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Flow resistance in gravel bed open channel flows case: intense transport condition

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ABSTRACT

Gravel bed can be categorised into three bed load conditions, i.e. no load, moderate and intense. An experimental investigation was carried out in an open channel flow with gravel bed surface of grain size of D_{50} values 6.5 mm intense load conditions. The investigation of the roughness characteristics of gravel bed open channel flows under intense load conditions over various discharges and flow depths are presented. Variation of friction factor for the roughness conditions for different flow depths is estimated. The intensity of the bed load is calculated with the help of sediment transport rate. The bed load transport rate for the intense load conditions is also determined from experimentation on gravel beds of 6.5 mm gravel size for different flow depths. Using the data-set of other researchers and present experimental data, a new model as a function of intensity of the bed load transport with respect to Shield's parameter is formulated. The modified model gives satisfactory results as compared to previous works, which is displayed through error analysis.

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KEYWORDS
Gravel bed open channel; Darcy–Weisbach friction factor; bed load; sediment transport; Shield's parameter; boundary shear stress

1. Introduction

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 quiet convoluted. The effects of bed load on the estimation of friction factor *f*, is possible because of low sediment transport rates (Hey 1979; Van Rijn 1984; 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 1984; 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, proposed correction factor to take into account such effects including flatbed problems (Wong and Parker 2006). By comparing resistance produced by flow over a mobile bed, Bathurst et al. (1982) 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 decades researchers have 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; Campbell et al. 2005; Carbonneau and Bergeron 2000; Mahdavi and Omid 2004; Omid et al. 2003). 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) 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 D_{50} sediment diameters of 6.5 mm.

2. Previous investigations

The most widely used flow resistance models (Manning, Chezy and Darcy–Weisbach equation) shows the relation between the linear energy losses to the mean flow velocity. All these models are semi-empirical and validation 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, a high/intense load is considered, which is suggesting high sediment transport flows. These transport flow condition can be characterised by a resistance coefficient *f* that decreases as the relative depth decreases.

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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:

$$\sqrt{\frac{8}{f}} = \frac{u}{\sqrt{gRS}} = \frac{u}{u*} \tag{1}$$

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 has been made between $(8/f)^{1/2}$ and relative depth, *R/D* Figure 2.

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 (Equation 2):

$$\frac{u(z)}{u*} = \frac{1}{k} ln \frac{z}{z_0} \tag{2}$$

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; Nikora and Smart 1997; Smart 1999; Song et al. 1995) 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 Equation (3) (for $k = 0.4$):

$$\sqrt{\frac{8}{f}} = \frac{U}{u^*} = \frac{1}{k}\left[\left(\frac{R}{R - z_o}\right)\ln\left(\frac{R}{z_o}\right) - 1\right] \approx 5.75 \log \frac{0.368R}{z_o} \tag{3}$$

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:

$$\sqrt{\frac{8}{f}} = E + 5.75 \log\left(\frac{R}{D}\right) \tag{4}$$

where *E* is a constant, which is dependent 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 the trapezoidal open channel.

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-, 22- 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 discussions 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 a slope, which is responsible for the incipient motion of sediments, is a new approach. The semi-logarithmic model proposed by Cao et al. (2006) as:

$$\sqrt{\frac{8}{f}} = 3.75 + 5.91 \log\left(\frac{R}{D}\right) \tag{5}$$

The Manning-Strickler equation given is:

$$u = K_s R^{\frac{2}{3}} S_0^{\frac{1}{2}} \tag{6}$$

where K_s is the grain resistance Manning-Strickler coefficient. From the definition of K_s which is:

$$K_s = \frac{21.1}{D^{1/6}} \tag{7}$$

It becomes

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

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. Rather an empirical law was suggested, essentially affecting the slope of the logarithmic law. 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:

$$\sqrt{\frac{8}{f}} = 2.5 + 9.5 \log\left(\frac{R}{D}\right) \tag{9}$$

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. 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 suggestively named regime 3 and which was similar to sheet flow regime. The model concerned with the intermediate-scale roughness ($R/D < 17$) must be modelled with the slope coefficient of 9.5. The semi-logarithmic model proposed by Recking (2006) as:

$$\sqrt{\frac{8}{f}} = -1 + 9.5 \log\left(\frac{R}{D}\right) \tag{10}$$

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

$$\sqrt{\frac{8}{f}} = 5.75 \log\left(\frac{2R}{D}\right) \tag{11}$$

There were two common methods used towards the theory of bed load transport. One of the most popular method was based on variables such as stream power, shear stress, discharge, 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, which can be exhibited through number of equations presented below. Dependability for the usage of these equations was on the assumptions of the existing models. The second method was a 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 was intricate and challenging (Yang 1996).

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

$$\theta = \frac{\tau}{(\gamma_s - \gamma)D} \tag{12}$$

$$\theta_c = \frac{\tau_c}{(\gamma_s - \gamma)D} \tag{13}$$

where τ is bed shear stress, D is particle diameter and τ_c is critical bed shear stress

$$\theta_c = \frac{0.273}{1 + 1.2d*} + 0.046\left(1 - 0.576e^{-0.05d*}\right) \tag{14}$$

where d^* is:

$$d_* = D\left[\frac{s-1}{v^2}\right]^{.33} \tag{15}$$

Shields (1936) was the first investigator, who described the critical shear stress as a function of different particles sizes, D (irrespective of uniform or non-uniform shape) which supervises the threshold motion by a uniform flow and expressed as:

$$\frac{q_b\gamma s}{q\gamma S} = 10\frac{\tau_0 - \tau_c}{(\gamma_s - \gamma_0)D} \tag{16}$$

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

$$\Phi = \frac{q_b}{\sqrt{\left(\frac{\rho_s}{\rho} - 1\right)gD^3}} \tag{17}$$

Following approaches and the developed equations by the previous researchers are given in the Table 1.

3. Experimental set-up

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. For the 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 a 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 is fitted for the reducing the turbulence and make a uniform flow throughout the channel section. Head gate 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 of the flume. There was the 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 by hydraulic jack provision flume can be pivoted over the centre in lengthwise direction to provide bed sloping. The plan view of the experimental channel used in the present study is shown in Figure 1. Experiments are conducted in a straight simple trapezoidal channel with uniform cross section built inside a metallic flume for intense load conditions. The whole channel is fabricated using D_{50} value of gravel size 6.5 mm for the intense load condition over bed. The roughness height is found to be 2.5 cm, which is estimated approximately from the number of layer (i.e. four in this case) of gravel, covered one upon another over bed. The slope of the flume is fixed at 0.0025 (0.25%) for all runs. Refer Table 2 for description of the bed load characteristics. Water for the experiment is provided from an overhead tank to which a water level indicator is attached for maintaining constant water level for test run discharge. 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.

