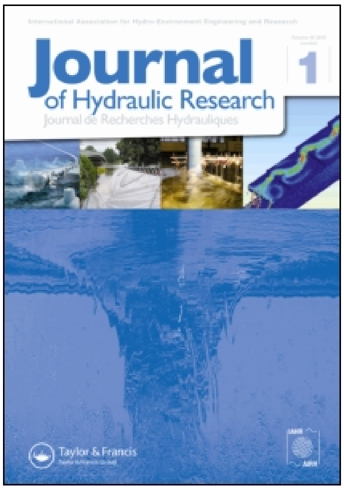

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Discharge estimation in open channels by means of water level hydrograph analysis

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Research paper

Discharge estimation in open channels by means of water level hydrograph analysis

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ABSTRACT
A new methodology, based on the synchronous measurement of stage hydrographs in two river sections located some kilometres from each other, was developed to estimate the discharge hydrograph in the upstream section. The methodology is based on the one-parameter calibration of a numerical flow routing algorithm, solving the Saint-Venant equations in diffusive or complete form. The methodology was validated using results of laboratory experiments carried out at the Polytechnic of Bari University. A known discharge hydrograph was generated in the upstream tank of a rectangular flume, where two water level sensors were located. Two different bed materials have been used to account for different roughness coefficients. Eight measured discharge hydrographs have been compared with the hydrographs computed using both a diffusive and a fully dynamic model. The diffusive model provides a good estimate of the measured discharge in the experiments with the highest roughness value.

Keywords: Calibration, discharge estimation, experimental validation, flow meter, shallow water flow

1 Introduction

Direct measurement of discharge in large channels or natural rivers is traditionally obtained by spatial integration of measured local velocities. The velocities are obtained with mechanical or electromagnetic probes, in full contact with the flow at the measured point. More recently, all velocity profiles along a given radius starting from the instrument transducer can be obtained from acoustic Doppler current profilers (Mueller 2003, Hirsch and Costa 2004, Stone and Hotchkiss 2007), measuring the Doppler shift of the backscattered acoustic signal reflected by the solid particles moving within the flow. These and also other instruments fully submerged into the flow may be easily subject to damage, rendering their use during floods difficult.

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The main problem in flood discharge measurement is that it is difficult or even impossible to measure the flow velocity in the lower flow portion. This implies the absence of experimental flow velocity data for high water levels which are important in the rating curve assessment. This issue has been addressed in the literature by analysing the relationship between the mean and the maximum velocity, which is generally located in the upper flow portion, where velocity measurements can be conducted also during high flow conditions without affecting the safety requirements. Based on the entropy model developed by Chiu (1988, 1991), a linear relationship between the maximum and the mean velocity has been observed (Chiu and Chen 2003, or Moramarco *et al*. 2004). The maximum velocity can be approximately measured at the water surface by means of radar or optical instruments (Fujita *et al*. 1997).

Because of the above-mentioned difficulties for direct discharge measurement, gauged sections in natural rivers and artificial channels are usually equipped with water level sensors and the measured water levels are related to the discharges by means of a rating curve, based on a one-to-one relationship between water depth and discharge, a hypothesis that is strictly true only according to the kinematic assumption. This holds in many gauged sections, located in the upper part of a basin, with errors in the discharge estimation of only a few percents for a given water depth. Yet, the use of a rating curve has several drawbacks. The rating curve is difficult to compute because for almost all natural rivers it requires direct velocity measurements. The hydraulic resistance and the river geometry are subject to frequent changes due to erosion/deposition processes, as well as to seasonal changes in vegetation (Barry *et al*. 1992, Burguete *et al*. 2007), implying frequent curve reconstruction. Further, direct velocity measurement is hardly made during extreme hydrological events. For almost all the available rating curves, the higher stage–discharge data are obtained from the analysis of really measured values.

To partially cope with these difficulties, the use of two water level sensors located in two different river sections was proposed approximately 10 years ago (Aricò *et al*. 2009). Various physically-based models were proposed to relate the measured upstream discharge hydrograph and/or the measured water level hydrographs in both sections (Moramarco *et al*. 2005, Tayfur and Moramarco 2008). All mentioned models require knowledge of at least one directly measured discharge for model calibration. Perumal *et al*. (2007) and Aricò *et al*. (2007, 2009) applied their flow routing algorithms to directly relate the upstream flow depth hydrograph with the downstream discharge hydrograph, using the measured downstream flow depth hydrograph for the flow routing model calibration. The proposed approach was validated using field data from Italian rivers where rating curves along with synchronous stage hydrographs were available located some kilometres apart. Due to the previously discussed uncertainty in the rating curve, a set of experiments aimed at the laboratory validation of the proposed methodology was recently conducted in the laboratory flume of the Water Engineering and Chemistry Department, Technical University of Bari (Italy) with the results presented below.

