Practical application of numerical modelling to overbank flows in a compound river channel

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ABSTRACT: Compound sections formed by a river channel and floodplains, are used in river channels design to provide additional conveyance capacity during high discharge periods. When the overbank flow occurs, the flow in the river channel is affected by the momentum transfer between the main channel and floodplains, which modifies water levels and velocity distributions given by traditional methods. One-dimensional (1D) models using the Single Channel Method (SCM) and the Divided Channel method (DCM) have been proven to be not accurate enough in compound channel flows. New more advanced models have been developed in order to accurately estimate discharge flows and depth-averaged velocity distributions. The quasi-two dimensional model Conveyance Estimation System (CES) estimates discharges and velocities in a cross-section based on the Lateral Distribution Method (LDM). Two-dimensional (2D) modelling solves the depth-averaged Navier-Stokes equations in a discretized reach of a river. In this work, published field measurements of the River Main (UK) are analyzed via 1D, CES and 2D modelling, in order to find a practical solution to give good predictions of water levels and velocity distributions in overbank flows in river channels with floodplains. The results show that 1D modelling combined with CES gives reasonable accurate values and is a complementary tool for advanced 2D models in real conditions.

1 INTRODUCTION

World population growth has gradually resulted in increased human settlements, developments and activities around the floodplains of rivers which lead to disastrous effects during flooding of natural rivers. River floods result in huge losses in human lives and economic losses. A third of the world's losses due to natural disasters is caused by flood disasters, flooding also accounts for half the loss of life with analyses of the trend showing that this figures have significantly increased (Berz, 2000). Accurate estimation of flow rate in channels is of enormous significance for flood prevention. Flooding occurs when the quantity of water flowing along a channel is higher than its carrying capacity. Hence the need for accurate prediction of river discharges during flood conditions to mitigate the impact, thereby saving lives and properties has drawn greater attention of researchers and engineers in recent times. There are numerous methods and approaches that have been employed in recent times to facilitate accurate estimation and prediction of discharge, conveyance and water surface level of rivers during overbank flow.

Previous work in compound open channels has been mainly focused on modelling uniform flow conditions and compared with experimental data in laboratory flumes. Shiono and Knight (1991) developed a quasi-two dimensional model (based on lateral distribution method) to model conveyance in compound cross sections. This approach has been used in the Environment Agency's Conveyance Estimation System (CES). Mc Gahey et al (2008) demonstrated the ability of CES to accurately estimate lateral velocity distribution and discharges assuming uniform conditions in real rivers.

The aim of this work is to validate the application of one-dimensional Lateral Distribution Method (LDM) via the use of Conveyance Estimation System (CES) which is a commercial software for the estimation of discharge/conveyance capacity of compound channels and compare the results to that of the traditional one-dimensional methods, Single Channel Method (SCM) and Divided Channel

Method (DCM) using Hydrological Engineering Centre River Analysis System (HEC-RAS) software. The two-dimensional SRH-2D (Sedimentation and River Hydraulics 2 Dimensional) model is also used for comparison of velocity distributions.

The data that were utilized for successfully carrying out the simulation in this research work were obtained from the previous work of Martin and Myers (1991), Myers and Lyness (1994); Lyness and Myers (1994a) and Lyness and Myers (1994b), conducted on river Main, Northern Ireland. The aim of the project was achieved by simulation of the study reach of river Main (which is a reconstructed prototype river reach in Northern Ireland, United Kingdom) on both CES and HEC-RAS computational modelling software and the two-dimensional code SRH2D. The three codes have been applied by using the same boundary conditions, cross-section data and flow parameters in order to have the same criteria for comparison and validation. Finally the water surface level and velocity distribution results obtained from this software were analysed and compared with available field data to validate and verify the results.

A great effort has been made over the last decades to improve calculation of water levels and velocities in real rivers by the use of 2D and 3D modelling. However some important uncertainties are still unsolved. In this context, an accurate 1D model easy to calibrate and with the support of the CES can be an improved tool for comparison.

