Discharge estimation in compound channels with fixed and mobile bed

GALIP SECKIN¹, MUSTAFA MAMAK¹, SERTER ATABAY² and MAZEN OMRAN³

¹School of Civil Engineering, University of Cukurova, Adana, Turkey

²Civil Engineering Department, American University of Sharjah, PO Box 26666, Sharjah, United Arab Emirates

³Arup, The Arup Campus, Blythe Gate, Blythe Valley Park, Solihull, West Midlands, B90 8AE, UK

e-mail: gseckin@cu.edu.tr; mmamak@cu.edu.tr; satabay@aus.edu; mazen.omran@arup.com

MS received 29 August 2008; revised 10 August 2009

Abstract. Two-dimensional (2-D) formulae for estimating discharge capacity of straight compound channels are reviewed and applied to overbank flows in straight fixed and mobile bed compound channels. The predictive capabilities of these formulae were evaluated using experimental data obtained from the small-scale University of Birmingham channel. Full details of these data and key references may be found at the following www.flowdata.bham.ac.uk (university website). 2-D formulae generally account for bed shear, lateral shear, and secondary flow effects via 3 coefficients f , λ and Γ . In this paper, the secondary flow term (Γ) used within the 2-D methods analysed here is ignored in all applications. Two different 2-D formulae almost give practically the same results for the same data when the secondary flow term is ignored. For overall test cases, the value of dimensionless eddy viscosity λ used in 2-D formulae was kept at 0.13 as recommended for open channels. 2-D formulae gave good predictions for most of the data sets studied in comparison with the traditional 1-D methods, namely the Single Channel Method (SCM) and the Divided Channel Method (DCM). The accuracy of predictions of 2-D formulae was increased by calibrating of λ value where the calibration was needed. For overall data, the average errors for each method were Lateral Division Methods (LDMs), with λ value of 0.13, **2.8%**, DCM **14.3%** and SCM **–26.8%**. The average error was **0.5%** for LDMs with the calibrated values of λ .

Keywords. Flood and floodworks; river engineering; mathematical modelling.

1. Introduction

Compound channels which consist of generally a main river channel and its floodplain are very important for environmental, ecological, and design issues. Therefore, it is essential

to understand the flow mechanism of rivers in both their inbank and overbank conditions. In a flood event the discharge for a particular river may increase so rapidly that the bankfull condition is breached and the flow passes over onto the floodplain. The structure of the flow then becomes more complex by the momentum transfer between the floodplain and the main channel due to the significant dissimilar velocity distributions in these sub-areas. In this case, the prediction of discharge is more difficult than that when the river is flowing just inbank.

The flow mechanisms in straight compound channels are now well-understood (Knight 1999). In the past two decades, many methods for computing overbank flow have been developed based on either one-dimensional (1-D), or two (2-D) and three-dimensional (3-D) hydrodynamic approaches.

It is well-known that the single channel method (SCM) underestimates the discharge capacity for compound channels. Most divided channel methods (DCM) overestimate the discharge capacity. Nevertheless, SCM and DCM are still widely used in engineering practice, due to their simplicity in use, and can give satisfactory results under certain conditions. See Wright & Carstens (1970), Wormleaton *et al* (1982), Prinós & Townsend (1984), Wormleaton & Hadjipanós (1985), Myers (1978), Knight & Hamed (1984), Myers *et al* (2001), Cassells *et al* (2001), Seckin (2004) and Atabay (2006) for a comparison of the accuracy of such methods.

Early work by Myers & Elsayy (1975), Myers (1978), Wormleaton, *et al* (1982), Knight & Demetriou (1983), Knight & Hamed (1984) indicated the importance of taking into account the main channel/floodplain interaction effects which were first recognized and investigated by Sellin (1964) and Zheleznyakov (1971). Ackers (1993) and Bousmar & Zech (1999) developed 1-D methods; Coherence method (COHM), 1-D Exchange Discharge method (EDM) respectively. Shiono & Knight (1989), Wark *et al* (1990), Lambert & Sellin (1996), Irvine *et al* (2000), and Prooijen *et al* (2005) developed 2-D methods; Shiono & Knight method (SKM), 2-D Lateral Division methods (LDMs), respectively. All these methods take into account momentum transfer due to lateral shear and vorticity at the main channel/floodplain interface.

Seckin (2004) applied four 1-D methods, namely SCM, DCM, COHM and EDM to a major coverage of both experimental and field data obtained from the large-scale UK Flood Channel Facility, small-scale University of Birmingham channel, and a prototype compound river channel (Main River). These data include smooth or rough surfaces for the floodplain proportions, and rigid or mobile surfaces for the main channel section of a compound channel. Seckin (2004) concluded that both EDM and COHM gave better predictions than the SCM and DCM.

It is well-known that three-dimensional (3-D) models require more information and turbulence coefficients and are at present not immediately useful for design purposes due to the calibration requirements. Current study has therefore focused on investigation of the performance of 2-D methods using data from the small-scale University of Birmingham channel. The general validity of 2-D methods has been extended by testing those methods against data sets other than those used in their original formulation.

2. Theoretical background for the methods

2.1 Traditional 1-D methods

The traditional methods for predicting the discharge conveyed by a compound channel are based on one of the well-known flow formulae, such as the Manning, Chezy or

Darcy–Weisbach equations, as shown below:

$$n = R^{2/3} S_0^{1/2} / U_0 C = U_0 / (R S_0)^{1/2} f = 8g R S_0 / U_0^2, \quad (1)$$

where n , C , and f are overall resistance coefficients, U_0 is the section mean velocity, R is the hydraulic radius ($= A/P$ in which A is flow area and P is wetted perimeter), S_0 is bed slope and g is gravitational acceleration.

When predicting the discharge in a compound channel using the Single Channel Method (SCM), the whole compound channel section is treated as a single section and the average velocity can be used to predict the discharge as shown in (2):

$$Q = (AR^{2/3} S_0^{1/2}) / n = K S_0^{1/2}, \quad (2)$$

where K is the section conveyance.

