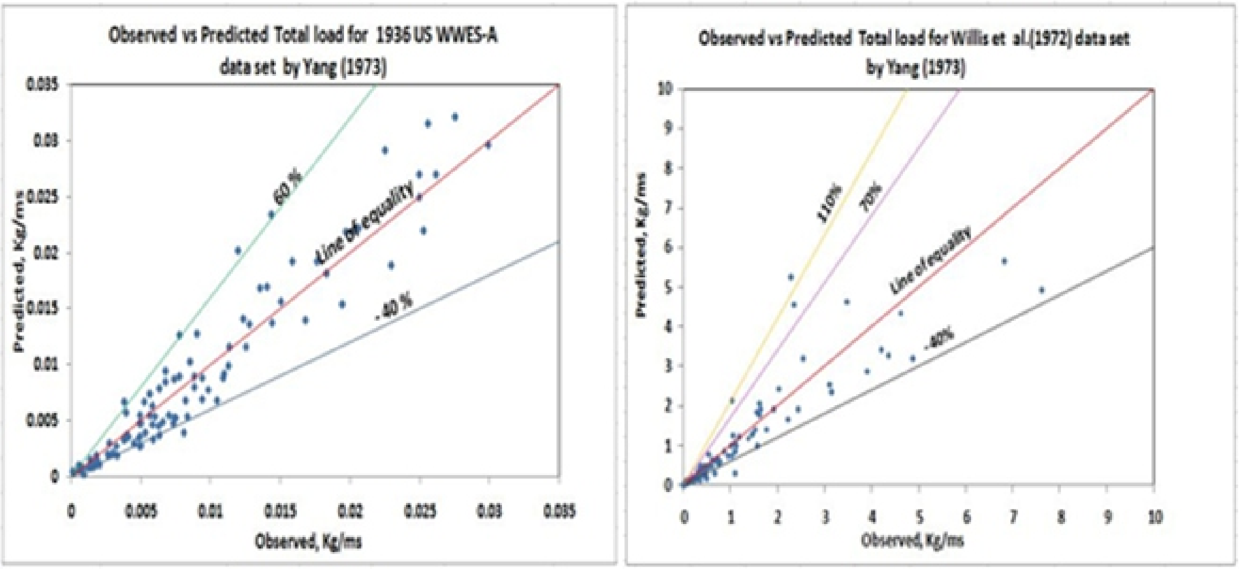

Figure 2 Observed vs Predicted value for Yang (1973) total load using 1936 US WWES-A and Willis et al (1972) data sets.

From Fig.2, it is observed that Yang (1973) sediment transport functions over predicts as well as under predict for the data set of US WWES-A (1936)and Willis et al (1972) data set .The deviation of predicted total load transport for US WWES-A (1936) data set from the observed value lies in with an error range of -40% to +70% with most of data lying between an error ranging from +40% to -40% and -40% to 110% for Willis et al (1972) data set .