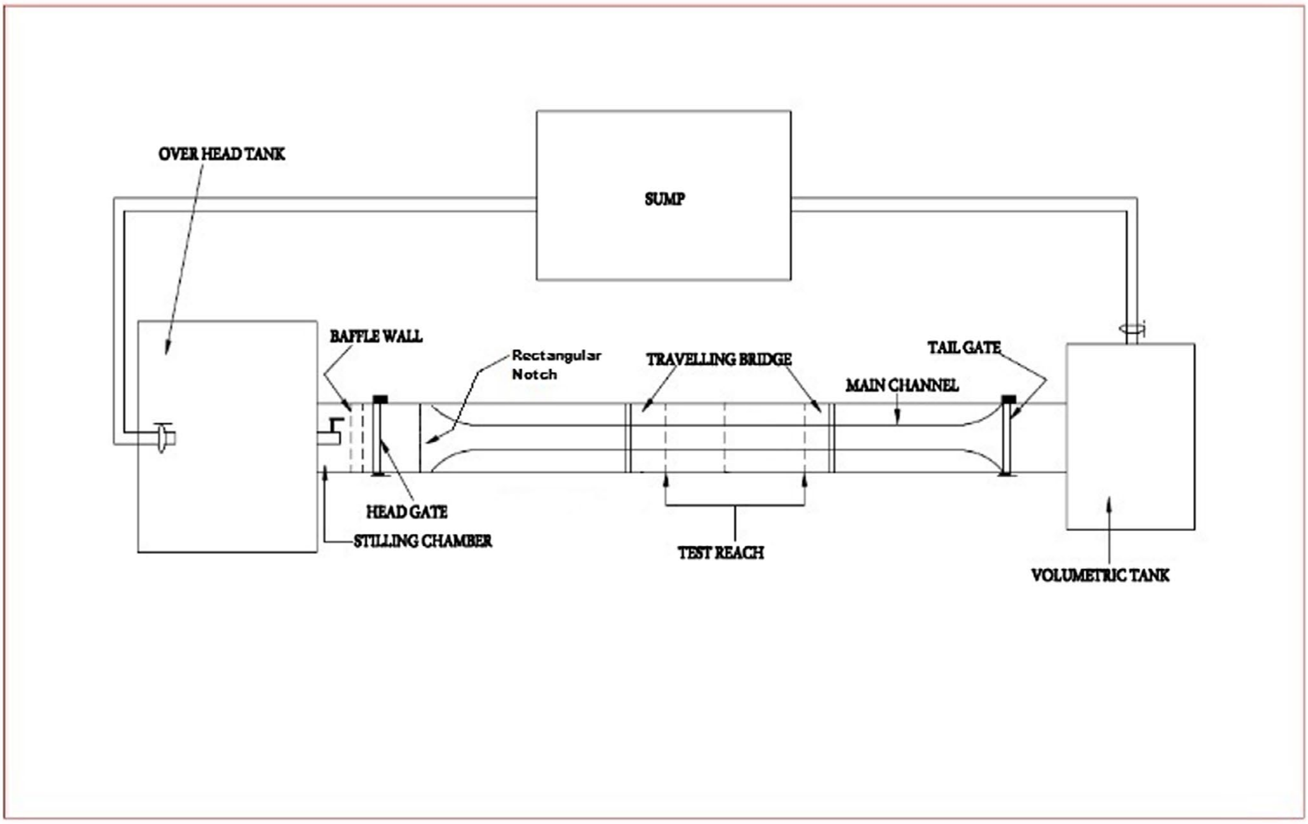
Main parameters to be measured during the present experiment are discharge, bed slope, depth of flow and the velocity of flow and boundary shear stress. The measurement procedure of these parameters is briefly described as follows. The depth of flow in the channel is measured using a point gauge fixed into the travelling bridge and operated manually. Point velocities are measured using a Micro-Pitot tube of 4.77 mm external diameter with a 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 normally to the mainstream direction until it gives a maximum deflection of manometer reading.

Table 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)	$\phi = 10.4\theta^{2.5}$ when $\theta \geq 0.068$
5.	Recking (2006)	$\phi = 14\theta^{2.45}$ when $0.08 < \theta < 0.25$

4. Model development

The measured values of geometric and hydraulic parameters are mentioned in Table 3 For all the experimental runs, the diameter of gravel was kept constant as 0.0065 m. Moreover, a constant bed slope of the channel (0.25%) maintain the subcritical flow conditions, which was used for all the runs. Total



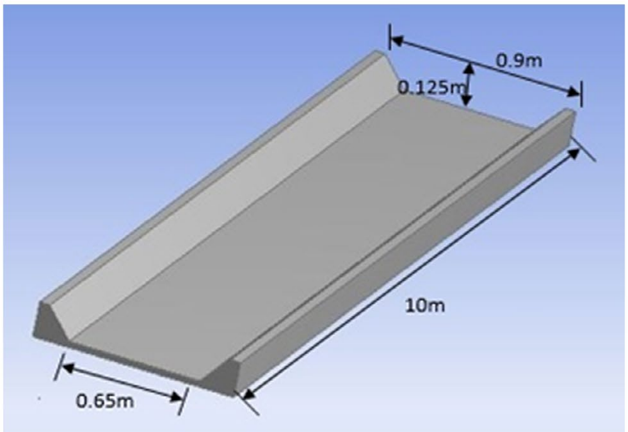


Figure 1. Plan view of the experimental channel and overall view of the flume with experimental set up.

Table 2. Detailed geometrical features for both the load conditions.

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

18 experimental runs for uniform flow conditions were taken for the study.

A linear regression model is developed to predict friction factor, f for gravel bed under uniform flow condition with the intense load Figure 2. Recking (2006) developed a model for predicting friction factor, f for the 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 0.08–0.25, 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 utilised in developing the new model.

Figure 2 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:

$$\sqrt{\frac{8}{f}} = 3.55\ \ln\left(\frac{R}{D}\right) + 0.2(R^2 = 0.92) \tag{18}$$

Table 3. Experimental data-set for intense load condition.