When predicting the discharge in a compound channel using the Divided Channel Method (DCM), zonal resistance coefficients have to be calculated. In order to estimate the zonal resistance coefficients, the cross-section of the compound channel is divided into a number of individual sub-sections or zones. The cross-sectional area velocity given in (1) may then be replaced by the sub-section velocity.

The local friction, f_b , is usually defined in a similar way to the global friction factor, except that the local boundary shear stress, τ_b , and the depth-averaged velocity, U_d , are required instead of the τ_o and U_o . These ‘global’, ‘zonal’ and ‘local’ resistance coefficients were determined from the following equation. See Knight & Shiono (1996), and Atabay & Knight (1999) for further details.

$$\begin{array}{lll} \tau_o = \left(\frac{f_o}{8} \right) \rho U_o^2; & \tau_z = \left(\frac{f_z}{8} \right) \rho U_z^2; & \tau_b = \left(\frac{f_b}{8} \right) \rho U_b^2 \\ \text{(global)} & \text{(zonal)} & \text{(local)} \end{array}$$

2.2 2-D Lateral division methods

There are several 2-D hydrodynamic methods that have been developed by Shiono & Knight (1989, 1991), Wark *et al* (1990), Lambert & Sellin (1996), Ervine *et al* (2000), and Prooijen *et al* (2005). Large-scale Flood Channel Facilities data (FCF), UK have been commonly used to prove the validity of all 2-D methods mentioned above. Here, Shiono & Knight method (1989, 1991) and Ervine *et al* method (2000) are chosen to investigate the performance of 2-D methods using the data obtained from the small-scale University of Birmingham channel. These 2-D methods are summarized here.

Shiono & Knight (1989) presented an analytical solution to the Navier–Stokes equation to predict the lateral variation of depth-averaged velocity in compound channels. The Navier–Stokes equation may be written in the following form for a fluid element in steady uniform flow in which there are both bed generated shear and lateral shear

$$\rho \left[v \frac{\partial u}{\partial y} + w \frac{\partial w}{\partial z} \right] = \rho g \sin \theta + \frac{\partial \tau_{yx}}{\partial y} + \frac{\partial \tau_{zx}}{\partial z}, \quad (3)$$

(i.e. Secondary flows = weight force + Reynolds stresses(lateral + vertical), where u , v and w are the local velocities in the x (streamwise), y (lateral) and z (vertical) directions respectively; $S_0 = \sin \theta$, is the bed slope; τ_{yx} and τ_{zx} are the Reynolds stresses on planes

perpendicular to the y and z directions respectively; ρ is the water density; and g is the gravitational acceleration.

Shiono & Knight (1989) obtained the depth-averaged velocity equation by integrating (3) over the water depth H based on the eddy viscosity approach and, by ignoring the secondary flow contribution, arrived at

$$\rho g H S_0 - \frac{1}{8} \rho f U_d^2 \left(1 + \frac{1}{s^2}\right)^{1/2} + \frac{\partial}{\partial y} \left\{ \rho \lambda H^2 \left(\frac{f}{8}\right)^{1/2} U_d \frac{\partial U_d}{\partial y} \right\} = 0, \quad (4)$$

where U_d is the depth-averaged mean velocity; λ is the dimensionless eddy viscosity; f is the Darcy–Weisbach friction factor and s is the main channel lateral side slope.

Shiono & Knight (1989) solved the (4) analytically and obtained the following equation for the case of $H = \text{constant}$ in the form

$$U_d = \left[A_1 e^{\gamma y} + A_2 e^{-\gamma y} + \frac{8gS_0H}{f} \right]^{1/2} \quad (5)$$

and for linearly varying depth as

$$U_d = [A_3 Y^{\alpha_1} + A_4 Y^{-\alpha_2} + \omega Y]^{1/2}, \quad (6)$$

where A_1 , A_2 , A_3 and A_4 are unknown constants; and γ , α_1 , α_2 and ω are the ancillary terms of (5) and (6) and are given elsewhere by Shiono & Knight (1989, 1991).

Equation (4) is only valid when secondary flows are not considered. However, secondary flows are important in many cases. In such a case, the right-hand side of (4) is not zero (Shiono & Knight 1989, 1991) and then (4) will be;

$$\begin{aligned} \rho g H S_0 - \frac{1}{8} \rho f U_d^2 \left(1 + \frac{1}{s^2}\right)^{1/2} + \frac{\partial}{\partial y} \left\{ \rho \lambda H^2 \left(\frac{f}{8}\right)^{1/2} U_d \frac{\partial U_d}{\partial y} \right\} \\ = \frac{\partial}{\partial y} [H(\rho U V)_d] = \Gamma. \end{aligned} \quad (7)$$

Shiono & Knight (1991) used the approximation in the right hand side of (7) to solve it analytically.

The Shiono & Knight method (SKM) was originally developed for straight and nearly straight channels. Attempts have been undertaken to use the SKM in modelling non-prismatic and meandering channels (Omran 2005).

The method of Ervine *et al* (2000) is also similar to the Shiono & Knight (1991) method and can be applied to both straight and meandering channels. Ervine *et al* (2000) solved the Navier–Stokes equation (3) analytically in a similar approach used by Shiono & Knight (1991) and proposed the following formula, by adding the secondary flow contribution, for computing the lateral distribution of depth-averaged velocity;

$$U_d = \sqrt{\left(\frac{8}{\rho f} \left\{ \frac{b'}{M+N} \Phi + C_1 \Phi^{\left(\frac{L+M-\sqrt{L^2+2LM+M^2+4LN}}{2L} \right)} + C_2 \Phi^{\left(\frac{L+M+\sqrt{L^2+2LM+M^2+4LN}}{2L} \right)} \right\} \right)}, \quad (8)$$

where C_1 and C_2 are unknown constants; b' , L , M , N and Φ are the ancillary equations of (8) and are given elsewhere by Ervine *et al* (2000).