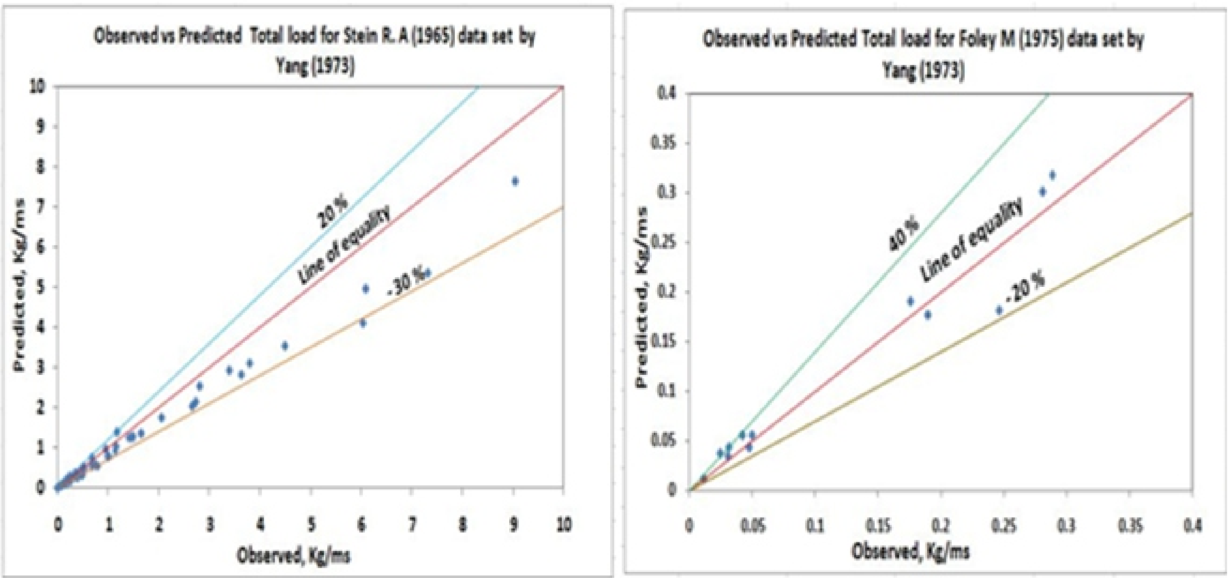

Figure 3 Observed vs Predicted value for Yang (1973) total load using Stein R. A(1965) and Foley M (1975) data sets.

From Fig.3, it is found that Yang (1973) sediment transport functions under predicts as well as over predict for Stein R. A (1965) data set and Foley M (1975) data set with % error ranging from 20% to -30% and from -20% to 40% respectively. From Fig.4, it is found that Yang (1973) sediment transport functions under predicts for most of the data points for Chyn S. D (1935) with percentage error in the range of -60% to 20% but most of data lies in between error ranging from -20% to -60%.

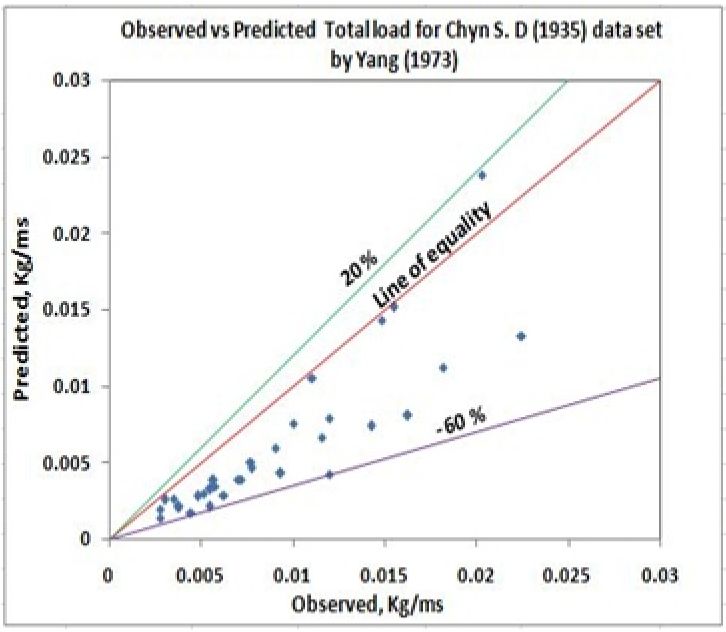

Figure 4 Observed vs Predicted value for Yang (1973) total load using Chyn S. D (1973) data set.