Table 1 Possible boundary conditions of Eqs. (1)–(3)

	$\mathsf{F} > 1$	$\mathsf{F} < 1$
Upstream	$h = h_u^*$ and ($\mathrm{d}h/\mathrm{d}x = 0$ or $q = q_u^*$)	$h = h_u^*$
Downstream	None	$h = h_u^*$ or zero diffusion

2 Discharge hydrograph from synchronous water level measurements

The one-dimensional shallow-water continuity and momentum equations are

$$\frac{\partial A}{\partial t} + \frac{\partial q}{\partial x} = 0 \tag{1}$$

$$\frac{\partial q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{q^2}{A}\right) + gA\frac{\partial h}{\partial x} + gA(S_f - S_o) = 0 \tag{2}$$

where x and t are the space and time coordinates, A the cross-section area, q the discharge, h the flow depth, g the gravity acceleration, and S_f and S_o the energy line and bottom slopes. According to the Chézy relationship

$$S_f = \frac{q^2 n^2}{A^2 R_h^{4/3}} \tag{3}$$

where R_h is the hydraulic radius and n Manning's coefficient. The basic idea of the indirect approach for discharge estimation in rivers (Perumal *et al*. 2007, Aricò *et al*. 2009) is to estimate n by calibrating Eqs. (1)–(3) using synchronous stage hydrographs measured at two different river sections.

Observe that the proper boundary conditions required for the existence of a unique solution to Eqs. (1)–(3) depend on the Froude number at the two ends of the computational domain at any given time (Akan 2006). Possible boundary conditions are stated in Table 1, in which $\mathsf{F} = q/[A(gh_a)^{1/2}]$ is the Froude number, h_a the cross-sectional average flow depth, subscripts u or d denote "upstream" and "downstream" sections, respectively, and (*) stands for "assigned value". The upstream condition required for supercritical flow should be the second within the brackets of Table 1 to obtain the real profiles in the field. Since the upstream stage–discharge relation is unknown, it is replaced by the first boundary condition, which is equivalent to assuming in the upstream section the kinematic approximation for supercritical flow. If the flow is subcritical in both gauged sections, the flow depth at the two ends of the reach are always fixed as a boundary condition and the computed stage–discharge relation is a function of the actual, unknown Manning coefficient.

Strictly speaking, three gauged sections are required to make the corresponding calibration problem always well-posed. Using numerical experiments (Aricò *et al*. 2009), the water level profiles resulting from the use of a Manning coefficient different from the real value and the known stage hydrographs, given both as boundary conditions, display artificial waves along the reach that can only be justified if special perturbations downstream of the final section occur. A good calibration of n can still be obtained using only two gauged sections by extending the computational domain beyond the second gauged section and approximating the downstream boundary condition at the new end of the computational domain with the zero diffusion approach if $\mathsf{F} < 1$ (Table 1). Extending the computational domain by half the distance between the two gauged sections is enough to obtain the maximum improvement in discharge estimation.

The calibration of n is carried out by minimizing an error function, given by the square difference between the computed and the observed water levels in the downstream gauged section. The observation period is chosen before and immediately after the peak of the downstream stage hydrograph because (1) the corresponding peak discharge is usually the most important discharge hydrograph parameter, and (2) the slope of the downstream stage hydrograph is usually larger in its rising part, along with the sensitivity of the error function. The minimum search is equivalent to the solution of

$$\int_{t_1}^{t_2} \left(h_d(t,n) - h_d^*\right) \frac{\partial h_d}{\partial n}\,\mathrm{d}t = 0 \tag{4}$$

where t_1 and t_2 are temporal observation limits. The sensitivity of the downstream flow depths with respect to the Manning coefficient, which is the derivative in the left-hand side of Eq. (4), is numerically estimated at each iteration of the root solver by perturbing n with a small quantity of $\Delta n = 0.00001$ s/m$^{1/3}$.