For the case of no secondary currents, K which is included within M and N in (8) is equal to zero.

Figure 1. Cross-section of flume at Hydraulic Laboratory of Birmingham University.

3. Experimental work

Twelve series of experiments were carried out on the flume of Birmingham University were considered for analysis in this paper. The flume was a non-tilting 22 m long with a test length of 18 m. The flume was 1213 mm wide, comprising a 398 mm wide, 50 mm deep main channel and two rigid 407.3 mm wide floodplains, as shown in figure 1. The bed slope of the flume was set to 2.024×10^{-3} . The flume had three different water circulation systems: two internal ones, which re-circulated water from the downstream end, and one external one, which passed water through the flume to the main laboratory sump. The flow was supplied by 50 mm, 100 mm, and 150 mm diameter pipelines, and the various discharges were measured by an electro-magnetic flow meter, a venturimeter and a dall tube, respectively. For a given test discharge, the tailgate at the downstream end of the flume was adjusted to produce uniform flow conditions throughout the 18 m test length. Water surface profiles were measured directly using pointer gauges.

For the smooth main channel and smooth floodplain experiments, both the main channel bed and floodplains were covered with PVC materials (Atabay & Knight 1999; Atabay 2001; Seckin 2004; Atabay & Knight 2006).

For either smooth or roughened main channel, and roughened floodplain experiments, A-frames of aluminum wire grids, as shown in figure 2, were placed along the channel at different interval spacings (i.e. $L_m = 3$ m, 2 m, 1 m, 0.5 m, and 0.25 m) to create rough surfaces on the main channel and floodplains (Atabay 2001; Seckin 2004; Seckin & Atabay 2005).

For the mobile bed and smooth floodplain experiments, a uniform sand size of $d_{35} = 0.80$ mm was used. (Ayyoubzadeh 1997; Knight *et al* 1999; Atabay 2001; Atabay & Seckin 2000; Atabay, Knight *et al* 2004, 2005).

For the mobile bed and roughened floodplain experiments, A-frames of aluminum wire grids were placed along the channel on the floodplains at different interval spacings (i.e. $L_m = 3$ m, 1 m, 0.5 m, and 0.25 m) (Tang & Knight 2001, 2006).

Figure 2. Schematic of metal mesh for the roughness on the floodplain.

Table 1. Summary of the cross-sectional and roughness parameters for each test case.

Test case	Cross section	Main channel boundary	Floodplain boundary	Flow description
Fixed boundary tests				
F1	asymmetrical	smooth	smooth	overbank
F2	symmetrical	smooth	smooth	overbank
F3	symmetrical	smooth	rough ($\lambda_d = 1$ m)	overbank
F4	symmetrical	smooth	rough ($\lambda_d = 0.5$ m)	overbank
F5	symmetrical	rough ($\lambda_d = 2$ m)	rough ($\lambda_d = 0.5$ m)	overbank
F6	symmetrical	rough ($\lambda_d = 3$ m)	rough ($\lambda_d = 0.25$ m)	overbank
Mobile boundary tests				
M1	asymmetrical	mobile	smooth	overbank
M2	symmetrical	mobile	smooth	overbank
M3	symmetrical	mobile	rough ($\lambda_d = 3$ m)	overbank
M4	symmetrical	mobile	rough ($\lambda_d = 1$ m)	overbank
M5	symmetrical	mobile	rough ($\lambda_d = 0.5$ m)	overbank
M6	symmetrical	mobile	rough ($\lambda_d = 0.25$ m)	overbank

All the above experimental arrangements are given in table 1 in which *F* and *M* denotes fixed and mobile bed experiments, respectively. Full details of these data and key references can be found at www.flowdata.bham.ac.uk.

3.1 Stage-discharge curves

Stage-discharge data will be demonstrated graphically in the next sections of this paper and they are also formulated in table 2. Full discussion of these stage-discharge relationships are given in many papers referenced earlier.

Table 2. Stage-discharge relationships for each test case.

Test No	Discharge range (m ³ /s)	Depth range (m)	Equation	R^2
Fixed boundary cases				
F1	0.015–0.050	0.061–0.106	$H = 0.4074 Q^{0.4489}$	0.9969
F2	0.015–0.055	0.060–0.095	$H = 0.267 Q^{0.3672}$	0.9973
F3	0.015–0.035	0.062–0.104	$H = 0.8363 Q^{0.6197}$	0.9985
F4	0.015–0.050	0.065–0.168	$H = 1.7909 Q^{0.7995}$	0.9921
F5	0.010–0.035	0.071–0.163	$H = 1.58 Q^{0.6829}$	0.9929
F6	0.011–0.027	0.071–0.141	$H = 2.2867 Q^{0.7698}$	0.9971
Mobile boundary cases				
M1	0.010–0.027	0.059–0.099	$H = 0.6423 Q^{0.5228}$	0.9982
M2	0.012–0.045	0.058–0.101	$H = 0.3617 Q^{0.4092}$	0.9985
M3	0.012–0.028	0.060–0.096	$H = 0.5781 Q^{0.5072}$	0.9942
M4	0.012–0.028	0.061–0.110	$H = 1.1665 Q^{0.6634}$	0.9972
M5	0.012–0.028	0.062–0.121	$H = 1.8011 Q^{0.7601}$	0.9960
M6	0.012–0.028	0.063–0.126	$H = 2.2496 Q^{0.8022}$	0.9963

3.2 Flow resistance in the main channel and on the floodplains

In order to estimate the discharge capacity of a compound channel, its cross-section is divided into a number of zones or panels. In this case, particular care needs to be taken over the definition and use of resistance coefficients, as highlighted by Knight (2001). The flow resistance of the main channel and the floodplain proportions of the experimental compound channels were determined from a separate series of experiments or previously published results, as highlighted out by Cassells *et al* (2001). Herein, for each series of test cases, the main channel and floodplain zonal resistance coefficient, n_{mc} and n_{fp} , respectively, is derived from fully low ($H \leq 0.05$ m) and high ($H \geq 0.05$) inbank flow measurements. For the high inbank measurements the main channel of Birmingham University flume was isolated at the bankfull level ($h = 0.05$ m) using the adjustable side walls on the floodplains (see Ayyoubzadeh 1997; Atabay 2001).

