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Determination of apparent and composite friction factors for flooded equatorial natural rivers

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ABSTRACT
This paper presents results, calculated from field measurements taken in several frequently flooded natural rivers, which include D and R relationships, variation of flow resistance with depth of flow, the apparent friction factor, and the composite friction factor for flooded natural rivers. The results obtained have shown the complexity of flow resistance in natural rivers due to the interaction between the main channel and floodplain flow. The interaction has given rise to a pair of apparent shear stresses at the interface region, which can significantly reduce the discharge capacity of the rivers. The apparent shear was quantified in terms of an apparent friction factor, f_a, and it was found that the apparent shear stress is many times greater than the averaged boundary shear stress of the rivers. Based on the averaged boundary shear stress and apparent shear stress, the composite (actual) friction factor for the rivers can be estimated accurately ($R^2 = 0.99$) using a statistical method that had been derived.

Keywords: Flow resistance; friction factor; momentum transfer; natural river; overbank flow.

1 Introduction

The estimation of resistant coefficient and hence discharge capacity in a channel or river is one of the fundamental problems facing the river engineers. Without an accurate estimate of conveyance, very little confidence can be placed in the subsequent design calculations or predictions.

At the present moment, the accuracy of the friction factor for predicting flow characteristics in a particular reach, with a dynamic vegetation and flow regime remains questionable. Many studies of flow resistance have been carried out to provide an accurate estimate of resistance coefficient in any given circumstance especially under overbank conditions. However, none as yet has lead to a general applicable method. In addition, as most of this work is based on laboratory experiments, these results may not reflect the real situations in natural rivers with highly irregular shape and variations in surface roughness. In the work presented, an attempt was made to focus on the estimation of flow resistance in natural rivers under flood conditions.

2 Reviews

An important component in open channel flow is the estimation of flow resistance resulting from the viscous and pressure drag over the wetted perimeter. Such resistance is commonly represented by parameters such as Manning's roughness coefficient (n), or the Darcy-Weisbach friction factor (f), as given below.

The Manning equation gives

$$n = (R^{2/3} S_o^{1/2})/V \tag{1}$$

The Darcy-Weisbach equation for channel flow gives

$$f = (2gDS_o)/V^2 \tag{2}$$

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where n is the Manning roughness coefficient, f is the Darcy-Weisbach friction factor, R is the hydraulic radius $= A/P$, A is the cross sectional area, P is the wetted perimeter, D is the hydraulic diameter, S_o is the bed slope, g is gravitational acceleration, and V is the average cross-sectional velocity.

In applying the Manning and the Darcy-Weisbach formulas, the greatest difficulty lies in the determination of the roughness coefficient n, and friction factor, f, for there is no exact method of selecting the n and f values. The Task Force on Friction Factors in Open Channel [1] found that Manning's formula with a constant n-value is only applicable to fully rough turbulent flow. Additionally, employing Manning's formula in heavily vegetated or urbanized areas is particularly problematic [2].

Some guidance is available for resistance coefficient estimation from a variety of sources, most accessibly in Chow [3] and French [4]. Chow contributed a series of tables presenting the values of Manning's n for a variety of river conditions. These are supplemented by photographs of rivers for which resistance coefficients have been measured. French presented a more rational approach in the form of the US Soil Conservation Method, which involves identifying a basic n value depending on the bed and bank material of the river. An advantage of French's approach is that it assists the engineer to identify the many factors, which influence flow resistance in a natural river. These include irregularities in cross-sectional and platform geometry, together with roughness arising from bed and bank material. Such data, however, are relevant to simple channel shapes, and may lead to serious error when extrapolated to overbank flow depths [5–8].

Apart from the resistance factors in open channels, the behaviour of resistance coefficients under flood conditions are expected to be much more complex, not only because of the spatially variable geometrical parameters and boundary roughness along the channel, but also due to the complexity of the phenomenon occurred [6, 9, 10].

2.1 *Apparent friction factor*

One of the major problems in relation to overbank flow is that rivers may occupy, at flood discharges, a compound cross-section, consisting of a deep main channel with associated shallow floodplains. In this case, there may be a sudden change of depth at the transition between the main channel and the floodplain. Moreover, the hydraulic roughness of the floodplain is often greater than that of the main channel. The combined effects of the greater depth of flow and smaller hydraulic roughness of the main channel can lead to significantly higher velocity than that occurring on the floodplain. This velocity difference results in a pair of apparent shear stresses at the interface regions. These apparent shear stresses have the same magnitude but act in opposite directions as shown in Figure 1.