3 Laboratory experiments

The experimental validation of the above procedure was made in a laboratory flume of the Water Engineering and Chemistry Department, Technical University of Bari (Italy). The channel of bottom slope $S_o = 0.0006$ was 25 m long, 0.40 m wide and 0.50 m high. The bottom and the walls of the channel were of Plexiglas. A sharp-crested weir, located at the downstream channel end, was used to provide initial uniform flow conditions. The flood discharge entering the upstream tank was regulated by an electronic control device, which controlled the opening and the closing of an electro-valve to obtain the Q_0 discharge hydrograph. The actual discharge was measured by an electromagnetic flow meter.

The previously described apparatus had a maximum discharge of nearly 0.080 m^3/s with a minimum rising time of about 3 s and a return time of about 12 s. Two suitably calibrated level gauges were installed at different channel sections to measure the flow level hydrographs. A third level gauge was used to measure the water level hydrograph inside the tank. A computer interlocking system for acquisition and control of the various data was used. The LabView™ interface on the computer also allowed generating the desired hydrograph. Two series of runs were conducted. For Run A1 and Run A2 the channel bottom was smooth, whereas for Runs B1–B6, the channel bottom was covered with fixed, rough gravel of diameter ~40 mm. Figure 1 shows a definition sketch of the experimental device.

4 Estimation of upstream discharge hydrograph from instrument data and sensitivity analysis

The discharge Q_0 pumped in the upstream tank and the water levels h_1, h_2 and h_3 in the up- and downstream monitored sections and in the upstream tank were recorded (Fig. 1). All experimental data were sampled with a frequency of $f = 12.5$ Hz. An analysis of the raw data power spectra was carried out to identify the lower noise frequency, henceforth called cut frequency f_{cut}. The data were filtered by dropping data with $f > f_{cut}$ by means of a low-pass filter (Pulci Doria 1992, Tropea *et al*. 2007). The cut frequencies range from 0.037 Hz, for h_1 in Run B6, to 0.746 Hz for Q_0 in Run B1.

The upstream reference discharge hydrograph was estimated by assuming a constant (in space) water level inside the upstream

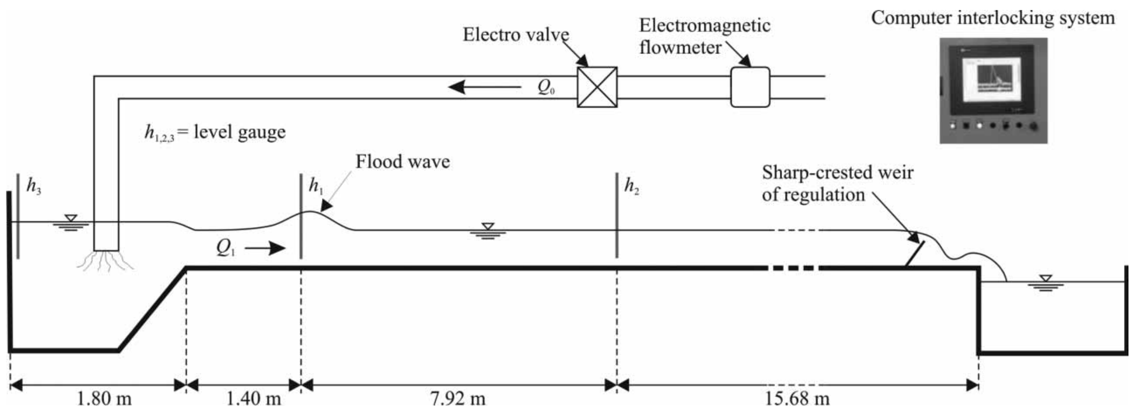

Figure 1 Definition sketch of experimental channel (side view)

tank. The mass conservation can then be written as

$$\frac{\mathrm{d}V}{\mathrm{d}t} \cong \Sigma \frac{\mathrm{d}h_3}{\mathrm{d}t} = Q_0 - Q_1 \tag{5}$$

where V is the volume, Σ the horizontal upstream tank area (Fig. 1) and Q_1 the discharge entering the channel. The upstream discharge was estimated according to Eq. (5) as

$$Q_1(t) = Q_0(t) - \Sigma \frac{h_3(t+\Delta t) - h_3(t)}{\Delta t} \tag{6}$$

Because the discharge estimation error is proportional to the error in the Manning coefficient estimation, it is important to associate to each estimated discharge hydrograph the n estimation error. Assuming n to be the most likely value of a random parameter, linked to the error function by a deterministic model, the parameter estimation theory of Kendall and Stuart (1973) was applied to estimate the variance of its expected value. The optimum Manning (subscript M) coefficient variance σ_M is given by the inverse of the curvature of the error function around its minimum (Carrera and Neuman 1986). Assuming a first-order approximation of the computed water levels around their optimum value, it can be shown that

$$\frac{1}{\sigma_M} = \frac{\sum_{i=1,N} J_i^2}{\sigma_h} \tag{7}$$

where N denotes the number of time steps in observation period, σ_h the variance of head (subscript h) measurement error and J_i the sensitivity of the downstream water head computation error at time t_i (subscript i) with respect to n, i.e.