Discharge *Q* (m^3/sec)	Flow depth *h* (m)	Cross section area *A* (m^2)	Wetted perimeter *P* (m)	Hydraulic radius $R = A/P$	Mean channel velocity *u* (m/sec)	Relative Depth (*R/D*)	u/u^*
1	2	3	4	5	6	7	8
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

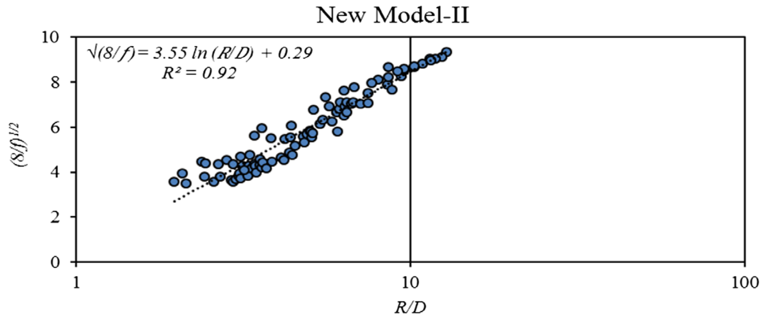


Figure 2. New friction factor model for intense load condition.

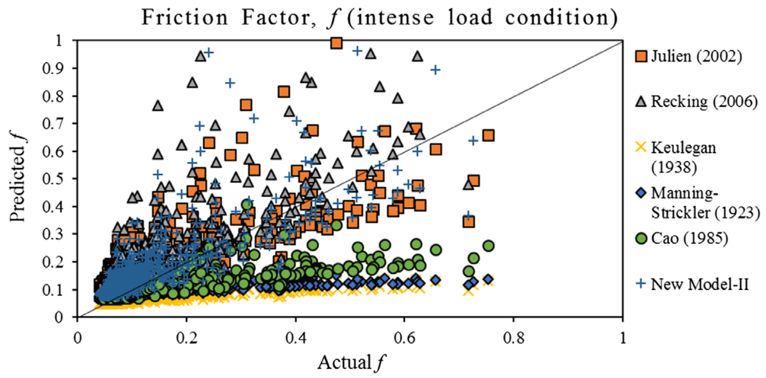


Figure 3. Validation of friction factor model for intense load condition with other author data.

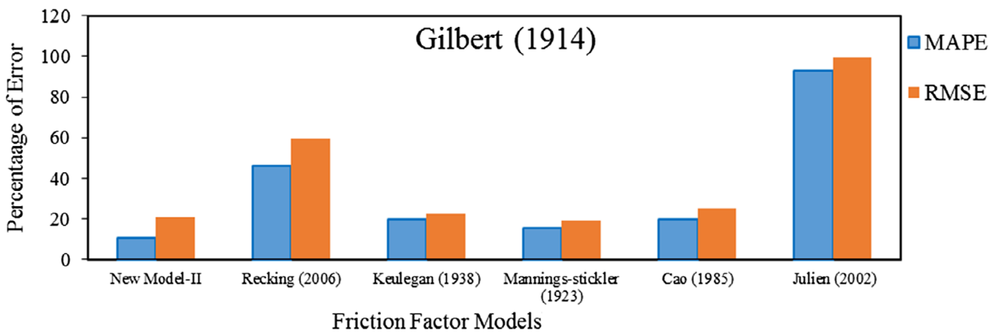


Figure 4a. Error analysis of friction factor model for intense load condition with Gilbert (1914) data.

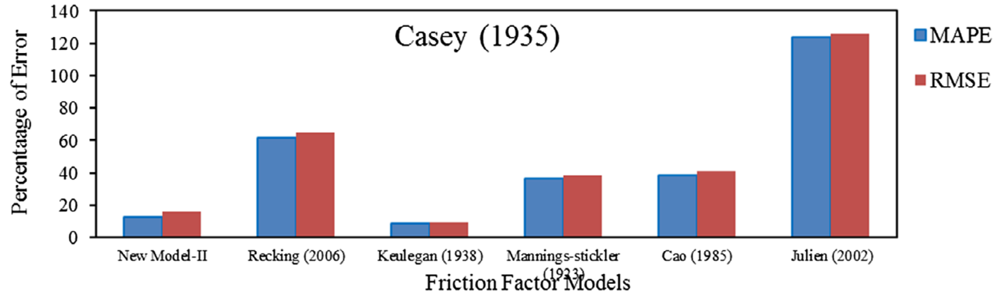


Figure 4b. Error analysis of friction factor model for intense load condition with Casey (1935) data.

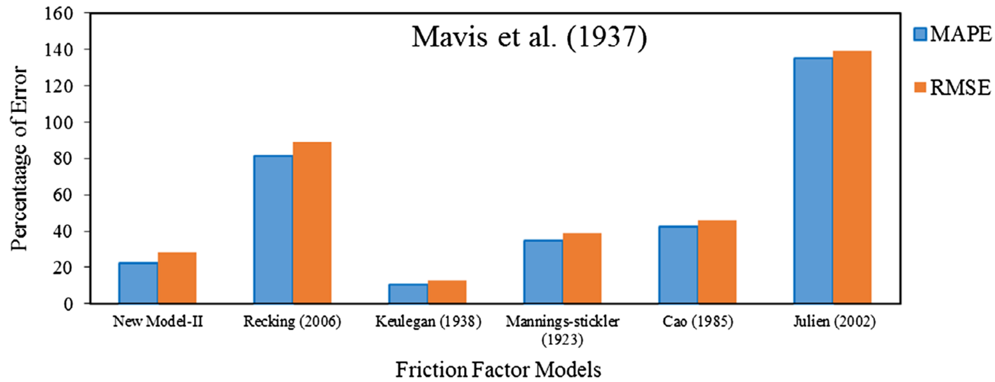


Figure 4c. Error analysis of friction factor model for intense load condition with Mavis et al. (1937) data.

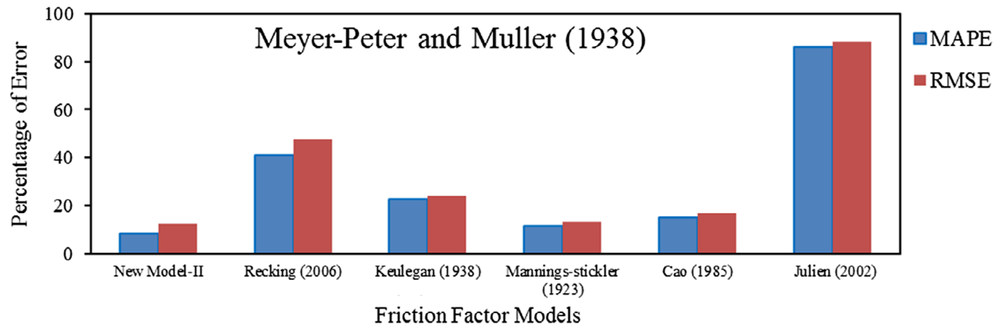


Figure 4d. Error analysis of friction factor model for intense load condition with Meyer-Peter and Muller (1948).