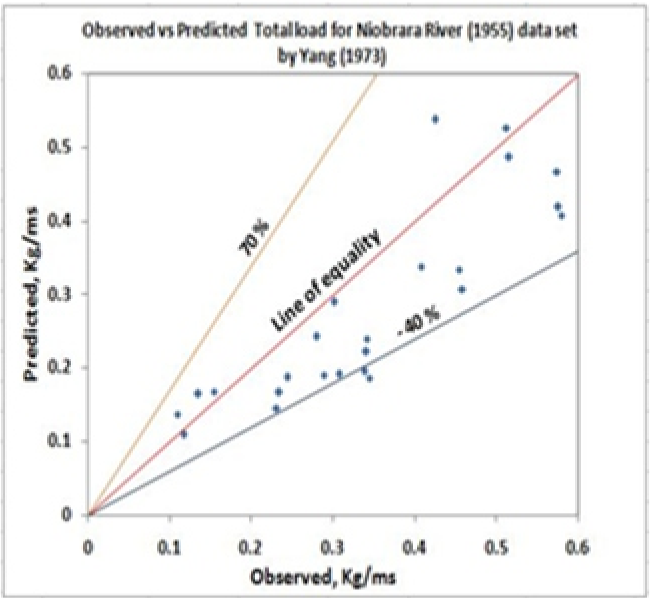

Figure 5 Observed vs Predicted value for Yang (1973) total load using Niobrara (1955) River data.

From Fig.5, it is observed that Yang (1973) total load transport functions under-predicts for most of the data points and over-predicts few data point for Niobrara river data. The predicted values lies within

5. Statistical parameters calculation

Presently, statistical parameters are used to evaluate the performance of Yang (1973) total load transport approach. Statistical parameters and their results are described below in Table 3

Root Mean Square Error (RMSE) measures the deviation between the trend of the predicted results and observed values.

$$Rmse = \left[\sum_{i=1}^{n} \frac{\left(Q_{bo} - Q_{bp}\right)_i^2}{n}\right]^{\frac{1}{2}} \qquad (4)$$

Discrepancy ratio(r) is the ratio of predicted to observed data whose value signifies the accuracy of each of the selected equations.

$$r = q_{(predicted)} / q_{(observed)} \qquad (5)$$

Inequality coefficient (U): It refers to a simulation statistics related to the RMSE, defined as under,

$$U = \frac{Rmse}{\left[\frac{1}{n}\sum_{i=1}^{n} (Q_{bo})_i^2\right]^{1/2} + \left[\frac{1}{n}\sum_{i=1}^{n} (Q_{bp})_i^2\right]^{1/2}} \qquad (6)$$

Ranking is determined to compare the performance of selected function for different data set. The calculation is done by calculating the ratio between the predicted sediment transport and the measured sediment transport. For the classification of the transport rate predictions, the ratio is defined as:

$$\text{ratio}\,(j) = \frac{q_{b(Predicted)}}{q_{b(measured)}} \qquad (7)$$

The Ranking is calculated as:

$$\text{Ranking} = \frac{\sum_{j=1}^{n} factor\ (j)}{n} \qquad (8)$$

$$\text{Where, factor (j)} = \begin{cases} ratio(j) if ratio(j) \leq 1 \\ \frac{1}{ratio(j)} if ratio(j) > 1 \end{cases}$$

In which n is the total number of tests, and j the specific test. The maximum rank is one.

Table 3 Summary of Statistical Parameters used to analyze Yang (1973) total load function.

Statistical Parameters									
Sr. No	Data set		D50(mm)	RMSE	DR	Inequality coefficient	AVG %	Score	Ranking
1	Stein R. A (1965)		0.399	0.5117	0.8553	0.1193	-14.464	0.76905	6
2	Willis et al (1972)		0.1	0.6062	0.9337	0.1745	-6.6273	0.77843	5
3	1936 us WWES -A		0.95	0.0023	0.9697	0.1009	-3.029	0.8298	3
4	Chyn S. D (1935)		0.59 to 0.89	0.004	0.627	0.2186	-37.297	0.61753	7
5	Samaga et al. (1986 a, 1986 b)	M1	0.35	0.1545	0.5378	0.2959	-46.213	0.48994	10
		M2	0.29	0.1817	0.5778	0.2609	-42.214	0.57787	9
		M3	0.27	0.1584	0.6014	0.2343	-39.852	0.60148	8
		M4	0.2	0.0522	1.1126	0.0752	11.2691	0.87983	**1**
6	Foley M(1975)		0.29	0.0231	1.1084	0.0727	10.8433	0.85137	2
7	Niobrara river (1955)		0.25 to 0.35	0.2097	0.8701	0.1443	-12.991	0.78823	4

Conclusion

Following findings can be summarized from the present study:

1. Stein R. A (1965) flume data under predicts the total sediment load transport rate of Yang (1973) approach with an average error of - 14.4%.