$$J_i = \left.\frac{\partial(h_i - h_i^*)}{\partial n}\right|_{n=n_{optimum}} \tag{8}$$

where $n_{optimum}$ represents optimum n value obtained from the calibration procedure.

5 Performance criteria

The performance of the discharge estimation procedure was evaluated using three criteria (Aricò *et al.* 2009):

(1) Nash and Sutcliffe (1970) criterion:

$$\eta_q = \left[1 - \frac{\sum_{i=1,N}(q_i^* - q_i)^2}{\sum_{i=1,N}(q_i^* - \overline{q^*})^2}\right] \times 100 \tag{9}$$

where q_i^* is the ith data of benchmark discharge hydrographs, q_i the ith data of simulated discharge hydrographs and $\overline{q_i^*}$ the average value of benchmark discharge hydrographs.

(2) Relative magnitude peak error:

$$q_{per} = \left[\frac{q_p}{q_p^*} - 1\right] \times 100 \tag{10}$$

where q_p denotes the peak (subscript p) value in computed discharge hydrographs, while q_p^* the peak reference value.

(3) Relative time to peak error:

$$t_{pqer} = t_{pq} - t_{pq^*} \tag{11}$$

where t_{pq} stands for the peak time value of computed discharge hydrographs, while t_{pq^*} the reference value.

6 Estimation of discharge hydrographs for Runs A

The measured downstream A stage hydrographs are shown in Fig. 2. According to the above criteria, the observation period $t_1 = 43$ s and $t_2 = 48$ s was chosen for the square error computation in both Runs A1 and A2. The roots of the resulting error functions are shown in Fig. 3. Note that they are very flat in the region of the physically feasible Manning coefficients. According to these results, the sensitivity of the downstream stage hydrograph to n is small and the estimation of its optimum value very ambiguous. On the other hand, Fig. 4 shows that the shape of the discharge hydrographs computed with two feasible Manning coefficients are close to that

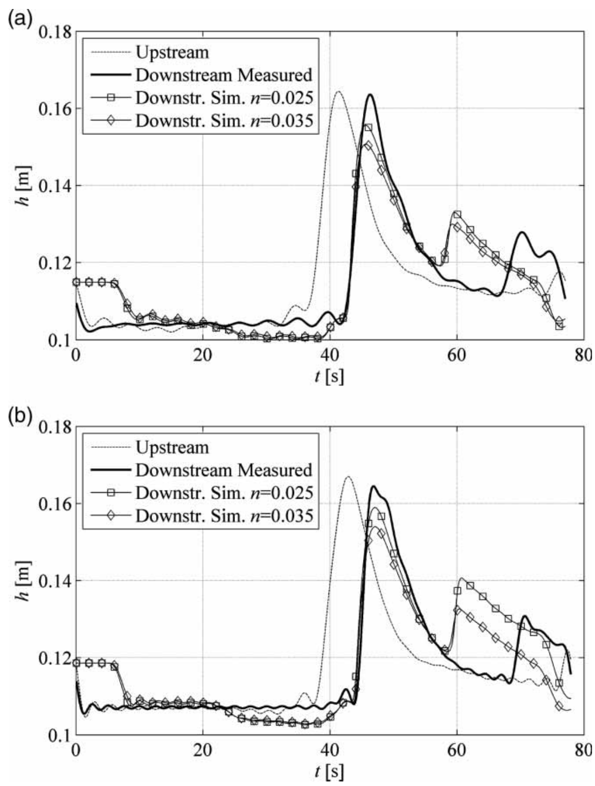

Figure 2 Measured and computed water stage hydrographs of Runs (a) A1, (b) A2, n (s/m$^{1/3}$)