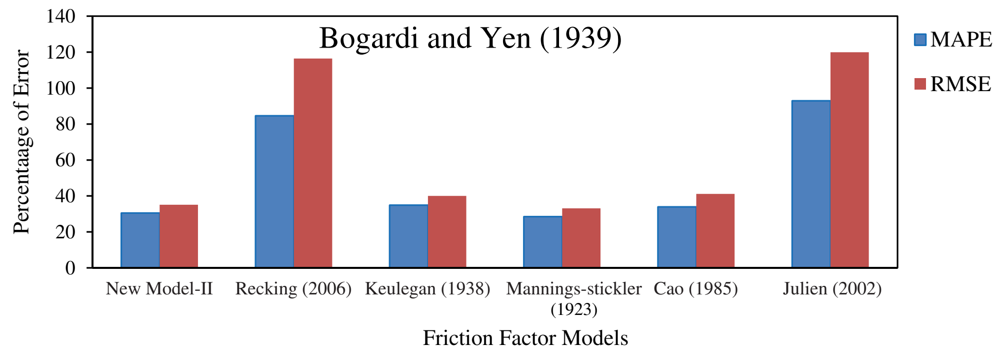


Figure 4e. Error analysis of friction factor model for intense load condition with Bogardi and Yen (1939).

The Equation (18) established above is verified with the data-sets of other investigators such as Bogardi and Yen (1939), Casey (1935), Gilbert (1914), Graf & Suzuka (1987), Ho (1939), Mavis et al. (1937), Paintal (1971), Recking (2006), Rickenman (1990), Smart and Jaeggi (1983) along with the experimental observations of present study. The total number of data-sets being 666 taken with experimentation as well as from other authors.

The functionality of the models suggested by previous researchers has also been evaluated for all of the available data-sets. Figure 3 shows the contrast between different models through predicted values of friction factor versus the

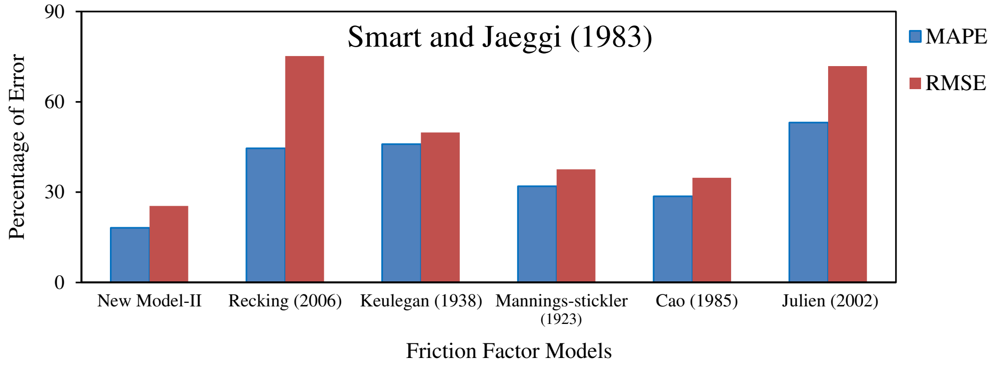


Figure 4f. Error analysis of friction factor model for intense load condition with Ho (1939).

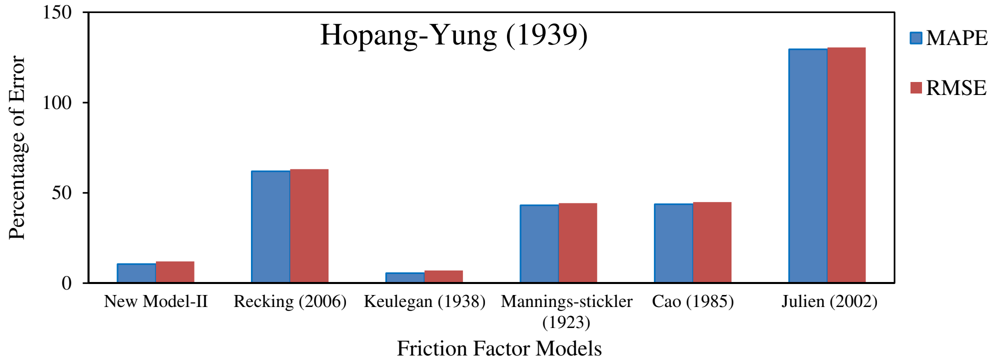


Figure 4g. Error analysis of friction factor model for intense load condition with Smart and Jaeggi (1983) data.

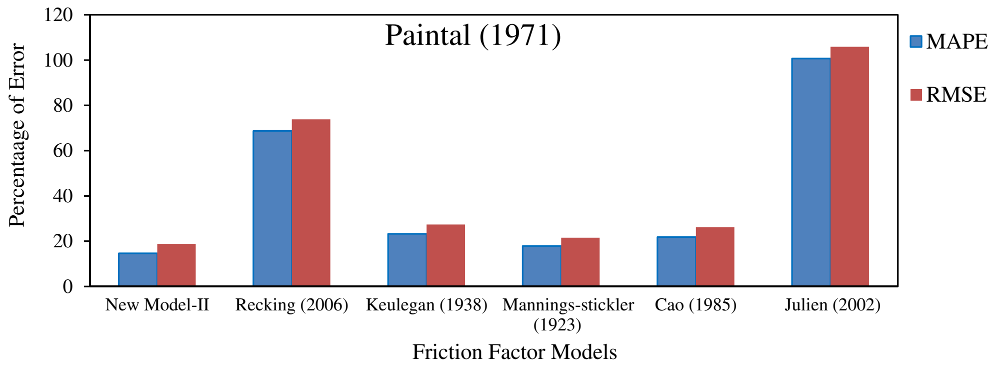


Figure 4h. Error analysis of friction factor model for intense load condition with Paintal (1971).

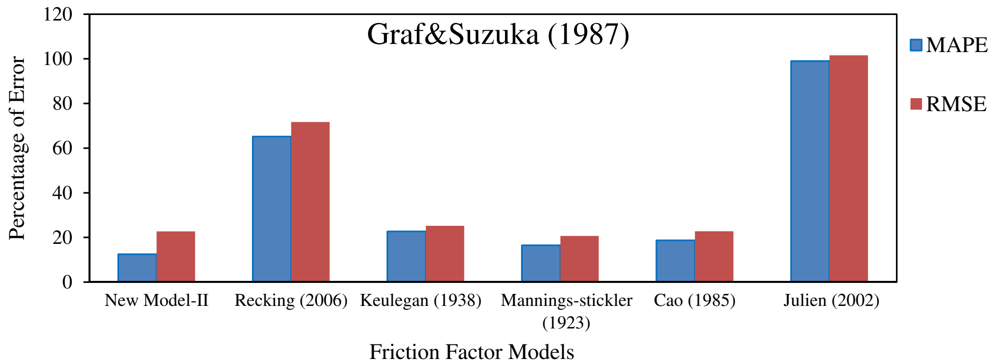


Figure 4i. Error analysis of friction factor model for intense load condition with Graf & Suzuka (1987) data.