For the smooth main channel and smooth floodplain experiments, both n_{mc} and n_{fp} are equal to 0.0091 measured at bankfull level ($h = 0.05$ m) as given by Atabay (2001), Atabay & Knight (2006).

The mobile main channel Manning's n value was taken as 0.015 by Seckin (2004), its measured mean value from inbank flow experiments (see Ayyoubzadeh 1997). Seckin (2004) adopted this constant Manning's n value within EDM and COHM applied to University of Birmingham channel data. The results showed that both EDM and COHM gave large errors up to 19% for mobile bed experiments.

Atabay & Knight (2006) highlighted that for mobile beds the alluvial resistance is flow (or depth) dependent. Atabay & Knight (2006) made three different assumptions concerning the mobile main channel Manning's n :

- (i) Single Manning's n value calculated at the bankfull level,
- (ii) Variable Manning's n values calculated for wholly inbank flow data ($H \leq 0.05$ m), and
- (iii) Variable Manning's n values calculated from zonal velocity data ($H > 0.05$ m).

Atabay & Knight (2006) adopted these three assumptions within COHM to see which might be the most appropriate for simulating the University of Birmingham flume data and concluded that COHM may be used in mobile bed channels, with n varying with H .

Based on the findings of Atabay & Knight (2006), in this current study, the (9) was adopted to model for the mobile boundary cases instead of constant value of Manning's n to be analysed here:

$$n_{mc} = 1.0862H^2 - 0.1216H + 0.0176(R^2 = 0.7868). \quad (9)$$

Equation (9) was derived from the work of Ayyoubzadeh (1997) for wholly inbank flow data.

In order to investigate the roughness characteristics of the aluminum wire grid A-frames mentioned before, Tang (2002) carried out a series of experiments for fully rough inbank flows, and developed a technique that allows the zonal flow resistance for the aluminum wire grid A-frames plus one side wall to be predicted. As a result, Tang (2002) suggested a series of equations giving Manning's n -total depth (H) relationships for each space of L_m in order to estimate zonal Manning's n coefficients which can be used for overbank flow:

for $L_m = 3$ m :

$$n = -107.21H^4 + 31.535H^3 - 3.8985H^2 + 0.3056H + 0.0098 \quad (10)$$

for $L_m = 1$ m :

$$n = -80.338H^4 + 36.646H^3 - 6.4343H^2 + 0.6083H + 0.0101 \quad (11)$$

for $L_m = 0.5$ m :

$$n = -62.658H^4 + 36.428H^3 - 7.9726H^2 + 0.9017H + 0.0103 \quad (12)$$

for $L_m = 0.25$ m :

$$n = -534.79H^4 + 190.73H^3 - 25.253H^2 + 1.7857H + 0.0099 \quad (13)$$

The determination coefficient (R^2) of equations (10)–(13) is equal to 0.9999, 0.9995, 0.9992, and 0.9997, respectively.

The above equations (10)–(13) were applied to the measured overbank flow depths to be analysed herein, and their average Manning's n values were determined, as shown in table 3. In table 3, the values of n_{fp} for both fixed or mobile series of experiments were obtained from the (10)–(13). The values of n_{mc} for roughened main channel cases, tests F5 and F6, were obtained from the measurements at the bankfull level ($h = 0.05$) as 0.021 and 0.018 for $L_m = 2$ m and $L_m = 3$ m, respectively.

4. Results and discussion

After determining the hydraulic resistance of the compound channel sections, the stage-discharge relationship can be estimated and compared with the measured values. In this current work, four different methods, namely, SCM, DCM, Shiono & Knight (1989, 1991), and the method of Ervine *et al* (2000), were applied to different sets of small-scale data.

As explained before, both the methods Shiono & Knight (SKM) and Ervine *et al* include secondary flow term. In this study the secondary flow term was ignored for all applications. In this case, 2-D methods required only two parameter, hydraulic resistance coefficient, f ,

Table 3. Manning's n roughness coefficients used for each method.

Test No.	Low and High inbank measurements	
	n_{mc}	n_{fp}
F1	0.0091	0.0091
F2	0.0091	0.0091
F3	0.0091	0.033
F4	0.0091	0.052
F5	0.021	0.053
F6	0.018	0.075
M1	Eq. [9]	0.0091
M2	Eq. [9]	0.0091
M3	Eq. [9]	0.021
M4	Eq. [9]	0.034
M5	Eq. [9]	0.049
M6	Eq. [9]	0.072

and dimensionless eddy viscosity, λ . Eddy viscosity is a coefficient relating the average shear stress within a turbulent flow of water to the vertical gradient of velocity. It actually represents the molecular viscosity and the effects of turbulence from the Reynold's stress terms. Eddy viscosity depends on the momentum of fluid, gradients of the velocity and the scale of flow phenomenon. Knight (1999) pointed out that typical default values of dimensionless eddy viscosity λ used within 2-D methods are in the range of 0.07–0.50, with standard values being 0.067 (boundary layers), 0.13 (open channels), 0.16 (trapezoidal data), 0.27 (FCF, smooth floodplains), and 0.22 (FCF, rough floodplains). Here, λ was set to 0.13 for all applications.

The Γ term in the SKM represents the secondary current flows. Omran (2005) and Knight *et al* (2007) showed that this term has an important role when the focus is on the boundary shear stress distribution across the channel on the zonal discharge. The aim of this paper was to study the overall discharge of the channel, hence the Γ term was not considered and λ is used as a 'catch all' parameter to represent both lateral shear and secondary flow. This facilitates the modelling approach since default values of, λ are adopted and as a result the user has less parameter to deal with.