2 LITERATURE REVIEW

In compound channels, the velocity gradient between the main channel and floodplain flows generates shear forces in the main channel-floodplain interfaces. Sellin (1964) presented photographic evidence of the bank horizontal vortices acting along the interface and together with Zheleznyakov (1971) demonstrated a decrease in the main channel discharge after overbank flow occurs, only partially compensated by some discharge increase on the floodplain. The physics of flood hydraulics has been widely studied during the last 30 years (Knight and Shiono, 1996; Sellin, 1996 and Wormleaton et al 2004), concluding in a deep knowledge and understanding of the phenomenon involved.

Commercial models, such as HEC-RAS and MIKE 11, use calculation methods like SCM and DCM. The SCM considers the same velocity for the whole section. The DCM separates the cross-section into areas of different flow characteristics, such as the main channel and floodplains. Wormleaton et al. (1982) demonstrated that the SCM underestimates the conveyance capacity and the DCM overestimates compound channel. In the following years, several researchers presented some improved methods for compound channel flow estimation, Wormleaton and Merret (1990) proposed a simple modification that improves the DCM estimation and the DCM was empirically corrected by Ackers (1992). An alternative and more advanced method was developed in those years, the lateral distribution method (LDM) formulated by Wark et al. (1990) and the method by Shiono and Knight (1991). These two methods are based on the same equations and calculate the lateral velocity distribution in the cross section, like a quasi-2D model.

This paper aims to discuss refinements in 1D modelling that are able to cope with such complexities in a straightforward way. The research focuses on the prediction of the velocity distribution across the river. While the free surface profile is computed reasonably well by 1D numerical models, the same does not hold for the velocities unless the appropriate term at the interface is used. The methodology presented herein uses the HEC-RAS and the CES in order to improve 1D numerical modelling. The interaction between the main channel and the floodplain is modelled by using the lateral distribution of velocities given by CES. This method is applied to previously published field data from River Main. Moreover, the results given by widely used 2D models (SRH2D), will be used for comparison.

3 RIVER MAIN FIELD DATASET

The river data under study consist of a reach of the river Main, in Northern Ireland, which has some length of its reach reconstructed and realigned (between 1982 and 1986). This reach of the river comprises a trapezoidal compound channel with a centralised deep main channel bordered by one or two side berms. Numerous number of research works have been carried out on this river reach (Martin and Myers, 1991; Myers and Lyness, 1994; Lyness and Myers, 1994; Defra /Environmental Agency, 2003), with the aim of having a better understanding of the hydraulic behaviour of two-stage waterways. The measured study reach is found to have a longitudinal length of 800 meters from upstream (section 14) to downstream (section 6) with an average longitudinal bed of 0.003 or 1:520 with flood plains slope towards the main channel having a gradient of 1:25. It is divided into nine cross sections, situated at equal intervals of 100 meters apart. The plan view, upstream and the downstream cross sections of the river Main reach under investigation are shown in Figures 1 and 2.