Measurements of boundary shear stresses have made possible the calculation of apparent shear stresses on these interface planes. Several investigations have studied the apparent shear force or stress on the (imaginary) interface between main channel and floodplain and showed that apparent shear stress on the channel floodplain interface is many times greater than the

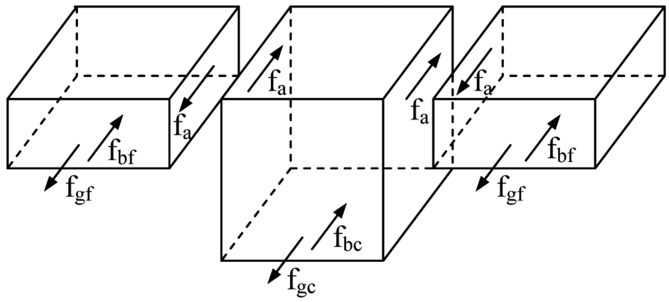

Figure 1 Exploded view of compound section showing forces acting on subdivisions [11].

average shear stress around the solid boundaries, e.g. Wormleaton *et al.* [11]; Knight and Demetriou [12]; Baird and Ervine [13]; Wormleaton and Merrett [14]. In an extreme case, Stephenson & Kolovopoulos [15] shows that it can be 260% greater than that of the averaged boundary shear stress.

According to Wormleaton *et al.* [11], the apparent shear stress increases with a decrease in floodplain depth and with an increase in floodplain roughness. For the lower floodplain depth, the apparent shear stress becomes very much larger than the averaged boundary shear stress, and thus the assumptions of the apparent shear stress being equal to zero (no apparent shear), or even to the averaged boundary shear stress, ceased to be valid. Therefore, at these low floodplain depths, the momentum transfer effect is far greater than allowed for in the previously mentioned conventional methods of discharge calculation. This, in turn, means that they will tend to overestimate channel discharge capacity.

Knight and Hamed [16] presented experimental results of boundary shear stress and boundary shear force distribution in a compound section comprising of one rectangular main channel and two symmetrically disposed floodplains. Equations were presented giving the shear force on the floodplains as a percentage of the total shear force in terms of four dimensionless parameters. Supplementry equations were also presented giving the apparent shear force on vertical, inclined and horizontal interfaces within the cross section. The influence of momentum transfer between sub areas on the vertical and lateral distribution of longitudinal velocity was also assessed. The discharge and apparent shear force results revealed certain weaknesses in the four commonly adopted design methods (SCM, HDCM, VDCM and DDCM) used to predict the discharge capacity of compound channels.

In relation to the four commonly accepted design methods, the Single Channel Method (SCM) treats the compound geometry as a single unit, assigning single values of roughness and hydraulic radius. This method has been shown by Myers and Brennan [6] and Myers *et al.* [8] to be significantly in error at low over-bank flow depths, leading to an underestimation of discharge capacity. However, this assumption can be seen to improve with increasing depth. The Divided Channel Methods (HDCM, and VDCM) divide the cross-section into main channel and flood plain zones using straight horizontal/vertical lines, and each zones is then treated separately. Neither method takes account of momentum transfer across the interface between main channel and flood plain. Myers [17] has shown that these methods tend to under-and over-estimate the discharge in the main channel. Myers *et al.* [8] further shows that the DCMs overestimate compound discharge, with errors of up to 45% in the rough flood plain case. Smooth main channel and floodplain are more accurately modeled by these methods with 10% error. While the river discharge errors peak at almost 30%. Whereas the DDCM [18] uses diagonal lines/inclined divisions to compensate for the under-and over-estimation of the DCM methods. However, little success has been achieved, due to the difficulty to generalize the position of these shear free division lines for all types of channel shapes.

Further investigations such as by Nalluri and Judy [19], and Christodolou and Myers [20] for example, have led to the derivation of empirical relationships by which the additional flow resistance in a compound channel can be estimated as a function of geometrical parameters and the velocity deficit between the floodplain and the main channel. These empirical relationships, as well as experimental data, have shown that the vegetative resistance varies with the flow depth or the degree of submergence.

A major series of resistance in compound channel researches in recent years were those of the large-scale Flood Channel Facility (FCF) in United Kingdom, which can be divided into 3 phases. Phase A of the FCF programme centred on straight and skewed fixed boundary compound channels, and the results of this have been presented by Myers & Brennan [6], Wormleaton & Merritt [14], Knight & Shiono [21], and Elliott & Sellin [22]. Phase B explored meandering platforms having fixed boundary and whivh has been reported by Sellin *et al.* [23], Ervine *et al.* [24], Greenhill & Sellin [25]. Phase C of this experimental programme included investigations of the straight and meandering platforms with mobile boundaries and the results have been presented by Myers *et al.* [7], Knight *et al.* [26]. These studies together, with those that have been described earlier, have provided invaluable insights into the interaction between the main channel and flood plain, and show that flow resistance characteristics in compound channels are complex and do not conform to the representations described by Eqs (1) and (2).