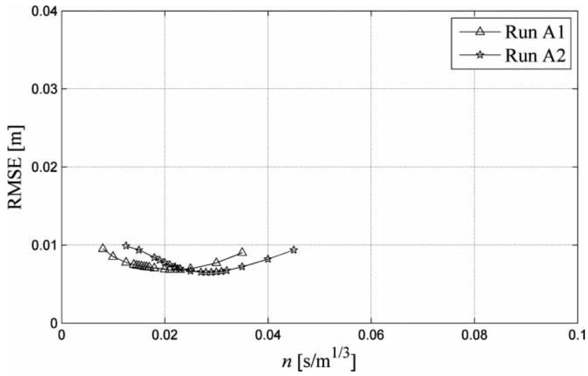

Figure 3 Root mean square error (RMSE) function of Runs A1 and A2

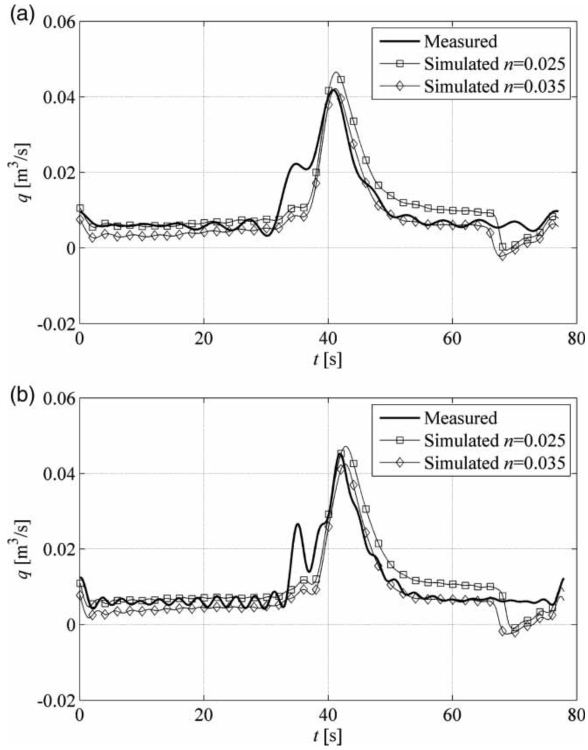

Figure 4 Upstream measured and computed discharge hydrographs of Runs (a) A1, (b) A2, n (s/m$^{1/3}$)

measured, especially in Run A2. For the given small bottom slope and bed roughness, the inertia terms prevail over the gravity and resistance terms in Eq. (2) and it is always possible to obtain a good discharge estimation by simply assigning the measured stage hydrograph as upstream boundary condition of the routed wave and using a physically feasible value of the Manning coefficient.

7 Estimation of discharge hydrographs for Runs B

A diffusive flow routing solver was applied in the calibration procedure of Runs B, along with the previous for solving Eqs. (1) and (2). The diffusive solver neglects the inertial terms in Eq. (2) and always requires one upstream and one downstream conditions, as used in Table 1 for the complete model under subcritical flow. In Figs. 5–7, the measured and the computed stage and discharge hydrographs of Runs B1, B2 and B4 are shown. The variance of the optimal Manning coefficient, computed

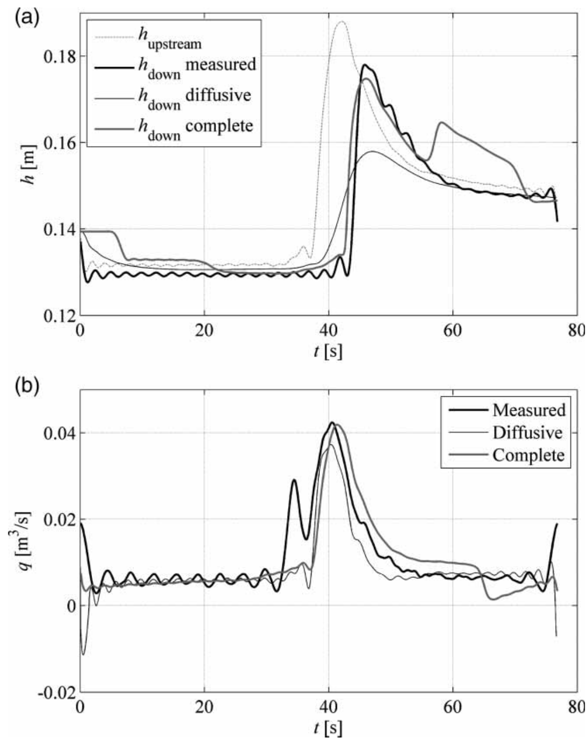

Figure 5 (a) Measured and computed flow depths and (b) discharge hydrographs of Run B1