2. Total load transport rate predicted for Yang (1973) approach using data set of Willis et al (1972) provide an avg. error of -6.62% with discrepancy ratio of value 0.93.

3. Yang (1973) total load transport rate approach predict well for 1936 US WWES – A data set, with an average error of -3% with discrepancy ratio near equal to 1.

4. For data set of Samaga et al. (1986 a, 1986 b),Yang (1973) approach under predicts the total load transport rate for mixture M1, M2 and M3 but over predict as the diameter decreases in M4 mixture, giving discrepancy ratio near equal to 1 with few degree (11.2 %) of deviation from line of equality.

5. Foley M (1975) flume data over predicts the total sediment load transport rate with an average error of 10.84%.

6. For Niobrara River (1955) data set, total load transport rate is well predicted for Yang (1973) transport function giving discrepancy ratio near equal to 1 with few degree (-9% to -12%) of deviation from the line of perfect agreement.

7. Overall consistency is observed in the trend of prediction in most of the selected range of hydraulic data.

8. Considering the maximum coverage of data, M4 mixture of Samaga et al. (1986 a, 1986 b) data set ranked first (1) followed by Foley M (1975) data set, 1936 US WWES.

9. Niobrara River (1955) data ranked fourth in predicting total load transport rate for Yang (1973) transport function for the selected range of data sets.

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Protection Measures For Railway Track Along A River In Tamilnadu

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A B S T R A C T

The Vellar river basin is located in the Northern part of Tamil Nadu State in South India, between the latitudes 110 13' N – 120 00' N and longitudes 780 13' E – 790 47' E. The total area of the basin is 7520.87 Sq. Km. The total length of the river is about 150 km. The river Vellar emanates on the southern slope of Kalvarayan hills in Salem Dist. and drains into Bay of Bengal near Parangipettai (Port-O-Nova) in Chidambaram taluk of Cuddalore District. The normal rainfall of the area is 130cm and receives maximum rainfall during north east monsoon period from October-December with in 15 to 20 days. In the lower reaches the river is meandering resulting in heavy erosions due to very high discharge during north east monsoon rainfall resulting erosion of railway line connecting pilgrimage centers with Chennai. Initial field surveys were conducted for actual field information and it was found that the river bank containing railway track is under heavy threat of erosion. Using the data numerical modeling studies were performed using HECRAS. Based on the studies vulnerable locations were identified and structural measures were designed. The protection structure is in the form of anchored steel sheet pile wall connected by a tie rod. The factor of safety against slope stability and the bending moment in sheet pile and tie rod force are obtained by modelling .The details of the studies and conceptual layout of remedial measures are detailed in the paper.

Keywords: modelling; sheet pile; stability

1. Introduction

The Vellar river originates in the Kalrayan hills of Salem district in the reserve forest area at of Salem in Salem District and drains into Bay of Bengal near Parangipettai (Port-O-Nova) in Chidambaram taluk of Cuddalore District.TheVellar river basin is located in the Northern part of Tamil Nadu State in South India, between the latitudes 110 13' N – 120 00' N and longitudes 780 13' E – 790 47' E. The total area of the basin is 7520.87 Sq. Km. The total length of the river is about 150 km (Fig 1). During the flash floods of north east monsoon the river causes very heavy erosion at a village Killai located along the bank. The very important train route connecting Chennai to pilgrimage centers like Rameshwaram, Velankanni, Nagapattinam runs along Killai. Apart from this the goods train service is also provided along this coast. The area experiences heavy rainfall during North East monsoon during October-heavy flash floods the bank adjacent to railway track is under heavy threat of erosion. Hence the study was entrusted to IIT Madras to suggest suitable remedial measures.

2. Nature Of Problem {Main Headings [Times New Roman, 12pt + Bold]}

The River Vellar as the name itself depicts a river which carries heavy floods. During the flood times the river quite often finds its path in a meandering way at many locations. The satelliteimage shown below (Fig 2) denote the severity of meandering of the Vellar River. On November2005 unprecedented heavy rain occurred causing heavy floods in Vellar and Manimuktha river, which realized a maximum flood discharge of 105423 cusecs. This resulted in the concentration of flow in the right bank resulting erosion of right bank.