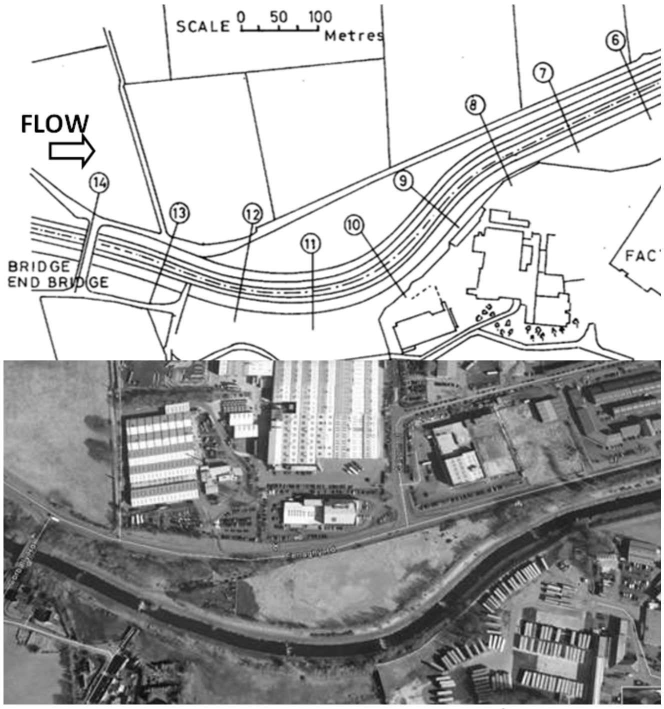

Figure 1. River Main plan view. Location of cross-sections from upstream (s14) to downstream (s6).

Figure 2. Upstream (doted) and downstream (full line) cross sections, numbers 14 and 6 respectively, of River Main reach of study.

The river bed material comprises of a very coarse gravel with a D_{50} size ranging between 100 and 200 mm. Quarried stones of up to 0.5 tonne in weight and having a size up to 1 m in diameter are used as a rip-rap to protect the side slopes of the main channel. The grass and weed that cover the berms are maintained regularly by keeping them short (Martin and Myers 1991). Figure 3 shows a cross-sectional view of the compound river channel and the bed materials in floodplains and river banks.

Table 1. Main geometric and hydraulic parameters in the River Main study reach.

	Upstream s14	Downstream s06
Long.. Bed slope	0.0052	0.0019
Bankfull flow	20.1	11-12
Manning n (m.c.)	0.39	0.39
Manning n (f.p.)	0.40	0.40
Bed width	12.2	11.1
Lateral slope (f.p.)	1:25	1:20


Figure 3. River Main: river channel and floodplains. Main channel is covered by cobbles, with medium rip-rap stones for the bank slopes and floodplains with natural grass.

Some typical water surface profiles measurements obtained using steady flow computation for the discharges of 10.5, 20.1 and 51.3 m3/s were shown by Myers and Lyness 1994 and reproduced in figures 5 and 6 in the next section. The 10.5 m3/s discharge corresponds to an inbank flow and the two higher discharges (20.1 and 51.3 m3/s) are overbank flows, the lower under the top floodplain level and the higher full covering the floodplains. Martin and Myers (1991), Lyness and Myers (1994b) and Lyness et al (1987) indicate that SCM underestimate discharges, while that of DCM overestimates it, revealing there is an exchange of momentum between floodplains and main channel, in overbank flow situation.

Table 2. Geometry in River Main sections.

Upstream S14		Downstream S06	
Y (m)	Z (m)	Y (m)	Z (m)
0.0	40.40	0.0	38.00
5.3	37.81	7.1	35.32
13.5	37.32	13.6	34.82
14.4	36.40	14.9	34.00
26.6	36.40	26.0	34.00
27.6	37.38	27.3	34.87
35.7	37.78	34.4	35.37
40.8	40.40	39.5	38.00

Roughness Manning's n coefficients were estimated by using uniform flow conditions in upstream cross-section 14 by Martin and Myers (1991) and Myers and Lyness (1994a). These studies found that using Manning's formula the inbank roughness decreased between n = 0.050 for low depths and n = 0.039 for bankfull. However the Manning's n for the gravels/cobbles in the bed varies between n = 0.025-0.039, and the Manning's n for the riprap on the bank is higher than n = 0.040. This means that as the water depth increases the main channel mean roughness should be greater, which does not fit the estimation of Manning's roughness by using the mean bed slope and field rating curves.

Table 3. Slope and distance between cross-sections.