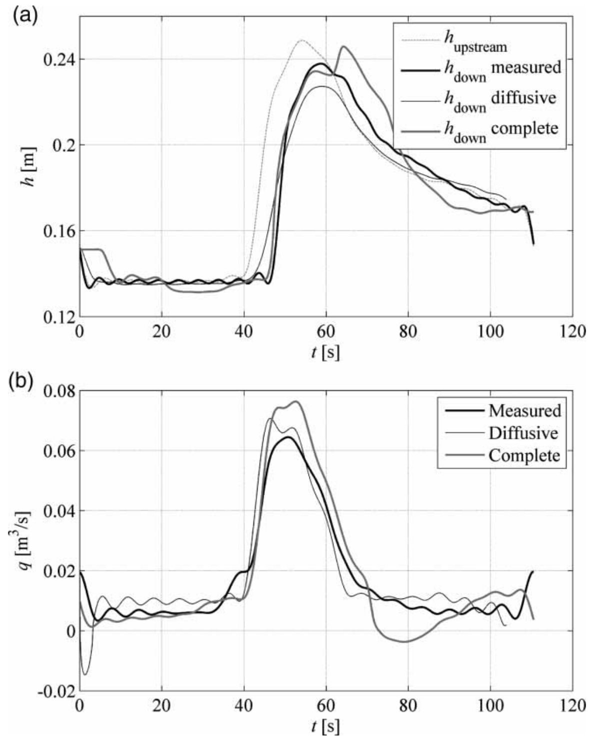

Figure 6 (a) Measured and computed stage and (b) discharge hydrographs of Run B2

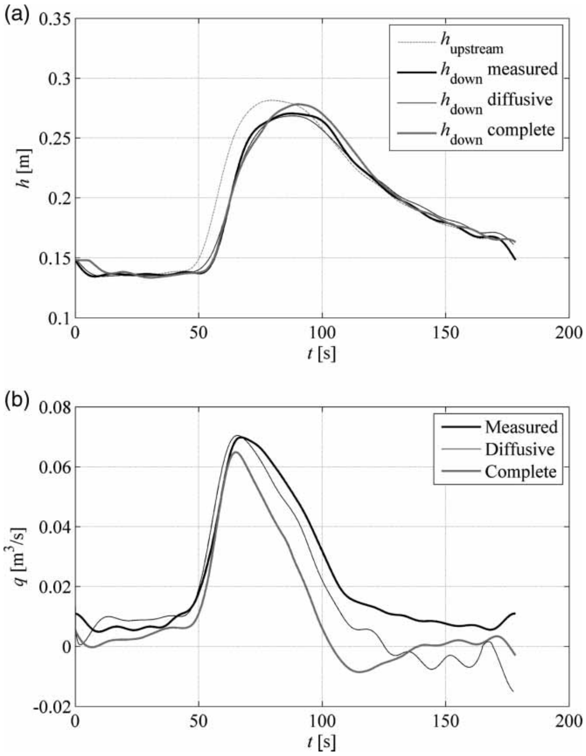

Figure 7 (a) Measured and computed stage and (b) discharge hydrographs of Run B4

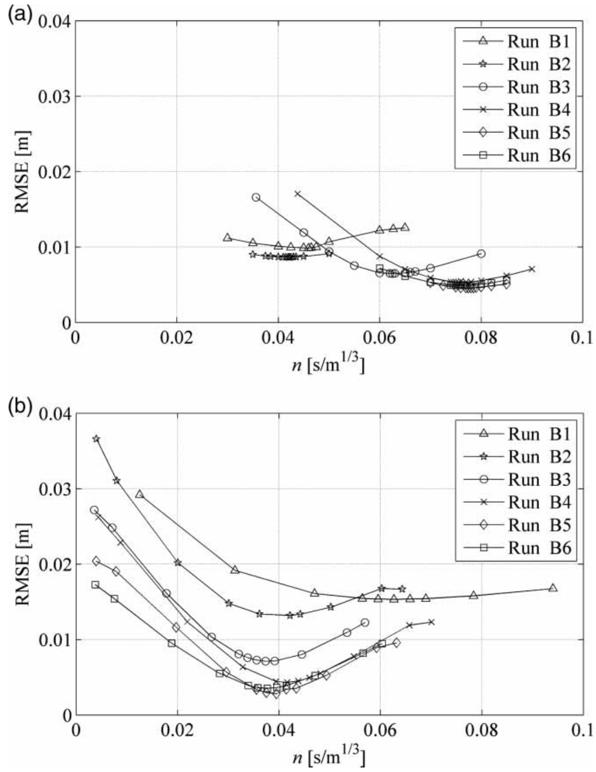

Figure 8 (a) RSME of Runs B1–B6 for complete and (b) diffusive numerical models

according to Eq. (7), is summarized in Table 2, while the corresponding performance criteria defined in Section 5 are summarized for all Runs B in Table 3, for both diffusive and complete modelling. RMSE values of the error function are shown in Fig. 8.