Figure 1 Index map

3. River behavior

The railway route connecting south pilgrimage centers of Tamilnadu runs through Killai village located on the south side of Vellar River. Apart from this goods trains also ply along this route connecting minor ports and power plants.Based on the information collected from the field it is inferred that the south bank of Vellar River was located at a distance of about 1 km from the tracki in the year 1912. As the river is having only seasonal flow, it has undergone migration to the south side. At present the bank is located at a distance of 15m from the track. It is temporarily protected by dumping sand bags .These serve as short term measures as they will sink and fail due to heavy surcharge traffic load. It is in such a stage that unless some permanent remedial measures are made, the route will be cutoff in future by seasonal flood waters

Figure 2 Details Vellar river

5. River discharge details

The river is having only seasonal flows. The flow will take place only during monsoon rainfall. The river basin is more influenced by north east monsoon from October to December (Fig 3). The flow data from 1980 to 2012 was collected and analysed by dividing the data in to three decades namely First decade from 1980-1990, second decade from 1991-2000 and third decade from 2001-2012. The analyses indicate that there is an increasing trend in the magnitude of discharge.The highest value of one day discharge during the first decade from 1980 to 1990 was 68762 cusecs.In the second it was 82475 cusecs. During the third decade it was 105423 cusecs.

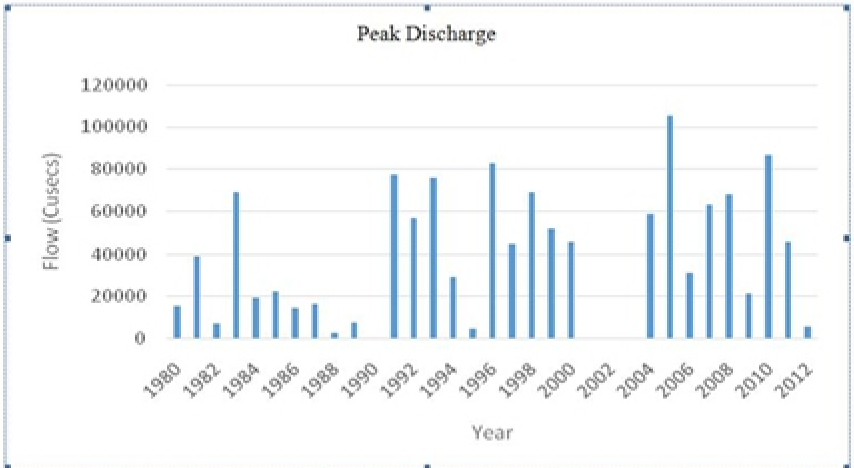

Figure 3 Details of placement of porcupine field in the flume

5. Rainfall pattern

The district has a hot tropical climate. The summer season, which is very oppressive, is from March to May. The southwest monsoon, which follows, lasts till September. October to December constitutes northeast monsoon season. January to February is the comparatively cooler period. The annual normal rainfall for the period (1901- 2000) ranges from 1050 – 1400 mm. The normal annual rainfall over the district varies from about 1050 mm.The contributions of individual seasons are as follows: NE-57%, SW-31%, Summer- 7% and winter 5%.But the rainfall pattern in third decade shows high magnitudes of north east monsoon.

6. Main issues

The very high value of discharge event will result in flooding of tracks above the banks. The bund adjacent to the track undergoes heavy erosion. Hence it is felt that the hydraulic performances of river cross sections are to be studied. Based on the studies suitable remedial structures are to be worked out.

7. Methodology

The methodology adopted are as detailed below

- Field inspection and reconnaissance survey
- Hydrographic survey
- Hydraulic model studies using HECRAS
- Protective structure modelling and design

7.1 Field inspection

The IIT team visited the site of the eroded Vellar stretch . During the field visit, the water resources department officials also accompanied him. There were detailed discussion on eroding pattern and it was informed that unless some permanent remedial measures were implemented, the railway route will be cut during flash floods. At present the stretch is protected by temporary measures which are not so stable (Fig 4 &5) .In order to understand the hydraulic behaviour, the cross sectional details are to be taken. Hence a hydro graphic survey was conducted,.