At the present moment, there is insufficient data to provide general guidance on the choice of resistance coefficient in any given circumstance, especially for flooded natural rivers. Added to which is the fact that most of these data represent laboratory studies, which may not be applicable to river geometries with different roughness and Reynolds numbers. Clearly, more reliable methods of analysis are needed for flooded natural rivers.

Figure 2 Morphological cross-section of River Senggai.

Figure 3 Morphological cross-section of River Senggi (B).

Figure 4 Morphological cross-section of River Batu.

3 Field study and data collection

The present study [28] was carried out in three natural rivers namely River Senggai, River Senggi (B) and River Batu located in Kuching, the capital city of Sarawak state, Malaysia. These rivers were selected due to serious flood occurrence during the monsoon season in the past few years. Extensive flood data from River Main [27] in North Ireland was also obtained for comparison.

The selected rivers are shown in Figures 2–5. They show that the rivers are almost straight and uniform in cross section, free from backwater and tidal effect. Table 1 shows the geometrical properties and surface conditions of the rivers at the gauging stations for comparison.

Flow gauging of the rivers was carried out from an adjustable bridge built across the rivers, using the velocity-area method, in which an electromagnetic flow meter was used to measure point velocity at 20%, 40%, 60% and 80% of flow depth at up to 20 verticals across the sections. The flow depths and point velocities were measured to an accuracy of 0.0005m (0.5 mm)

Figure 5 Morphological cross-section of River Main.

and 0.0001m/s respectively. For each measuring point, 3–5 reading were taken and averaged to give a mean point velocity. This was to reduce the error due to variation in water flow. Some 20 discharges were recorded for each river, covering a wide range of inbank and overbank flows. Table 2 presents values of measured depth and discharge collected from measurement for River Senggai.

4 Theory considerations

The Darcy-Weisbach equation is commonly used to express flow resistance in open channels, following the recommendation of the American Society of Civil Engineers (ASCE) in 1963 [1].

Table 1 Geometrical properties and surface conditions.

Geometrical properties	River Senggai	River Senggi (B)	River Batu	River Main
Bankfull depth, h (m)	1.060	1.306	1.544	0.900
Top width, B (m)	5.285	5.500	5.150	13.700
Aspect ratio, B/h	4.986	4.211	3.335	15.222
Bed slope – main channel , S_0	0.0010	0.0010	0.0016	0.0030
Bed slope – left floodplain, S_L	0.0010	0.00085	0.0013	0.0030
Bed slope – right floodplain, S_R	0.0010	0.00085	0.0013	0.0030
Surface condition – main channel	Erodible soil	Erodible soil	large boulder	coarse gravel
Surface condition – side bank	Erodible soil	long vegetation	Erodible soil	large boulder
Surface condition – floodplain	long vegetation	long vegetation	long vegetation	short vegetation

Table 2 Ranges of measured depths and discharges for River Senggai.

Depth, H, (m)	(H−h)/H	Main channel discharge, Q_{mc} (m^3/s)	Flood plain discharge, Q_{fp} (m^3/s)	Total discharge, Q_t, (m^3/s)
0.664	−0.596	0.2838	0	0.2838
0.770	−0.377	0.4138	0	0.4138
0.805	−0.317	0.3845	0	0.3845
0.855	−0.240	0.4573	0	0.4573
0.885	−0.198	0.5360	0	0.5360
0.968	−0.095	0.6855	0	0.6855
1.010	−0.050	0.7600	0	0.7600
1.048	−0.011	0.8534	0	0.8534
1.068	0.008	0.9034	0	0.9034
1.128	0.060	0.8547	0	0.8547
1.155	0.082	0.8975	0	0.8975
1.174	0.097	1.0278	0	1.0278
1.195	0.113	1.0852	0	1.0852
1.228	0.137	1.2409	0	1.2409
1.265	0.162	1.4649	0	1.4649
1.288	0.177	1.6008	0	1.6008
1.365	0.223	2.2069	0.0463	2.2532
1.480	0.284	2.8008	0.0649	2.8657
1.550	0.316	3.1797	0.0876	3.2673

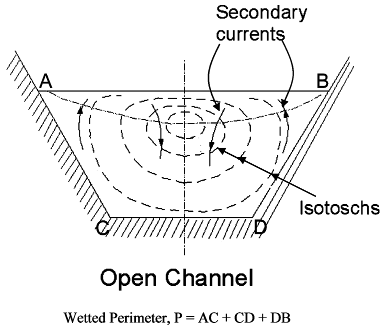

Figure 6 Comparison of open channel flow with closed-conduit flow.