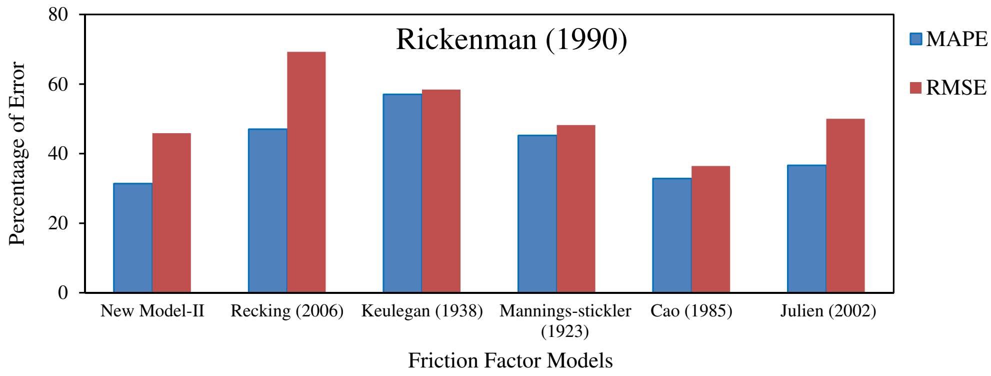


Figure 4j. Error analysis of friction factor model for intense load condition with Rickenman (1990) data.

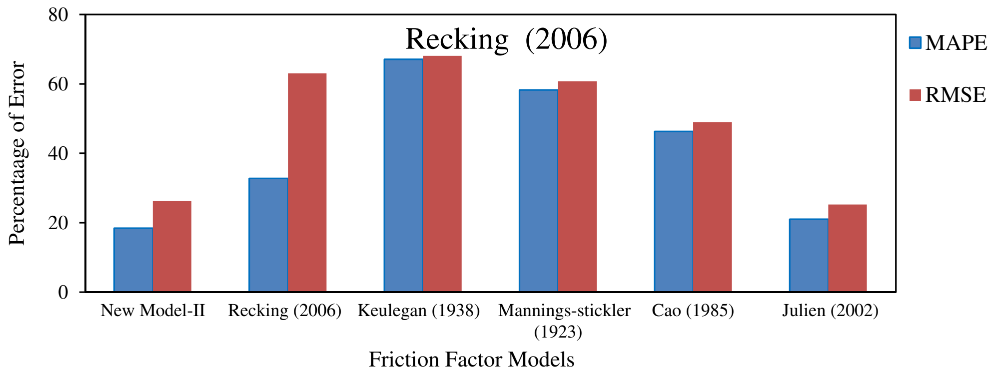


Figure 4k. Error analysis of friction factor model for intense load condition with Recking (2006) data.

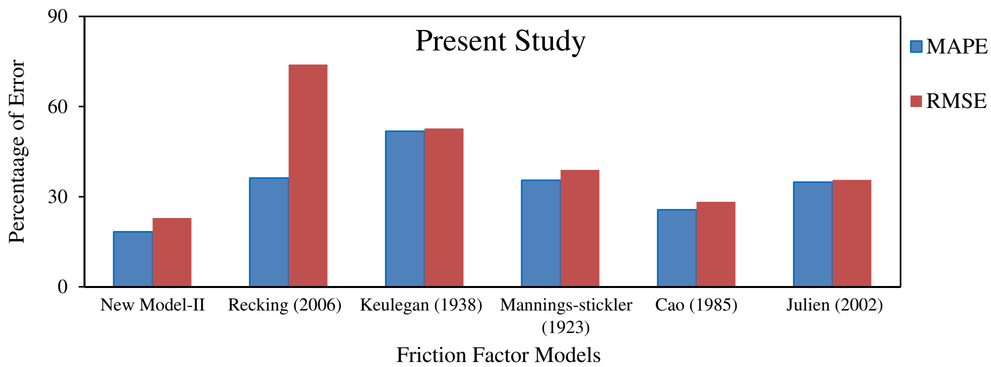


Figure 4l. Error analysis of friction factor model for intense load condition with Present study data.

measured value of friction factors. The present model on whole is observed to provide better predictions.

4.1. Error analysis of friction factor model for intense load flow condition

The error analysis of the different models is carried out for 12 data-sets of intense load conditions given in Figure 4(a)–(l). In almost all the sets of data, the developed expression is observed to provide with lower values of mean absolute percentage error (MAPE) as well as root-mean-square error (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), Ho (1939) but concurrently gives higher values for the other data-sets. The above observations are 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 newly developed expression can be accepted to give a satisfactory prediction of friction factor for intense load conditions.

4.2. Bed load intensity for intense load flow condition

In the present study, two parameters are 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. In addition, another part of this present study was bed load intensity, which is calculated with the help of bed load sediment transport rate.

All the remaining parameters were same and taken from the stage-discharge data of intense load conditions. For all the experimental runs, the D_{50} diameter of gravel was kept constant

Table 4. Experimental data-set of bed load intensity for intense load condition.

Q (m³/sec)	Flow depth h (m)	Cross section area A (m²)	Wetted perimeter P (m)	Hydraulic radius $R = A/P$	Mean channel velocity V (m/sec)	Relative depth (R/D)	u/u^*	Shields parameter θ	Φ^*
1	2	3	4	5	6	7	8	9	10
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	0.1347	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

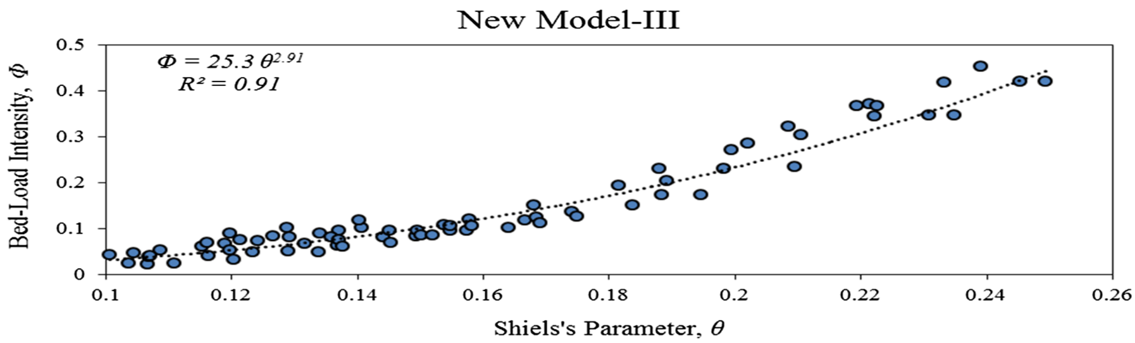


Figure 5. Bed load intensity model for intense load condition.