Ignoring the secondary flow term, the authors applied these three methods to both asymmetric and symmetric compound channel data sets; tests F1 and F2, respectively. The results are shown in figures 3 and 4. As seen in these figures, Shiono & Knight method (SKM) and the method of Ervine *et al* fit each other for each node. Therefore, these two different 2-D methods will be named as LDMs in all figures and tables presented herein as the secondary current term was ignored.

In this paper, the error between the calculated and measured discharge was determined using the following equation:

$$Error(\%) = \frac{Q_c - Q_m}{Q_m} \times 100, \quad (14)$$

where Q_c is the calculated discharge and Q_m is the measured discharge.

Figure 3. Comparison between analytical and experimental lateral distributions of depth-averaged velocity for test F1 ($H = 0.0908$ m, $l = 0.13$, $nfp = nmc = 0.0091$).

Figure 4. Comparison between analytical and experimental lateral distributions of depth-averaged velocity for test F2 ($H = 0.0761\text{ m}$, $l = 0.13$, $nfp = nmc = 0.0091$).

4.1 Results for smooth main channel and smooth floodplains (tests F1 and F2)

The stage-discharge results for tests F1 and F2 are shown in figures 5 and 6. Both figures show that the LDMs lie close to the observed values. As expected, DCM overestimates all the measured discharge values for these data sets. It appears in figure 5 that the SCM is also accurate in predicting the discharge for asymmetrical shape of the flume, but its accuracy decreases for low depth ratios for symmetrical shape of the flume, as seen in figure 6.

Figure 5. Comparison of the measured data with the estimated stage-discharge curves for test F1.

Figure 6. Comparison of the measured data with the estimated stage-discharge curves for test F2.

4.2 Results for smooth main channel and rough floodplains (tests F3 and F4)

The results of the predicted and the measured stage-discharge relationship are shown in figures 7 and 8 for the smooth main channel and the different floodplain roughnesses. Although both figures indicate that LDMs predict the measured data well for λ value of 0.13 in

Figure 7. Comparison of the measured data with the estimated stage-discharge curves for test F3.

Figure 8. Comparison of the measured data with the estimated stage-discharge curves for test F4.

comparison with the SCM and DCM, the accuracy decreases with increasing depth ratios, especially for test F4. As seen in these figures, the λ values of 0.17 and 0.26, for tests F3 and F4 respectively, which increased the accuracy of the LDMs predictions.

Figure 9. Comparison of the measured data with the estimated stage-discharge curves for test F5.

Figure 10. Comparison of the measured data with the estimated stage-discharge curves for test F6.

4.3 Results for rough main channel and rough floodplains (tests F5 and F6)

It is well-known that a compound channel behaves like a single channel for $Dr > 0.5$. Figures 9 and 10 show that LDMs, for the λ value of 0.13, give good results up to $Dr = 0.5$, but deviate from the measured data after that value. It should be highlighted that although the λ value of 0.22 decreased the mean error from 9.3% to 1.7% for overall data of test F5, it increased the errors for the data lower than $Dr = 0.5$. Figure 9 also shows that SCM is closer in estimating measured data after $Dr = 0.5$. As seen in figure 10, the λ value of 0.26 increased the accuracy of LDMs predictions for test F6.

4.4 Results for mobile main channel and smooth floodplains (tests M1 and M2)

The compound channel configuration of a smooth floodplain adjacent to a mobile main channel represents a situation that is not common in practice, as noted by Cassells *et al* (2001). Tests M1 and M2 presented here represent this uncommon situation. In this case, estimation of discharge is very problematic, as highlighted by Cassells *et al* (2001). For Tests M1 and M2, use of the constant Manning's n value for the mobile bed caused large errors up to 20% in estimation discharge by EDM and COHM (Seckin 2004). Similar errors were also noticed by the Weighted Divided Channel Method (WDCM) developed by Lambert & Myers (1998) when applied to FCF and the University of Ulster data having a mobile bed and smooth floodplains (See Cassells *et al* 2001). Atabay & Knight (2006) showed that COHM gives more accurate results using Manning's n varying with depth (H) for the same data analysed in the current study. Therefore, it is essential to apply 2-D methods to the same University of Birmingham data to compare their performance with the 1-D methods.

As mentioned before, (9) was adopted within all methods analysed here for all mobile boundary cases. The results of the predicted and the measured stage-discharge relationship are shown in figures 11 and 12. As seen in these figures, surprisingly, DCM and SCM produced

Figure 11. Comparison of the measured data with the estimated stage-discharge curves for test M1.

slightly better predictions than that of the LDMs for high depth ratios. The accuracy of DCM may arise from compensating errors, as highlighted by Cassells *et al* (2001). The maximum error produced by LDMs with the λ value of 0.13 for test M1 was 19.0%. It decreased to 12.4% when a λ value of 0.3 was used. The λ value of 0.3 also decreased the

Figure 12. Comparison of the measured data with the estimated stage-discharge curves for test M2.

Figure 13. Comparison of the measured data with the estimated stage-discharge curves for test M3.

mean error from 9.05% to 1.6%. It should be noted that the accuracy of the LDMs would have been increased if different values of λ had been used for each depth ratio. For example, for the highest depth ratio ($Dr = 0.49$), a λ value of 1.0 gives excellent result (e.g. measured

Figure 14. Comparison of the measured data with the estimated stage-discharge curves for test M4.

Figure 15. Comparison of the measured data with the estimated stage-discharge curves for test M5.

$Q = 0.027\text{ m}^3/\text{s}$ and predicted $Q = 0.027\text{ m}^3/\text{s}$). However, Knight (1999) pointed out that typical default values of dimensionless eddy viscosity λ used within 2-D methods are in the range of 0.07–0.50.

Figure 16. Comparison of the measured data with the estimated stage-discharge curves for test M6.

Table 4. Prediction errors of each discharge assessment method for fixed beds.