Therefore the Darcy-Weisbach friction factor, f is used to denote the resistance coefficient in this study.

For the use of the Darcy-Weisbach equation, an open channel is usually considered to be a conduit as shown in Figure 6. The appropriate hydraulic diameter, D would then be assumed equal to $4R$. However, it has been shown elsewhere [28] that the equivalence is not exact. In fact the value of D varies proportionately with the width and depth of the channels.

4.1 Expression for D

An expression of D for flow in open channel can be developed, based on the force balance equation

$$\tau_0 = \frac{A\Delta p}{Pl} = \rho gSR = \rho u_*^2 \tag{3}$$

where A is flow area $= \int_0^B h(y)dy$, P is the wetted perimeter $= \int_0^B \sqrt{1+[h'(y)]^2}dy$, Δp is the pressure drop along the length $= \rho f \frac{\overline{V^2}}{2}\frac{l}{h_0}$, $h(y)$ is the depth of flow at a distance y from the bank, $h'(y) = dh/dy$, $\overline{V^2}$ is the sectional average velocity $= \frac{1}{A}\int u^2 dA$, u_* is the shear velocity $= gSR$. B is the top width.

Consider the friction for a vertical line: $f(y)$, then h_0 becomes $h(y)$, $\overline{V^2}$ becomes $\overline{u^2}$ on the vertical line, $\overline{u^2} = \frac{1}{h(y)}\int u^2(y,z)dz$, $z =$ depth at $h(y)$. Equation (3) becomes

$$f(y) = \frac{2h(y)}{R(y)}\frac{u_*^2(y)}{\overline{u^2(y)}} \tag{4}$$

Further derivation leads to the mean cross-sectional value of f

$$f = M\frac{u_*^2}{\overline{\overline{V^2}}} \tag{5}$$

where

$$M = 2P/B. \tag{6}$$

Combine Eqs (2) with (5) and (6) yields $D = (P/B)R$, which depend on geometrical parameters only.

4.2 Apparent shear — apparent friction factor

Another factor, which has long been known to contribute to the uncertainty in the value of the resistance factor, is the interaction existing between the main channel and floodplain during overbank flow. To solve this problem, an increase in resistance factor is assumed due to a strong and invisible turbulent shear at the interface region, known as apparent shear, which can be quantified using an apparent friction factor, f_a.

Assuming uniform flow in a simple channel section with trapezoidal shape,

$$\text{Driving force} = \text{Resisting force}$$
$$W\sin\theta = \tau_0 P\Delta x$$

but $\tau_0 = C_f\rho\frac{V^2}{2}$, $C_f = \frac{f}{4}$, $W = \rho gA$, and $\sin\theta = S_0$. For a unit distance,

$$fPV^2 = 8gAS_0$$

In the case of overbank flow with vertical interface, an additional resisting force due to apparent shear must be added. Then, considering the balance of forces along the flow direction in the main channel leads to:

$$f_{am} = \frac{8gA_mS_0 - f_{mp}PV_m^2}{2y(\Delta V^2)} \tag{7}$$

Similarly, for the floodplain region,

$$f_{af} = \frac{f_{fp}PV_f^2 - 8gA_fS_0}{y(\Delta V^2)} \tag{8}$$

For Eqs (7) and (8) above, the flow velocities V_m and V_f are from measured values, the channel slopes S_0, the interface wetted perimeter y, the sub-sectional area A and the sub-sectional wetted perimeter P can be obtained easily from geometrical measurement. The boundary friction factors, f_{mp} and f_{fp} can be obtained by extrapolation using inbank data, assuming that no interaction existed between the main channel and floodplain. Since the measured velocities are strongly influenced by the floodplain and main channel interaction, the velocity differences (ΔV) between subsections were obtained, not from the measured values, but from estimated velocities for each sub-sections using traditional methods.

4.3 Regression Analysis for f_a

The values of f_{am} and f_{af} calculated using Eqs (7) and (8) above should in principle have the same values ($= f_a$) but acting in opposite directions, and depend on the geometrical cross section (B, b), flow depth $(H, H-h)$, geometrical relationships (M_f, M_m, R_f, R_m), boundary roughness (f_{fp}, f_{mp}), and velocity difference between the main channel and floodplain $(V_{mp} - V_{fp})$. In dimensionless form:

$$f_a, \propto \left[\frac{B}{b}, \frac{H-h}{H}, \frac{M_f}{M_m}, \frac{R_f}{R_m}, \frac{f_{fp}}{f_{mp}}, \frac{V_{mp}-V_{fp}}{V_{mp}}\right] \tag{9}$$

This relationship can be sorted using a multiple non-linear regression analysis approach to give the relationships on which f_a, depends. This f_a, depends. This approach is preferred as it allows the predictive sense, i.e. without the need for measuring the velocities of the subsections.