Note that the Manning coefficient estimation error is much smaller in all Runs B than in Runs A, along with the discharge estimation error, using the diffusive instead of the complete model, because of the lack of inertial terms in the momentum equation of the diffusive model. These terms are independent of n and cannot be fitted to the experimental data in the calibration procedure. This limits the capability of the calibration procedure in compensating the model error of the complete model, present mainly in the approximated boundary conditions.

The sensitivity of the error function and the corresponding Manning coefficient estimation error are also related to the maximum flow depth; for larger flow depths (and velocities) bed resistance plays the major role, the variance of n is much

Table 2 Optimum parameter error for unit head measurement error for complete and diffusive model for Runs B

σ_M/σ_h (s^2/m$^{8/3}$)	B1	B2	B3	B4	B5	B6
Diffusive	0.199	0.011	8.720×10^{-3}	2.166×10^{-4}	7.594×10^{-3}	9.537×10^{-3}
Complete	4.850	0.719	2.582	12.543	272.46	1405.4

Table 3 Performance of estimated hydrograph in Runs B with diffusive and dynamic numerical models

	Diffusive				Complete			
Run	$n_{opt.}$ (s/m$^{1/3}$)	q_{per} (%)	t_{pqer} (s)	η_q (%)	$n_{opt.}$ (s/m$^{1/3}$)	q_{per} (%)	t_{pqer} (s)	η_q (%)
B1	0.0627	−12.140	−0.24	67.915	0.0460	−1.205	0.8000	70.9376
B2	0.0402	9.676	−4.40	44.713	0.0422	18.384	1.8400	13.7146
B3	0.0356	10.267	−2.48	1.852	0.0625	−3.8624	−2.4800	56.7709
B4	0.0438	0.883	−1.52	73.170	0.0755	−7.1040	−2.1600	−65.0934
B5	0.0395	7.658	−3.68	70.130	0.0780	−10.6399	−4.1600	−118.4350
B6	0.0377	1.440	3.60	90.563	0.0760	−25.0984	−4.3200	17.2112

lower and the performance criteria of the computed discharges are much better in the diffusive model for Runs B2–B6, except for Run B3 (Table 3 and Figs. 5–7). For lower flow depths, like in Run B1 (compare Fig. 5a with Figs. 6a and 7a), the inertial terms play the major role and the diffusive model is no more suitable. Then, the complete model produces better results, as shown in Table 3 and Fig. 5. A comparison of stage and discharge hydrographs obtained using the calibration of the diffusive and complete models is also shown in Figs. 5–7. The larger error obtained with the complete models is consistent with the larger Manning coefficient estimation error shown in Table 3.

8 Conclusions

An analysis of experimental data collected in the Laboratory of the Polytechnic of Bari underlines the major role of the sensitivity analysis of the downstream stage hydrograph with respect to the Manning coefficient, as computed by a flow routing model in the application of the present procedure for discharge hydrograph estimation. If the sensitivity of the computed downstream flow depths with respect to the Manning coefficient is large enough, the application of the procedure using a diffusive model provides a stable estimation of the Manning coefficient and a good match between the measured and the estimated upstream discharge hydrographs. According to the laboratory data, a consistent change of the error function of 10% or more within a limited variation of the optimal Manning coefficient of 20% or less is a robust index of a reliable peak discharge estimation. In the remaining cases, a rough approximation of the discharge hydrograph of some 10% difference in the peak estimation can still be obtained using a physically-reasonable Manning coefficient instead of the optimum and using the complete dynamic instead of the diffusive model.