TEST F1					
% error for each depth ratio					
Dr	DCM	SCM	LDMs	LDMs (calibrated)	
0.18	13.5	−5.2	5.3	no calibration	
0.26	14.4	0.7	5.7		
0.32	17.9	7.4	8.5		
0.34	11.5	2.8	2.5		
0.39	14.0	6.9	4.4		
0.40	7.3	1.2	−1.8		
0.45	10.3	5.7	0.5		
0.50	6.9	3.7	−3.1		
0.53	5.6	3.1	−4.7		
TEST F2					
% error for each depth ratio					
Dr	DCM	SCM	LDMs	LDMs (calibrated)	
0.16	10.5	−17.0	3.7	no calibration	
0.22	12.6	−9.2	5.9		
0.26	14.0	−3.6	7.4		
0.30	12.8	−2.1	6.4		
0.32	7.1	−5.6	1.2		
0.34	6.4	−4.7	0.6		
0.37	2.6	−6.8	−2.9		
0.41	3.6	−4.1	−1.8		
0.44	2.8	−3.9	−2.4		
0.46	3.0	−2.8	−2.1		
0.48	0.2	−5.0	−4.7		
TEST F3					
% error for each depth ratio					
Dr	DCM	SCM	LDMs	LDMs (calibrated)	
0.20	15.7	−64.3	−0.5	−4.3	
0.28	18.0	−57.8	2.3	−2.0	
0.34	22.2	−51.7	4.0	−0.7	
0.39	23.5	−48.1	3.9	−1.0	
0.44	28.0	−43.2	3.8	−1.3	
0.48	32.1	−39.0	5.4	0.0	
0.52	37.5	−33.7	6.9	1.2	
TEST F4					
% error for each depth ratio					
Dr	DCM	SCM	LDMs	LDMs (calibrated)	
0.23	25.6	−72.1	10.4	−2.5	
0.30	22.2	−69.4	43.7	−8.5	
0.38	25.8	−64.4	3.8	−10.4	
0.44	39.1	−57.3	10.8	−5.3	
0.53	53.6	−47.9	14.9	−3.2	
0.58	62.7	−42.4	16.8	−2.2	
0.63	82.0	−32.8	23.4	2.9	
0.67	90.9	−28.0	24.4	3.6	
0.70	106.5	−20.7	28.0	6.6	

Table 4. (Continued).

TEST F5				
% error for each depth ratio				
Dr	DCM	SCM	LDMs	LDMs (calibrated)
0.29	-1.7	-51.3	-5.1	-10.0
0.44	7.4	-37.1	0.5	-5.6
0.49	7.8	-34.0	-0.4	-6.8
0.55	17.3	-25.2	6.5	-0.9
0.59	24.7	-18.4	11.3	3.2
0.63	33.0	-11.4	16.8	8.0
0.65	36.8	-7.8	18.6	9.4
0.69	48.9	1.7	26.3	16.2
TEST F6				
% error for each depth ratio				
Dr	DCM	SCM	LDMs	LDMs (calibrated)
0.30	5.0	-65.5	-1.1	-9.5
0.45	17.3	-52.7	5.3	-5.6
0.51	26.0	-45.7	9.9	-2.5
0.56	32.9	-40.0	12.7	-1.0
0.61	41.4	-34.1	16.7	1.8
0.64	51.0	-27.9	21.0	5.0

The maximum and mean error produced by LDMs with the λ value of 0.13 for test M2 were 16.0% and 7.5%, respectively. For test M2, some attempts were also made to calibrate λ , but no significant improvement was achieved as discussed above.

4.5 Results for mobile main channel and rough floodplains (tests M3, M4, M5 and M6)

The effect of various densities of A-frames of aluminum wire grids on the floodplain is shown in figures 13, 14, 15 and 16 for tests M3, M4, M5 and M6, respectively. As seen in these figures, the accuracy of the LDMs using the λ value of 0.13 improves better with increasing densities of A-frames. Using the calibrated values of λ , 0.05 for test M3 and 0.1 for tests 14–16, respectively, LDMs give more accurate results. As seen in these figures, SCM almost underpredicted the measured data for these series in contrast to DCM.

Prediction errors of each method, namely DCM, SCM and LDMs, are shown in tables 4 and 5 for fixed and mobile boundary cases, respectively.

5. Conclusions

The following conclusions may be drawn from this study:

For fixed boundary cases;

- For smooth surfaces, LDMs, with the λ value of 0.13, gave slightly more accurate predictions than the SCM and DCM, and no calibration of λ was needed.
- For roughened surfaces, although LDMs, with the λ value of 0.13, predict the data sufficiently accurately for depth ratios lower than 0.5, there were excellent correlations between the measured values and predictions, when the λ value was calibrated. SCM

Table 5. Prediction errors of each discharge assessment method for mobile beds.