5 Relationships between Hydraulic Diameter, D and Hydraulic Radius, R

The values of P/B calculated for the rivers are shown in Figure 7. It demonstrated that the values of P/B for a river are almost constant, but vary from river to river. Generally, the value of P/B is found ranges from 1.01–1.23, which means that the hydraulic diameter, D is equal to 1.01–1.23R for the selected rivers.

6 Flow resistance results

By substituting $D = (P/B)R$ into Eq. (2), the resistance to flow for the main channel region of the investigated rivers has been calculated in term of the Darcy-Weisbach friction factor, f_m as shown in Figure 8. For inbank flow, i.e. $(\mathrm{H}-\mathrm{h})/\mathrm{H} < 0$, the f value for the selected equitorial rivers was found to be in the range of ±0.2 at low flow, and it decreases linearly with flow depths towards the bankfull level, due to the decrease in relative roughness in the main channel region. An exception to this is in River Senggi (B), which experienced a slight increment of f_m value due to the vegetation at the side banks. The respective values for River Main were found comparatively small with f_m values equal to ±0.04.

The overbank flow is characterized by an increased roughness value. As the surface properties in the main channels remained the same, such an increment can be considered due to the apparent shear mentioned earlier, which slows down the flow in main channel. For River Senggai and River Batu with obvious roughness differences between the main channel and floodplain, the increase in roughness starts when the flow is just overbank. For example, the f values for River Senggai increased from 0.157 at the bankfull level to 0.208 at $(\mathrm{H}-\mathrm{h})/\mathrm{H} = 0.082$, before they continue to reduce at higher depths. For River Senggi (B) and River Main, the increse only starts after a certain stage of overbank flow, i.e. $(\mathrm{H}-\mathrm{h})/\mathrm{H} = 0.166$ for River Senggi (B) and $(\mathrm{H}-\mathrm{h})/\mathrm{H} = 0.302$ for River Main due to the bank vegetation and cross-sectional geometry which prevent the interaction occurring before the effective bankfull level.

For the floodplain regions, the velocities collected from field measurements during overbank flow are always close to zero, except under very high overbank flow depth. As a result, the f_f values obtained are very high. Such values are known to be seriously affected by the "ponding effects" of the floodplain vegetation, and they are not suitable for use in representing the actual floodplain roughness.

6.1 *Apparent Friction Factor, f_a*

The apparent friction factor, f_a calculated using Eq. (7) is shown in Figure 9. These results show that a large apparent shear exists at the interface region especially when the flow is just overbank. In an extreme case, the value of apparent friction factor for River Senggi (B) is found equal to 41.37. For other rivers such as River Senggai, River Batu and River Main the maximum apparent friction factors found are 18.51, 15.34 and 9.08 respectively.

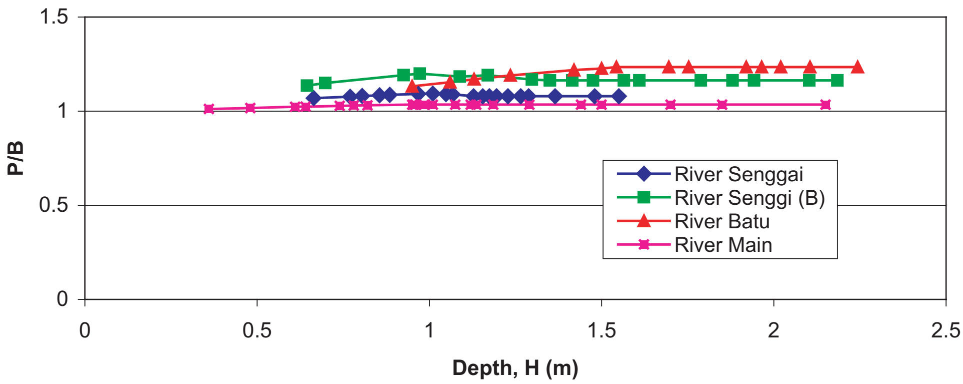

Figure 7 Values of P/B for the selected natural rivers.