as 0.0065 m. 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 4 shows the experimental results. Graph between bed load intensity and Shield's parameter is plotted which best fit on power law shown in Figure 5. 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 0.08 to 0.25. 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 observation, including that of Recking (2006) has been utilised in developing the new model.

$$\Phi = 25.3\, \theta^{2.91} \left(R^2 = 0.91\right) \tag{19}$$

Figure 5 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 (19). The model developed in the previous section is validated with the data-sets of other researchers such as Wong and Parker (2006), Ashmore (1988), Recking (2006), Graf and Suzuka (1987) etc. along with the present experimental observations. The number of data-sets being 253 in total. The performance of models suggested by previous investigators has also been analysed for the above data-sets.

Figure 6 shows the performance of different models through 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 as shown in Figure 7.

4.3. Error analysis of bed load intensity model for intense load flow condition

Bed load intensity is calculated using different models developed by the researchers and the values of MAPE and RMSE

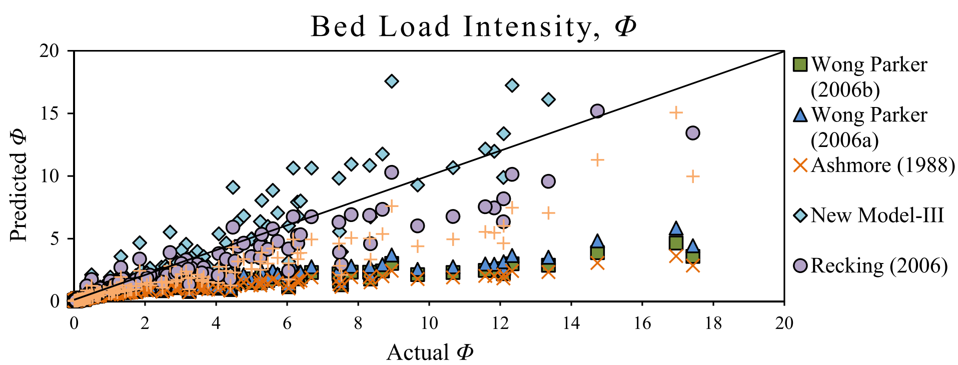


Figure 6. Validation of bed load intensity model for intense load condition with other author data.

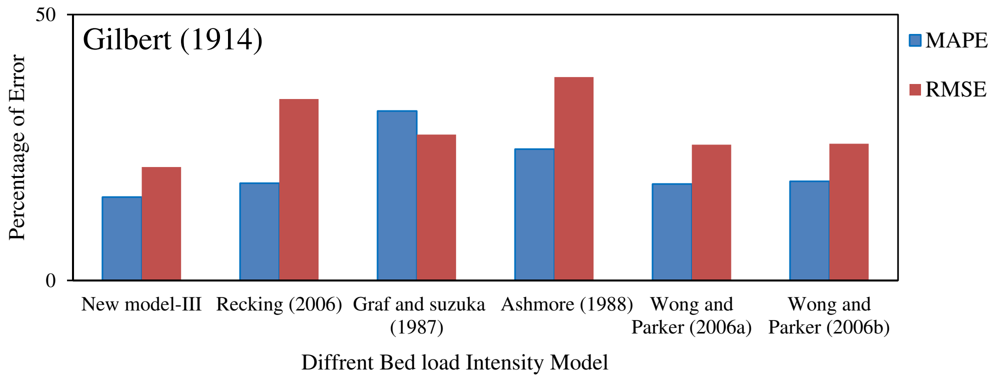


Figure 7a. Error analysis of bed load intensity model for intense load condition with Gilbert (1914) data.

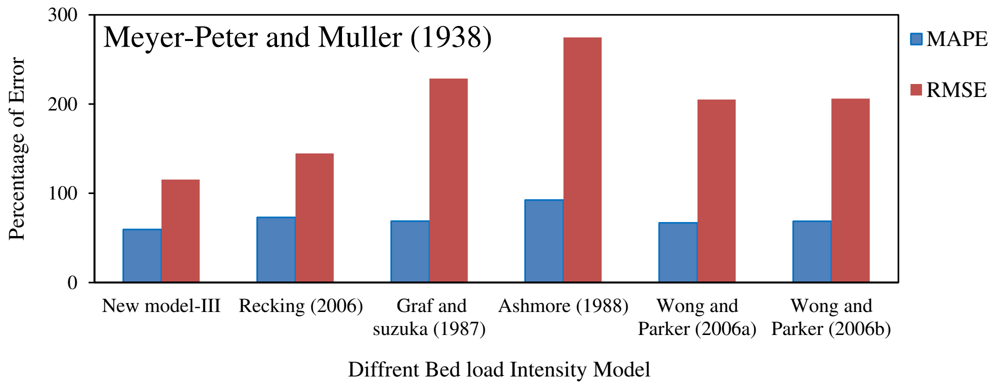


Figure 7b. Error analysis of bed load intensity model for intense load condition with Meyer-Peter and Muller (1948) data.

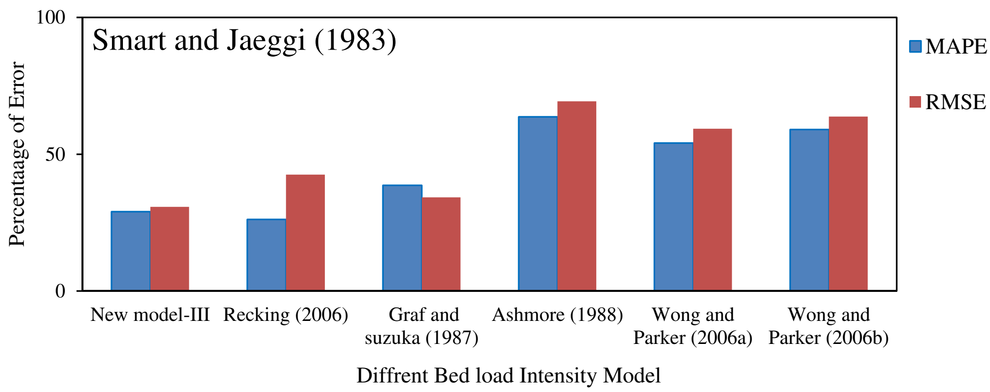


Figure 7c. Error analysis of bed load intensity model for intense load condition with Smart and Jaeggi (1983) data.