TEST M1					
% error for each depth ratio					
Dr	DCM	SCM	LDMs	LDMs (calibrated)	
0.15	0.0	-7.2	-1.3	-9.6	
0.22	4.4	1.2	3.8	-4.4	
0.30	6.0	5.1	6.3	-1.5	
0.36	6.8	6.8	8.1	0.7	
0.41	9.3	9.2	11.6	4.5	
0.46	12.1	11.4	15.8	8.9	
0.49	13.9	12.1	19.0	12.4	
TEST M2					
% error for each depth ratio					
Dr	DCM	SCM	LDMs	LDMs (calibrated)	
0.14	-8.2	-18.7	-8.0	no calibration	
0.20	-1.8	-8.1	-0.6		
0.23	-1.3	-5.5	0.6		
0.29	3.4	1.4	6.4		
0.34	7.8	7.1	12.1		
0.36	5.5	5.2	10.4		
0.39	3.3	3.2	8.8		
0.42	9.0	9.0	16.0		
0.45	5.3	5.1	12.7		
0.51	5.5	4.1	15.3		
TEST M3					
% error for each depth ratio					
Dr	DCM	SCM	LDMs	LDMs (calibrated)	
0.17	-5.2	-42.0	-8.6	-2.5	
0.19	-3.7	-38.9	-7.3	-1.0	
0.21	-8.0	-40.2	-11.6	-5.5	
0.30	-1.8	-28.3	-6.2	0.2	
0.30	-3.3	-29.3	-7.6	-1.3	
0.32	-5.4	-29.0	-9.7	-3.5	
0.37	-1.4	-22.1	-6.1	0.3	
0.44	5.0	-11.6	0.1	6.8	
0.48	4.0	-9.3	-0.5	6.0	
TEST M4					
% error for each depth ratio					
Dr	DCM	SCM	LDMs	LDMs (calibrated)	
0.18	-7.2	-59.7	-11.7	-9.5	
0.25	-2.4	-52.3	-8.1	-5.8	
0.33	1.0	-43.7	-6.3	-3.8	
0.35	5.4	-39.6	-2.6	0.1	
0.42	5.5	-33.5	-3.5	-0.7	
0.43	4.8	-33.5	-4.2	-1.5	
0.49	10.9	-23.1	0.8	3.7	
0.52	12.9	-18.4	2.7	5.6	
0.54	11.6	-16.8	1.6	4.5	

Table 5. (Continued).

TEST M5				
% error for each depth ratio				
Dr	DCM	SCM	LDMs	LDMs (calibrated)
0.19	-3.9	-69.2	-11.7	-9.5
0.21	-2.9	-67.8	-12.9	-10.7
0.33	4.5	-57.0	-5.0	-2.3
0.40	9.5	-49.0	-5.0	-2.0
0.48	14.2	-40.0	-5.0	-1.8
0.54	17.9	-30.8	-0.4	3.1
0.55	18.1	-30.1	0.2	3.7
0.59	20.0	-23.1	2.6	6.1
TEST M6				
% error for each depth ratio				
Dr	DCM	SCM	LDMs	LDMs (calibrated)
0.21	-4.5	-77.5	-10.4	-8.1
0.23	-11.0	-78.2	-16.9	-14.7
0.36	6.8	-66.4	-3.8	-0.8
0.45	13.8	-57.9	-0.3	3.3
0.46	16.0	-56.6	1.4	5.1
0.46	10.9	-58.2	-3.2	0.4
0.52	15.3	-50.9	-1.0	2.8
0.51	11.9	-53.3	-3.8	-0.1
0.56	13.2	-47.5	-3.6	0.2
0.59	13.9	-43.4	-3.3	0.5
0.60	14.8	-41.6	-2.6	1.2

consistently underpredicted and the DCM overpredicted the measured data for each case of roughened surfaces.

- (iii) For fixed bed data, the average errors for each method were **25.2%**, **24.2%** and **6.6%** for SCM, DCM and LDMs (λ value of 0.13) respectively and the average error was **0.3%** for LDMs with the calibrated values of λ .

For mobile boundary cases;

- (i) For mobile main channel and smooth floodplain cases, the modelling of the stage discharge relationship still remains as problematic as discussed by Cassells *et al* (2001), Seckin (2004), and Atabay & Knight (2006). LDMs, with the λ value of 0.3, produced more accurate results than those of SCM and DCM for asymmetrical case. LDMs failed for the symmetrical case for high depth ratios in comparison with the SCM and DCM. However, LDMs would have produced better results if the varying values of λ for the same data sets had been used. However, in this case, the values of λ would not have been in the range of its typical default values.
- (ii) For mobile main channel and rough floodplain cases, the LDMs, with the λ value of 0.13, still gave more accurate results than that of SCM and DCM for most of the cases. However, the accuracy of them was highly increased with the calibrated λ values. SCM consistently underpredicted the measured data for each case. The accuracy of DCM decreased with increasing densities of roughness elements.

- (iii) For mobile bed data, the average errors for each method were **28.3%**, **5.2%** and **−0.7%** for SCM, DCM and LDMs (with λ value of 0.13). The average error was **1.0%** for LDMs with the calibrated values of λ .

For overall data (104 measured data points), the average errors for each method were LDMs, with λ value of 0.13, **2.8%**, DCM **14.3%** and SCM **−26.8%**. The average error was **0.5%** for LDMs with the calibrated values of λ .

The authors are grateful to Geoff Denham, for his invaluable comments in improving this paper and for his proof reading.

A	cross-sectional area
C	Chezy friction factor
D_r	depth ratio = $(H - h)/H$
d_{35}	mean sediment size
f	Darcy–Weisbach friction factor
g	gravitational acceleration
h	main channel bankfull depth
H	flow depth
K	section conveyance
L_m	distance between two metal meshes
n	Manning roughness coefficient
n_{mc}	Manning roughness coefficient for the main channel
n_{fp}	Manning roughness coefficient for the floodplain
P	wetted perimeter
R	hydraulic radius
s	main channel lateral side slope
S_0	channel bed slope
Q	discharge
Q_c	calculated discharge
Q_m	measured discharge
u	local velocity in the streamwise (x) direction
U_0	mean velocity
U_d	depth-averaged mean velocity
v	local velocity in the lateral (y) direction
w	local velocity in the vertical (z) direction
λ	dimensionless eddy viscosity
τ_{yx}	Reynolds stress on plane perpendicular to the y direction
τ_{zx}	Reynolds stress on plane perpendicular to the z direction
ρ	water density
Γ	secondary flow term
ε	lateral eddy viscosity

References

- Ackers P 1993 Flow formulae for straight two-stage channels. *J. Hydraulic Res.* 31(4): 509–531