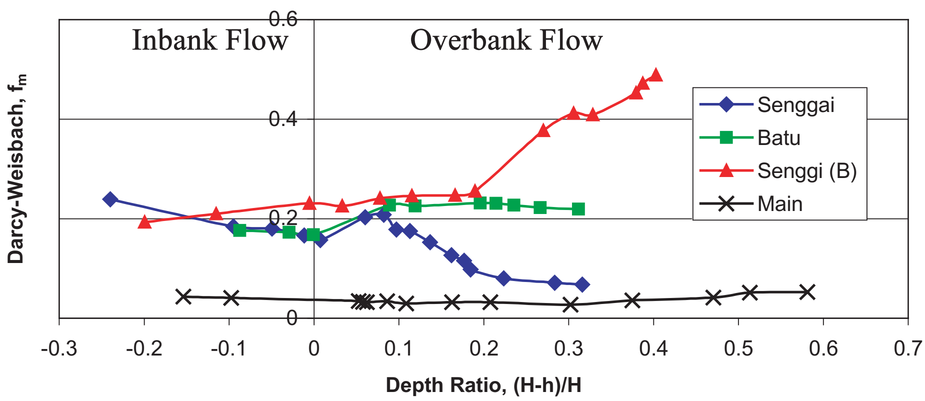

Figure 8 Variation of Darcy-Weisbach friction factor, f with depth of flow.

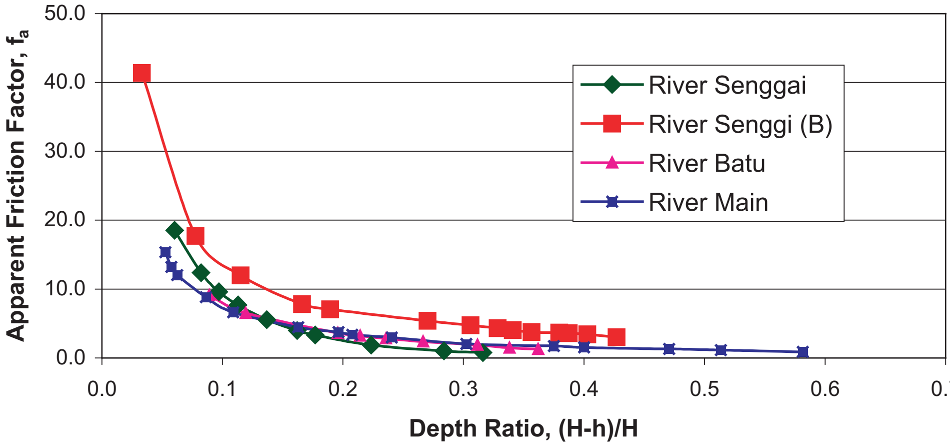

Figure 9 Variation of apparent shear with depth for overbank flow of natural rivers.

Table 3 Comparison of averaged boundary friction factor (f_m) and apparent friction factor (f_a) for overbank flow of natural rivers.

River Senggai				River Batu				River Senggi (B)				River Main			
(H−h)/H	f_a	f_a/f_m	ΔV	(H−h)/H	f_a	f_a/f_m	ΔV	(H−h)/H	f_a	f_a/f_m	ΔV	(H−h)/H	f_a	f_a/f_m	ΔV
0.060	18.509	140.8	0.297	0.089	9.076	56.2	0.476	0.033	41.370	177.6	0.269	0.053	15.341	444.9	1.167
0.082	12.366	102.2	0.319	0.119	6.530	41.1	0.489	0.0777	17.697	75.6	0.276	0.062	12.021	371.4	1.176
0.097	9.585	84.1	0.334	0.196	3.596	23.8	0.527	0.1153	11.966	50.9	0.279	0.109	6.631	207.0	1.262
0.113	7.682	72.1	0.350	0.214	3.200	21.4	0.537	0.1894	7.051	29.7	0.288	0.163	4.450	140.8	1.277
0.137	5.549	57.9	0.376	0.236	2.817	19.1	0.547	0.2701	5.391	22.5	0.300	0.240	2.955	95.7	1.306
0.177	3.310	42.1	0.429	0.266	2.368	16.3	0.565	0.3284	4.326	18.0	0.310	0.302	2.051	67.9	1.323
0.223	1.861	30.5	0.510	0.312	1.881	13.3	0.592	0.3795	3.648	15.1	0.321	0.375	1.730	59.2	1.342
0.284	1.007	23.2	0.643	0.338	1.445	10.3	0.608	0.4027	3.413	14.1	0.326	0.471	1.294	47.1	1.368
0.316	0.758	20.2	0.715	0.362	1.262	9.1	0.626	0.4271	2.993	12.3	0.332	0.581	0.869	35.4	1.434

When the apparent friction factor, f_a is compared with the averaged boundary friction factor, f_m as shown in Table 3, the maximum f_a/f_m ratios obtained for River Senggai, River Senggi (B), River Batu, and River Main are, 140.83, 177.58, 56.16, and 444.95, respectively. This implies that the apparent friction factor at the interface region is many times greater than the averaged boundary friction factor.

When the flow continues to rise, the value of f_a is found to decrease gradually with depth, while the velocity difference is increased with depth in all cases as shown in Table 3. This supports the finding that the apparent friction factor is inversely proportionate to ΔV^2 as reported by Christodolou and Myers [20].