Notation

A = cross-sectional area (m^2)
F = Froude number (–)
g = gravitational acceleration (m/s^2)
J = sensitivity of downstream water head computation error ($m^{4/3}/s$)
h = water level (m)
h^* = measured water level (m)
N = number of time steps in observation period (–)
n = Manning coefficient ($s/m^{1/3}$)
Q_0 = discharge entering upstream tank (m^3/s)
Q_1 = discharge entering channel (m^3/s)
q = discharge in channel (m^3/s)
q^* = measured discharge (m^3/s)
$\overline{q^*}$ = average measured discharge (m^3/s)
q_{per} = relative magnitude peak error (%)
R_h = hydraulic radius (m)
S_f = friction slope (–)
S_o = bottom slope (–)
t = time (s)
t_1, t_2 = time observation limits (s)
t_{pq} = time to peak of computed discharge hydrograph (s)
t_{pq^*} = time to peak of measured discharge hydrograph (s)
t_{pqer} = relative time to peak error (s)
V = tank volume (m^3)
x = streamwise coordinate (m)
η_q = Nash–Sutcliffe criterion (%)
Δt = time step (s)
Σ = horizontal tank area (m^2)
σ_M = Manning coefficient variance ($s^2/m^{2/3}$)
σ_E = variance of head measurement error (m^2)

References

Akan, A.O. (2006). *Open channel hydraulics*. Elsevier and Butterworth-Heinemann, Burlington MA.

Aricò, C., Nasello, C., Tucciarelli, T. (2007). Peak flow estimation by means of synchronous water level measurements. Proc. *EGU General Assembly*. Wien, Austria.

Aricò, C., Nasello, C., Tucciarelli, T. (2009). Using unsteady water level data to estimate channel roughness and discharge hydrograph. *Adv. Water Resour*. 32(8), 1223–1240.

Barry, M.F., Gates, T.K., Khattab, A.F. (1992). Field-measured hydraulic resistance characteristics in vegetation-infested canals. *J. Irrig. and Drain. Eng*. 118(2), 256–274.

Burguete, J., Garcia-Navarro, P., Murillo, J., Garcia-Palacin, I. (2007). Analysis of the friction term in the one-dimensional shallow water model. *J. Hydr. Eng*. 133(9), 1048–1063.

Carrera, J., Neuman, S. (1986). Estimation of aquifer parameters under transient and steady state conditions 1: Maximum likelihood method incorporating prior information. *Water Resour Res*. 22(2), 199–210.

Chiu, C.L. (1991). Application of entropy concept in open-channel flow study. *J. Hydr. Engng*. 117(5), 615–628.

Chiu, C.L. (1988). Entropy and 2-D velocity distribution in open channels. *J. Hydr. Engng*. 114(7), 738–756.

Chiu, C.L., Chen, Y.C. (2003). An efficient method of discharge estimation based on probability concept. *J. Hydr. Res*. 41(6), 589–596.

Fujita, I., Muste, M., Kruger, A. (1997). Large scale PIV for flow analysis in hydraulic applications. *J. Hydr. Res*. 36(3), 397–414.

Hirsch, R.M., Costa, J.E. (2004). U.S. stream flow measurement and data dissemination improve. *Trans. AGU* 85(20), 197–203.

Kendall, M.G., Stuart, A. (1973). *The advanced theory of statistics*. Griffin, London.

Moramarco, T., Saltalippi, C., Singh, V.P. (2004). Estimation of mean velocity in natural channels based on Chiu's velocity distribution equation. *J. Hydrologic Engng*. 9(1), 42–50.

Moramarco, T., Barbetta, S., Melone, F., Singh, V.P. (2005). Relating local stage and remote discharge with significant lateral inflow. *J. Hydrol. Engng*. 10(1), 58–69.

Mueller, D.S. (2003). Field evaluation of boat-mounted acoustic Doppler instruments used to measure stream flow. Proc. 7th IEEE Working Conf. *Current Measurement Technology* San Diego, 30–34.

Nash, J.E., Sutcliffe, J.V. (1970). River flow forecasting through conceptual models 1: A discussion of principles. *J Hydrology* 10, 282–290.

Perumal, M., Moramarco, T., Sahoo, B., Barbetta, S. (2007). A methodology for discharge estimation and rating curve development at ungauged river sites. *Water Resour Res* 43, W02412. doi:10.1029/2005WR004609.

Pulci Doria, G. (1992). Metodologie moderne di misure idrauliche e idrodinamiche. CUEN [in Italian]

Stone, M.C., Hotchkiss, R.H. (2007). Evaluating velocity measurement techniques in shallow streams. *J. Hydr. Res*. 45(6), 752–762.

Tayfur, G., Moramarco, T. (2008). Predicting hourly-based flow discharge hydrographs from level data using genetic algorithms. *J. Hydrology* 352, 77–93.

Tropea, C., Yarin, A.L., Foss, J.F., eds. (2007). *Handbook of experimental fluid mechanics*. Springer, Berlin.