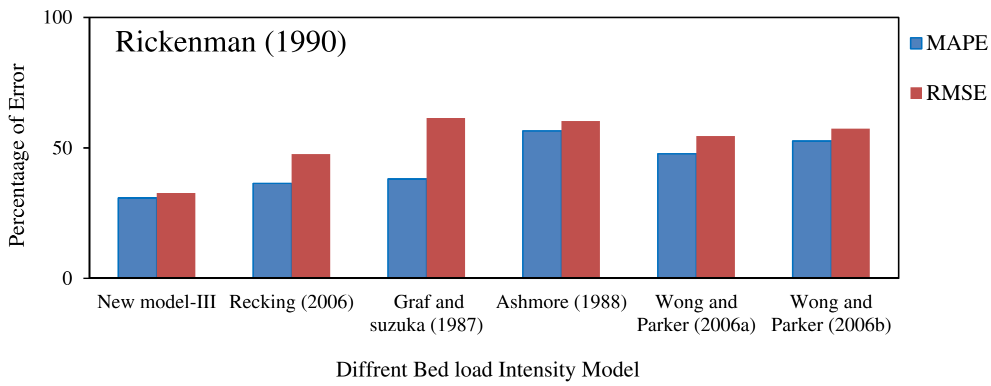


Figure 7d. Error analysis of bed load intensity model for intense load condition with Rickenman (1990) data.

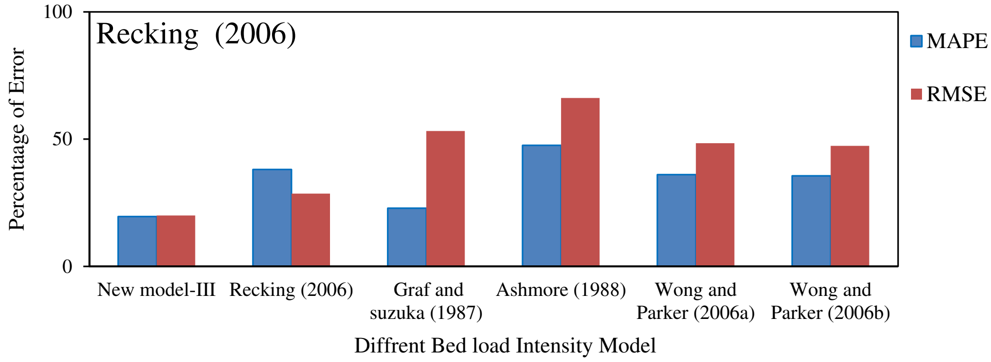


Figure 7e. Error analysis of bed load intensity model for intense load condition with Recking (2006) data.

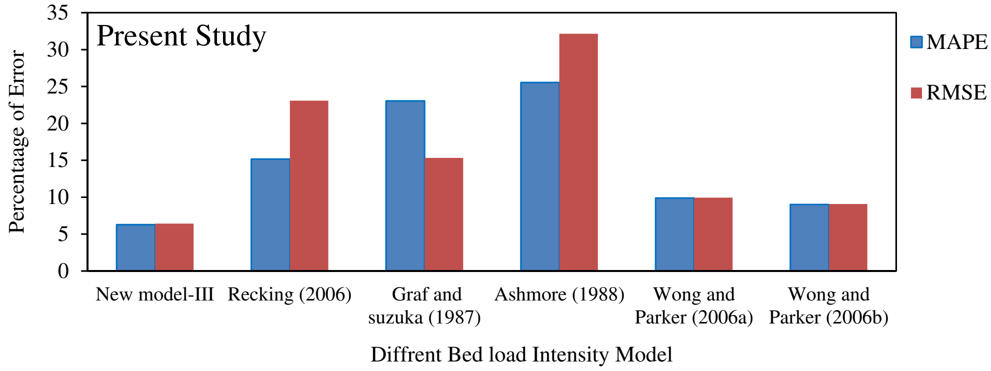


Figure 7f. Error analysis of bed load intensity model for intense load condition with present study data.

are demonstrated in Figure 7(a)–(f). The models developed are a power function of the shield's parameters, θ that depend on the boundary shear stress.

Total six number of data-sets 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 (19) gives lower values of MAPE and RMSE for all the available data. The values of RMSE and MAPE for the case of, is quite high for all the prediction models, it is because of the higher range of gravel size used i.e. 1.4–28.65 mm for a mild slope of 0.1–0.5%. 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 the lower size of gravel weight with a mild slope, with the present expression provided the better results.

5. Conclusions

Types of sediment are considered for the purpose of the present study is smaller. Following salient findings are obtained from the present research work.

- Bed load intensity and friction factor are found to be functions of boundary shear stress, sediment transport rate, velocity, discharge, shear velocity, hydraulic radius, grain size diameter, the slope of the channel and Shield's parameter.
- The model developed for friction factor under intense load conditions when validated with others model, is found to give comparatively less erroneous values than previous models.
- A bed load intensity model established for the case intense bed load transport condition is observed to provide better results in comparison to that of the other models.
- The linear regression curve shows a higher value of correlation coefficient, indicating a satisfactory relation between the two in friction factor calculation of intense load.
- The power law curve shows a higher value of correlation coefficient, indicating a satisfactory relation between the two for bed load intensity and Shield parameter.

Notations

d Diameter of the Preston tube

d^* Dimensionless particle parameter

D Diameter of the gravel

D_{50} 50% grain size diameter at

f Darcy–Weisbach's friction factor

g Acceleration due to gravity

q_b Bed load sediment transport rate

Q Channel discharge

R Hydraulic radius, defined as flow area/wetted perimeter

R^2 Coefficient of Determination

S Channel bed slope

u Measured velocity by the manometer
u^* Shear velocity
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
ν Kinematic viscosity of water
z Distance between the free surface with arbitrary datum
z_0 Fraction of the bed roughness
ρ Density of water
$\mathbf{\Phi}$ Dimensionless intensity of the bed load rate
$\boldsymbol{\gamma}$ Specific weight of water
$\boldsymbol{\gamma}_s$ Specific weight of sediment

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Disclosure statement

No potential conflict of interest was reported by the authors.