Figure 4 Eroded bank near track

Figure 5 Disturbed protection measures

7.2 Hydro graphic survey

The hydro graphic survey was conducted along the river longitudinal section. The entire survey area is divided in to three stretches as shown in fig below. The total length of the reach is 630m. The affected reach is 250m with 190m of upstream and downstream length. The cross sections are taken for very 10m length along the longitudinal section . The depth along cross section was taken for every 5m commencing from the depth adjacent to the bank. Here cross sections are taken from downstream to upstream of each reach. It is furnished vide Fig 5. The hydrographic survey data indicates that bank adjacent to the affected reach of 250m is deeper compare to its upstream and downstream reaches. The water depth is nearly 5m in the reach of 250m length. The width of river is about 160m in the u/s and d/s of affected reach. In the affected reach it reduces to 120m for a meandering length of 250m.

Figure 6 Survey area plan

7.3 Hydraulic model studies using HECRAS and findings

In order to assess the existing hydraulic conditions of the field, numerical model studies were made. For the present study the U.S. Army Corps of Engineers' River Analysis System (HEC-RAS) software is used. This software is developed by the Hydrologic Engineering Center (HEC), which is a division the Institute for Water Resources (IWR), U.S. Army Corps of Engineers. HEC-RAS allows users to perform one-dimensional steady and unsteady flow calculations (HEC, 2002). In a HEC-RAS steady state simulation, water surface profiles are computed from one cross-section to the next by solving the standard step iterative procedure to solve the energy equation. The energy equation is intended to calculate water surface profiles for steady gradually varied flow. The inputs are geometric, flow and boundary data. The geometric data consisting of cross section and chainages were keyed in after making a base map of the study area. The Right bank of Vellar River is found to be highly vulnerable to erosion. The effect of meandering diverts the flow creating erosion. The numerical experiments with HECRAS indicate that cross section U/S of affected reach can carry a maximum of about 1000 cumecs (Fig 7). The meandering effect with a cross slope in bed creates eddies which ultimately erodes the bank. Hence it is also suggested to raise the bund on the left side of river up to (+) 2.50m.

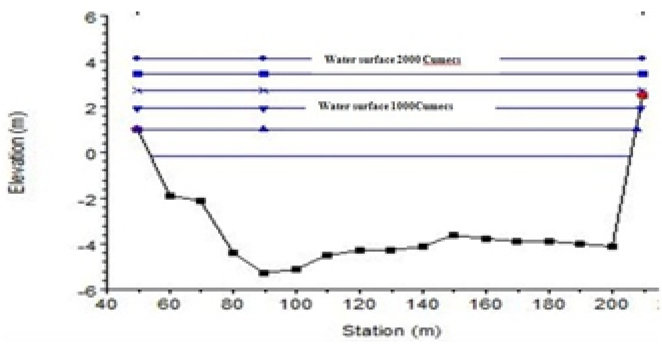

Figure 7 Water level from HECRAS model run

7.4 Protective structure modelling and design

In the present site of Killai the bank mainly composed of weak clay materials which is not in a position to resist the high eddies formed during high discharge values. Hence it is proposed to strengthen it by driving sheet piles backed by gabion bax. Sheet piling is a form of driven piling using thin interlocking sheets of steel to obtain a continuous barrier in the ground (Fig 8). The main application of sheet piles is in retaining earth and erected to act as a permanent remedial measure.