- Atabay S 2001 *Stage-discharge, resistance and sediment transport relationships for flow in straight compound channels*. PhD thesis, The University of Birmingham, England, U.K
- Atabay S, Knight D W 1999 Stage discharge and resistance relationships for laboratory alluvial channels with overbank flow. *Proc. of the 7th International Symposium on River Sedimentation, Hong Kong* : 223–229
- Atabay S, Knight D W, Seckin G 2004 Influence of a mobile bed on the boundary shear in a compound channel. *Proc. Int. Conf. on Fluvial Hydraulics, River Flow, Naples, Italia* Vol. I: 337–345
- Atabay S, Knight D W, Seckin G 2005 The effects of overbank flow on fluvial sediment transport rates. *Proc. of the Institution of Civil Engineers, Water Management* 158(1): 25–34
- Atabay S, Knight D W 2006 1-D modelling of conveyance, boundary shear and sediment transport in overbank flow. *J. Hydraulic Res.* 44(6): 739–754
- Atabay S, Seckin G 2000 Comparison of sediment transport rates between symmetric and asymmetric straight compound channels. *Proc. of the 8th International Symposium on Stochastic Hydraulics, China* : 139–144
- Ayyoubzadeh S A 1997 *Hydraulic aspects of straight compound channel flow and bedload sediment transport*. PhD thesis, The University of Birmingham, England, U.K
- Bousmar D, Zech Y 1999 Momentum transfer for practical flow computation in compound channels. *J. Hydraulic Eng.* American Society of Civil Engineers 125: 696–706
- Cassells J B C, Lambert M F, Myers R W C 2001 Discharge prediction in straight mobile bed compound channels. *Proc. of the Institution of Civil Engineers, Water & Maritime Engineering* 148(3): 177–188
- Ervine D A, Babaeyan-Koopaei K, Sellin R H J 2000 Two-dimensional solution for straight and meandering overbank flows. *J. Hydraulic Eng.* 126: 653–669
- Knight D W, Demetriou J D 1983 Floodplain and main channel flow interaction. *J. Hydraulic Eng.* 109: 1073–1092
- Knight D W, Hamed M E 1984 Boundary shear in symmetrical compound channel. *J. Hydraulic Eng.* ASCE 110(10): 1412–1430
- Knight D W 1999 Flow mechanisms and sediment transport in compound channels. *Int. J. Sediment Res.* 14(2): 217–236
- Knight D W, Brown F A, Ayyoubzadeh S A, Atabay S 1999 Sediment transport in river models with overbank flow. *Proc. of 7th International Symposium on River Sedimentation, Hong Kong* : 19–25
- Knight D W, Omran M, Tang X 2007 Modelling depth-averaged velocity and boundary shear in trapezoidal channels with secondary flows. *J. Hydraulic Eng.* ASCE 133(1): 39–47
- Lambert M F and Sellin R H J 1996 Discharge prediction in straight compound channels using the mixing length concept. *J. Hydraulic Res.* 34: 381–394
- Lambert M F, Myers W R C 1998 Estimating the discharge capacity of straight compound channels. *Proc. Inst. Civil Engineers, Water and Maritime Eng.* 130(2): 84–94
- Myers W R C and Elsayy E M 1975 Boundary shear in channel with floodplain. *J. Hydraulics Res.* 101: 933–946
- Myers W R C 1978 Momentum transfer in a compound channel. *J. Hydraulic Res.* 16: 139–150
- Myers W R C, Lyness J F, Cassells J 2001 Influence of boundary roughness on velocity and discharge in compound river channels. *J. Hydraulic Res.* 39: 311–319
- Omran M 2005 *Modelling stage-discharge curves, velocity and boundary shear stress distributions in natural and artificial channels using a depth-averaged approach*. PhD Thesis, The University of Birmingham, England, UK
- Prinos P, Townsend R D 1984 Comparison of methods for predicting discharge in compound open channels. *Advances in Water Resources* 7: 180–187
- Prooijen B C, Battjes J A, Uijtewaal W S 2005 Momentum exchange in straight uniform compound channel flow. *J. Hydraulic Eng.* 131: 175–183
- Seckin G 2004 A comparison of one-dimensional methods for estimating discharge capacity of straight compound channels. *Canad. J. Civil Eng.* 31: 619–631

- Seckin G, Atabay S 2005 Experimental backwater analysis around bridge waterways. *Canad. Civil Eng.* 32(6): 1015–1029
- Sellin R H J 1964 A laboratory investigation into the interaction between flow in the channel of a river and that of its floodplain. *La Houille Blanche* 7: 793–801
- Shiono K, Knight D W 1989 Two dimensional analytical solution compound channel. *Proc. of 3rd Int. Symp. on refined flow modelling and turbulence measurements*, Universal Academy Press : 591–599
- Shiono K, Knight D W 1991 Turbulent open channel flows with variable depth across the channel. *J. Fluid Mech.* Cambridge, U.K. 222: 617–646
- Tang X, Knight D W 2001 Experimental study of stage-discharge relationships and sediment transport rates in a compound channel. *Proc. of 29th IAHR Congress, Beijing, China* : 69–76
- Tang X 2002 Flow roughness of wire mesh (unpublished report, No. 7), Birmingham University
- Tang X, Knight D W 2006 Sediment transport in river models with overbank flows. *J. Hydraulic Eng.* 132(1): 77–86
- Wark J B, Samuels P G, Ervine D A 1990 A practical method of estimating velocity and discharge in compound channels. *Proc. of the International Conference On River Flood Hydraulics* (U.K: Wiley) 163–172
- Wormleaton P R, Allen J, Hadjipanios P 1982 Discharge assessment in compound channel flow. *J. Hydraulics Res.* American Society of Civil Engineers 108: 975–994
- Wormleaton P R, Hadjipanios P 1985 Flow distribution in compound channels. *J. Hydraulic Res.* American Society of Civil Engineers 111: 357–361
- Wright R R, Carstens H R 1970 Linear momentum flux to overbank sections. *J. Hydraulics Res.* American Society of Civil Engineers 96: 1781–1793
- Zheleznyakov G V 1971 Interaction of channel and flood plain flows. *Proc. of the 14th International Conference of the International Association for Hydraulic Research*, Paris : 144–148