References

Ashmore, P.E. (1988). “Bed load transport in braided gravel bed stream models.” *Earth Surface Proc. Landforms*, 13, 677–695.
Bathurst, J.C., Graf, W.H., and Cao, H.H. (1982). “Initiation of sediment transport in steep channels with coarse bed material.” *Euromech 156: Mech. Sediment Transport, Istanbul*, 207–213.
Bergeron, N.E., and Carbonneau, P. (1999). “The effect of sediment concentration on bedload roughness.” *Hydrol. Proc.*, 13, 2583–2589.
Bogardi, J., and Yen, C.H. (1939). “Tractation of pebbles by flowing water.” *State Univeristy of Iowa*.
Campbell, L., McEwan, I., Nikora, V.I., Pokrajac, D., Gallagher, M., and Manes, C. (2005). “Bed-load effects on hydrodynamics of rough-bed open-channel flows.” *J. Hydraul. Eng.*, 131(7), 576–585.
Cao, H.H. (1985). “Resistance hydraulique d’un lit à gravier mobile à pente raide; étude expérimentale.” PhD thesis. Ecole PolytechniqueFederale de Lausanne, Lausanne.
Cao, Z., Pender, G., and Meng, J. (2006). “Explicit formulation of the shields diagram for incipient motion of sediment.” *J. Hydraul. Eng.* 132(10), 1097–1099.
Carbonneau, P., and Bergeron, N.E. (2000). “The effect of bed load transport on mean and turbulent flow properties.” *Geomorphology*, 35, 267–278.
Cardoso, A.H., Graf, W.H., and Gust, G. (1989). “Uniform flow in s smooth open channel.” *J. Hydraul. Res.*, 27(5), 603–616.
Casey, H.J. (1935). “Uber Geschiebebewegung, Preuss.” *Versuchsanst. fur Wasserbau und Schifibau, Berlin*, 19, 86 pp.
Einstein, H.A. (1942). “Formulas for the transportation of bed-load.” *Trans. Am. Soc. Civ. Eng.*, 107, 561–597.
Einstein, H.A. (1950). *The Bed-Load function for sediment transportation in open channel flows*, United States Department of Agriculture – Soil Conservation Service, Whashington, DC, 71.
Einstein, H.A., and Barbarossa, N.L. (1951). “River Channel Roughness.” *Am. Soc. Civil Eng.*. Paper N°2528, 1121–1146.
Gilbert, G.K. (1914). “The transportation of Debris by running water.” *US Geological Survey*, No. 86.
Graf, W.H., and Suszka, L. (1987). “Sediment transport in steep channels.” *J. Hydrosci. Hydraul. Eng.*, 5(1), 11–26.
Hey, R.D. (1979). “Flow resistance in gravel bed rivers.” *J. Hydraul. Div.*, 105(4), 365–379.
Ho, P.Y. (1939). “Dependence of the sediment movement on the grain shape and the temperature.” *Preuss. Versuchsanst. fur Wasserbau und Schifibau, Berlin*, 37, 43 pp.
Julien, P.Y. (2002). “River mechanics.” *Cambridge University Press*, 434.
Keulegan, G. B. (1938). “Laws of turbulent flows in open channels.” *J. Res. Natl. Bureau Stand.*, 21, 707–741.
Mahdavi, A., and Omid, M. (2004). “The effect of bed roughness on velocity profile in open channels.” *River Flow Naples Italy*, 295–300.
Mavis, F. T., Liu, T., and Soucek, E. (1937). “The transportation of detritus by flowing water.” *IIHR Bull.* 11, University. of Iowa, Iowa City.
Meyer-Peter, E., and Muller, R. (1948). “Formulas for Bed-Load Transport.” *IAHSR, Stockholm*, 39–64.
Nikora, V.I., and Smart, G.M. (1997). “Turbulence characteristics of the New Zealand gravelbed rivers.” *J. Hydraul. Eng.*, 123(9), 764–773.
Nikuradse, J. (1933). “Strömungsgesetze in rauhen Rohren. Forsch.” *Arb. Ing. - Wes*, 361.
Omid, M., Mahdavi, A., and Narayanan, R. (2003). “Effects of bed load transport on flow resistance in rigid boundary channels.” *IAHR, Tessalonic*, IAHR, ed., 641–646.
Paintal, A.S. (1971). “Concept of critical shear stress in loose boundary open channels.” *J. Hydraul. Res.*, 1, 90–113.
Recking, A. (2006). “An experimental study of grain sorting effects on Bedload.” Ph.D. thesis, French engineering university INSA Lyon.
Rickenmann, D. (1990). Bed load transport capacity of slurry flows at steep slopes.” *Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie der Eidgenössischen, Zurich*.
Sheilds, A. (1936). “Anwendung der Ahnlichkeitsmechanik undTurbulenz-forschung auf Geschiebebewegung.” *Mitteilungender Preuss. Versuchsanst. f. Wasserbau u, Schiffbau, Heft, Berlin*, 26.
Smart, G. (1999). “Turbulent velocity profiles and boundary shear in gravel bed rivers.” *J. Eng.*, 125(2), 106–116.
Smart, G., and Jaeggi, M. (1983). “Sediment transport on steep slopes.” *Nr.64, Mitteilungen der Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie, Zurich*, 191.
Smart, G.M., Duncan, M.J., and Walsh, J.M. (2002). “Relatively rough flow resistance equations.” *Journal of Hydraulic Engineering*, 128(6), 568–578.
Song, T., Graf, W.H., and Lemmin, U. (1995). “Uniform flow in open channels with a movable gravel bed.” *J. Hydraul. Res.*, 32(6), 861–875.
Song, T., Chiew, Y.M., and Chin, C.O. (1998). “Effects of bed-load movement on flow friction factor.” *J. Hydraul. Eng.*, 124(2), 165–175.
Van Rijn, L.C. (1984). “Sediment transport, Part I: Bed load transport.” *J. Hydraul. Eng.*, 110(10), 14311457.
Whiting, P.J., and Dietrich, W.E. (1990). “Boundary shear stress and roughness over mobile Alluvial beds.” *Journal of Hydraulic Engineering*, 116(12), 1495–1511.
Wiberg, P., and Rubin, D.M. (1989). “Bed roughness produced by saltating sediment.” *J. Geophys. Res.*, 94(C4), 5011–5016.
Wong, M., and Parker, G. (2006). “Re-analysis and correction of bed load relation of Meyer-Peter and Muller using their own database.” *J. Hydraul. Engng.*, 132, 1159–1168.
Wong, M., and Parker, G. (2006a). “One-dimensional modeling of bed evolution in a gravel bed river subject to a cycled flood hydrograph.” *J. Geophys. Res.*, 111, F03018.
Wong, M., and Parker, G. (2006b). “One-dimensional modeling of morphodynamic bed evolution in a gravel-bed river subject to a cycled flood hydrograph.” *J. Geophys. Res.*, 111.
Wu, W., and Wang, S.Y. (1999). “Movable bed roughness in Alluvial rivers.” *J. Hydraul. Eng.*, 125(12), 1309–1312.
Yang, C.T. (1996). *Sediment transport theory and practice*, McGraw-Hill Series in Water Resources and Environmental Engineering, Singapore. ISBN 0-07-114882-5.