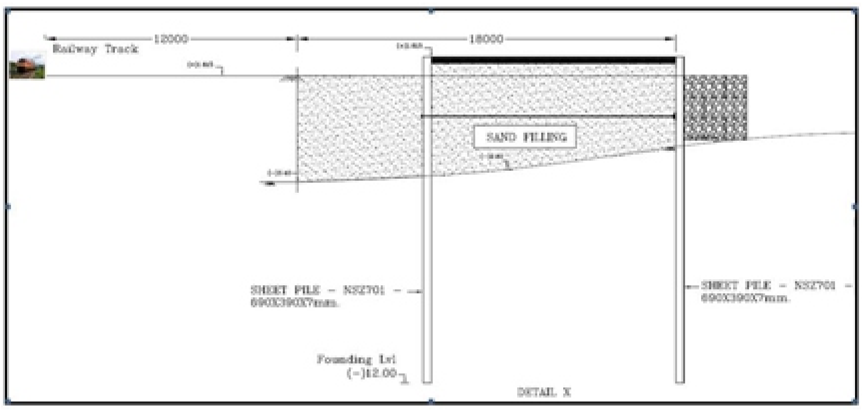

Figure 8 Details of protection measures

8. Analyses of sheet pile

The soil in this region is mainly composed of soft soil of SPT 'N' value ranging 2 to 4. The moist soil has a considerable amount of plasticity. This is semi-impermeable in nature and it is capable of inducing high hydrostatic pressure in the pore fluids and causing impounding of groundwater. The input consists of field properties such as cohesive strength, saturated weight, friction angle, poisson ratio, Youngs modulus, and permeability. Such soil structure interactions were detailed in Kasinathan Muthukkumaran and Ranganathan Sundaravadivelu (2007) and Anasua GuhaRay and Dilip Kumar Baidya (2014). The protection measures consist of double sheet piles connected by tie rod. Each sheet pile has thickness of 12 mm with yield strength 250 Mpa and height 14.5m with top level +2.5m and founding level -12 m separating apart with distance of 12m width. Both sheet piles are connected by tie rod of diameter 50mm with an interval of 1.38m. Since the training wall was proposed parallel along the Railway track, wheel load of 92214 N is considered as two different point load in the model as per Indian Railway Standard manual. A Gabion layer of height 3m varying -1.5m to +1.5m has to be installed on the river side which will dissipate the river currents and protect the sheet pile from scouring. Similar type of analyses was detailed for Orissa coast by Sherlin Prem Nishold et al (2014). The output gives deflection, moment and forces. The section was tested for bending stress.

9. Numerical Analysis using Plaxis 2D

Plaxis 2D is a finite element based software which is used to study common geotechnical problems, such as soil structure interaction as well as slope stability analysis. In this present study stability of sheeet was investigated based on the maximum bending moment and the bending stress generated by the lateral forces. Soft soil as soil material behaviors, Rubble stone as rock material, Pile was modeled as plate element, Geotextile was modeled with geogrid member in the finite element models. The soil constitutive model used in this analysis was the Mohr-coulomb model. Staged construction is a feature in Plaxis 2D with which it is possible to change the geometry and load configuration by deactivating or reactivating loads, volume clusters, or structural objects created in the geometry input. Staged construction enables an accurate and realistic simulation of various loading, construction, and excavation processes. Plaxis program allows the users to introduce an interface parameter, Rinter, between the soil and other materials, PLAXIS uses Rinter to calculate the strength parameters corresponding to the interface elements based on the following the rules

$C_i = R_{inter} * C_{soil}$

$\tan\varphi i = Rinter * \tan\varphi_{soil0}$

Where

C_i and φ_i are the interface material properties.

C_{soil} and φ_{soil} are the soil's properties.

R_{inter} is less or equal to 1.0 in the program.

10. Numerical experiment

The model of river training wall is drafted in Plaxis 2D . Appropriate field geometry and material properties had been used based on the complex stress strain characteristics of the selected material and field data (Fig 9 & 10). Generated sheet pile model is analyzed for wall maximum deflection, bending moment and slope stability under worst case , for various the units of length, force and time. In boundary conditions, standard fixities were employed in the bottom and rollers were employed in the sides of the model generated. For every stage construction, the structure was analyzed for limit equilibrium method and for global factor of safety, the structure was analyzed by both Finite Element and Limit Equilibrium method in terms of c-phi reduction to attain factor of safety.