The values of f_a calculated above have been tested against several independent variables such as B/b, $(H-h)/H$, M_f/M_m, R_f/R_m, f_{fp}/f_{mp}, and $(V_m - V_f)/V_m$ to develop an expression for f_a. This can be achieved using a multiple non-linear regression analysis approach. The relationship sought in the analysis is of the form

$$f_a = a\left(\frac{H-h}{H}\right)^{b1}\left(\frac{M_f}{M_m}\right)^{b2}\left(\frac{B}{b}\right)^{b3}\left(\frac{f_{fp}}{f_{mp}}\right)^{b4}\left(\frac{R_f}{R_m}\right)^{b5} \tag{10}$$

in which a, and $b1-b5$ are constants.

From the analysis, the correlations between f_a and each independent variable: $(H-h)/H$, R_f/R_m, B/b, f_{fp}/f_{mp}, M_f/M_m *are 0.952, 0.748, 0.270, 0.122, and 0.071*, respectively. This indicates the value of f_a strongly depends on the depth ratio, and moderately strong depends on the hydraulic radius ratio between flood plain and main channel. Whereas the rather weaker correlations between f_a and B/b, f_{fp}/f_{mp}, M_f/M_m would indicate that these variables will have lesser degree of influence to the values of f_a,

However, when the correlation is carried out with a combination of 2, 3, 4 and 5 variables, better results are obtained. This shows that all the variables are significant in the determination of an accurate value of f_a. Therefore, all the independent variables are retained, and the final regression equation obtained is:

$$f_a = 0.82\left(\frac{H-h}{H}\right)^{-2}\left(\frac{M_f}{M_m}\right)^{-11.5}\left(\frac{B}{b}\right)^{0.55}\left(\frac{f_{fp}}{f_{mp}}\right)^{-0.85} \times\left(\frac{R_f}{R_m}\right)^{0.31} + 0.1255. \tag{11}$$

The coefficient of multiple correlations is found to be 0.995. The determination coefficient of 0.991 indicates that the preceding equation would explain 99.1% of the total deviation in f_a.

The values of f_a estimated by Eq. (11) are compared with the observed values in Figure 10. The close agreement of the data, from flooded natural rivers, and over a range of geometrical conditions, is encouraging.

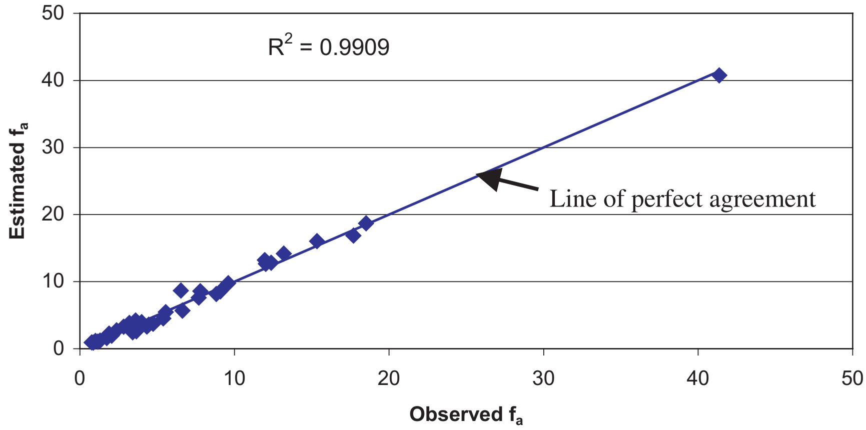

Figure 10 Apparent friction factor, f_a observed and predicted using Eq. (11).

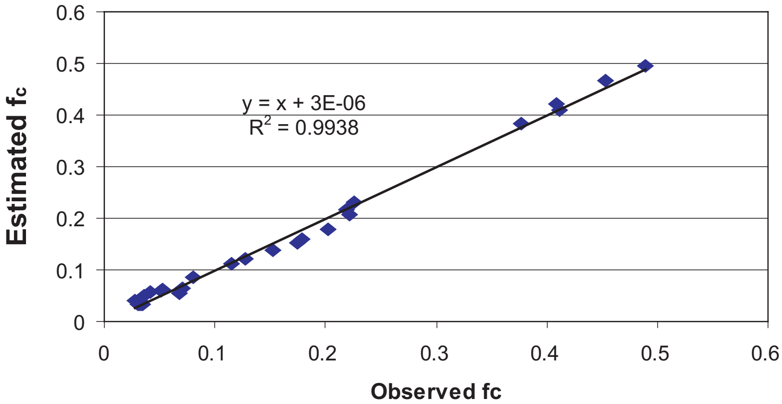

Figure 11 Observed and estimated composite friction factor using Eq. (12).