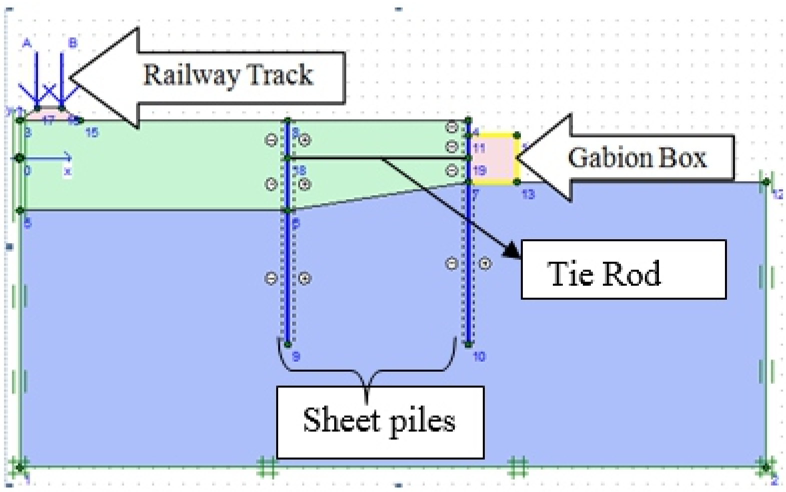

Figure 9 Numerical model of vellar river training wall

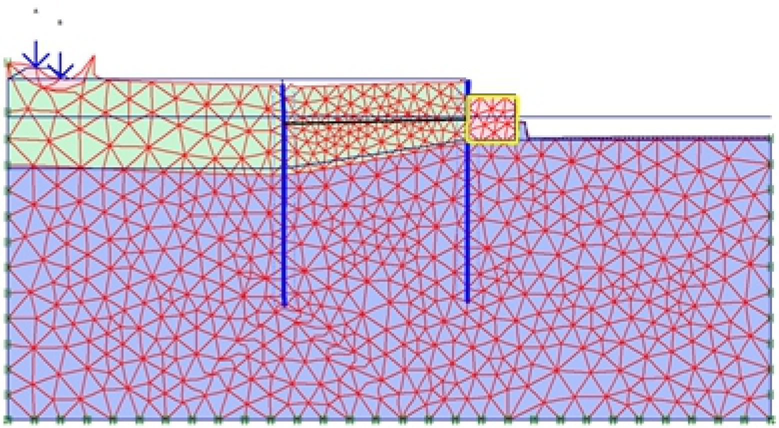

Figure 10 Deformed mesh of Numerical model

11. Comparison of Numerical Results

The model was analyzed with limit equilibrium method approach on each phase of stage construction to attain the minimum factor of safety under construction and for the final phase c-phi reduction was carried out to arrive the factor of safety against slope stability. The right and left sheet pile wall displacement , shear force, maximum bending moment (Table 1) and bending stress generated by the lateral forces are estimated. The calculated bending stress is within allowable stress. The factor of safety against slope stability has increased from 1.50 to 3.73 with the presence of sheet piles connected with tie rod.

Table 1 Results of analyses using Plaxis

Sheet pile	Deflection (mm)	Bending moment (KN-m/m)	Shear force (KN/m)
Right side	98	46.73	46.51
	0.818		
Left side	100.74	-62.5	35.74

8. Conclusions

In the present site of Killai the bank mainly composed of weak clay materials which is not in a position to resist the high eddies formed during high discharge values. Hence it is proposed to strengthen it by driving two rows of sheet piles. The details are as detailed belowThe protection measures consist of pair of sheet piles for a length of 350m.

- The Sheet piles recommended are straight steel sheet (690x390x12mm).
- The outer sheet pile is to be driven at a distance of 18m from the inner bank.
- The inner pile is to be driven at a distance of 6m from the inner bank.
- The Founding level of sheet piles are (-) 12.00m.
- The Space between the two piles is to be filled with sand dredged from left side bank.
- The two sheet piles are inter connected with a MS tie rod of size at (+)1.50m of diameter 50mm welded with outer sheet pile and bolted with inner sheet pile.
- The Gabion box of size 1mx1mx1m and 2mx1mx1m with 100x100mm mesh is to be placed in front of outer sheet pile for the whole proposed stretch.
- The Gabion box is to be placed in 3 layers as a protection for outer sheet pile.

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