6.2 *Composite friction factor*

Based on the estimated boundary shear stress, f_{mp}, and the apparent friction factor, f_a, the weighted ratio on how the composite (actual) friction factor, in the main channel, f_c depends on f_{mp} and f_a has been found in terms of the associated wetted perimeters P_m and y, as:

$$f_c = 1.03\left[\left[0.117\left(\frac{P_m}{P_t}\right)^{-7.06} f_{mp} + 0.507\left(\frac{2y}{P_t}\right)^{1.08} f_a\right] - 0.008\right] \quad (12)$$

in which P_t is the total wetted perimeter.

The results obtained using Eq. (12) are compared with the observed main channel friction factor in Figure 11. It can be seen that that most of the main channel friction factors can be calculated accurately, with a coefficient of determination of 0.994. However, as the data are based on only four natural rivers, only limited reliance can be placed on this. Additional data is needed before more reliable relationships for calculating the apparent friction factor and composite friction factor can be found. Nevertheless, the writers consider that this analysis has helped to pinpoint the important causative factors in determining the friction factors, and at best provide a simple means of determining it from easily calculated parameters.

7 Conclusions

Based on extensive data collected from four frequently flooded natural rivers and results obtained, the following conclusions have been made.

i. The geometrical relationship, $D = (P/B)R$ enables the calculation of Darcy-Weisbach friction factor, f for any complicated river cross-section. From the analysis carried out, it is found that the hydraulic diameter, D for the selected equatorial natural rivers is equal to $1.01-1.23R$.
ii. A strong apparent shear exists at the interface region of main channel and flood plain, the apparent shear has been quantified in the form of an apparent friction factor f_a. It is found that the apparent friction factor is many times greater than the averaged friction factor, i.e. for River Senggi (B), $f_a/f_m = 177.58$ at depth $(H-h)/H = 0.033$. The apparent friction factor is maximized when the flow is just overbank but reduced at higher depths.
iii. A high correlation between the apparent friction factor and depth ratio has been obtained through regression analysis, and this indicates that f_a strongly depends on $(H-h)/H$. However, for an accurate estimation of f_a, all the influencing factors: $(H-h)/H$, R_f/R_m, B/b, f_{fp}/f_{mp}, M_f/M_m have to be included. The results obtained using Eq. (11) show that that values of f_a can be estimated accurately from easily calculated parameters, with a correlation of $r = 0.995$

when compared to the values of f_a calculated from river measurements.

iv. A methodology has been proposed to estimate the composite (actual) friction factor for flooded natural rivers, in terms of f_{mp} and f_a. From the results obtained using Eq. (12), the value of f_c found can be estimated correctly when compared to the actual friction factor, f_m in the main channel, with a correlation of $r = 0.997$.

v. A statistical method is able to provide a simple means of determining the apparent friction factor and composite friction factor for overbank flow of natural rivers. However, more data are needed to further improve and generalize the equations proposed.

Notation

τ_0 = Mean shear stress on the bottom
Δp = Pressure drop along the length, l
Δx = Flowing distance in streamwise direction
ΔV = Velocity difference between the main channel and flood plain
A = Wetted area
A_m, A_f = Wetted areas for the main channel and flood plain
b = Bankfull width
B = Top width
C_f = Resistance coefficient
D = Hydraulic diameter
$DDCM$ = Diagonal Divided Channel Method
f = Darcy-Weisbach friction factor
f_a = Apparent friction factor
f_c = Composite friction factor
f_{am}, f_{af} = Apparent friction factors for the main channel and flood plain
f_f, f_m = Observed Darcy-Weisbach friction factors for the main channel and flood plain
f_{fp}, f_{mp} = Estimated Darcy-Weisbach friction factors for the main channel and flood plain
h_0 = A characteristic dimension of the flow
h = Bankfull depth
H = Depth of flow
$h(y)$ = Depth of flow at a distance y from the bank
$HDCM$ = Horizontal Divided Channel Method
M = A geometrical parameter
M_f, M_m = Geometrical parameters, M for flood plain and main channel
n = Manning's coefficient
P = Cross sectional wetted perimeter
R = Hydraulic radius
S, S_0 = Longitudinal bed slopes
SCM = Single Channel Method
$\bar{u}$ = Averaged velocity on the vertical line h(y)
u_*^2 = Shear velocity
u = Local time averaged velocity of the flow
$\overline{V}$ = Sectional averaged velocity
V = Mean velocity
V_m, V_f = Mean velocities for the main channel and flood plain
V_{mp}, V_{fp} = Estimated mean velocities for the main channel and flood plain
$VDCM$ = Vertical Divided Channel Method
W = Gravitational weight of fluid
y = Interfacr wetted perimeter

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