Section	Distance	Bed Level	Bed slope
14-13	100	36.40	0.0055
13-12	100	35.85	0.0032
12-11	100	35.53	0.0031
11-10	100	35.22	0.0012
10-9	100	35.10	0.0058
9-8	100	34.52	0.0017
8-7	100	34.35	0.0018
7-6	100	34.17	0.0017
6-…	0	34.00	0.0030

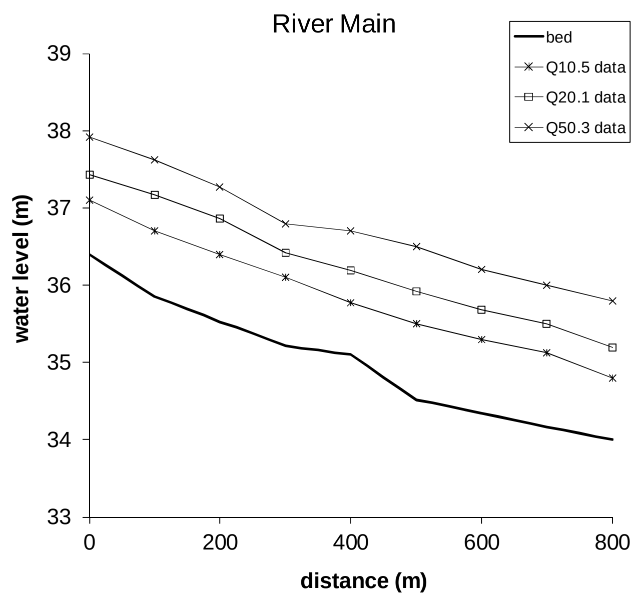

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Figure 4. River Main bed profile and water level profiles measured in fieldworks for inbank (10.5), and overbank (20.1 and 50.3) flows (after Myers and Lyness 1994).

4 NUMERICAL MODELS

In the next subsections the models used in the present work are briefly described. The 1D HEC-RAS model (USACE, 2008), the CES model (Environment Agency, 2004), and the SRH2D (Lai, 2008)

4.1 *HEC-RAS 1D Model*

The results obtained with 1D modelling based on the energy or Bernoulli equation (HEC-RAS) are compared here with the field measurements in terms of free surface profile and velocity distributions. The DCM and SCM were used by applying the HEC-RAS model, as well as CES in backwater computation mode. Under steady conditions, the one dimensional hydraulic equations to be solved are the conservation of mass:

$$\frac{\partial Q}{\partial x} = 0 \tag{1}$$

and the conservation of energy:

$$\frac{\partial\left(Q^2/A\right)}{\partial x} + gA\frac{\partial H}{\partial x} + gA\left(S_o - S_f\right) = 0 \tag{2}$$

where A = cross-sectional area normal to the flow; Q = discharge; g = acceleration due to gravity; H = elevation of the water surface above a specified datum, also called stage; S_o = bed slope; S_f = energy slope; x = longitudinal coordinate. Equations (1) and (2) are solved using the well known four-point implicit box finite difference scheme (USACE, 2008). HEC-RAS solves these equations using the standard step method as follows:

$$Y_2 + Z_2 + \frac{\alpha_2 V_2^2}{2g} = Y_1 + Z_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \tag{3}$$

where Y_i = depth of water at cross-sections; Z_i = elevation of the bed; V_i = average velocities at cross-sections; α_i = velocity weighting coefficients; h_e = energy head loss. The energy head loss can be calculated multiplying the length between the cross-sections times the friction slope, S_f. HEC-RAS uses two methods for computing the value of S_f, depending on whether the cross-section is treated as a unique compound section (SCM) or it is divided in sub-sections (DCM). The equations for the SCM are:

$$S_f = \frac{Q}{K} \tag{4}$$

$$K = \frac{AR^{2/3}}{n} \tag{5}$$

where R = hydraulic radius of the whole section, K = hydraulic conveyance, and n = Manning's roughness coefficient for the whole section. The DCM divides the cross-section into a main channel and two lateral floodplains applying eq. (3) for each subdivision and calculating the friction slope separately:

$$S_f = \frac{Q_i}{K_i} \tag{6}$$

$$K_i = \frac{AR_i^{2/3}}{n_i} \tag{7}$$

where the subscript i differentiate the three subsections. The total $K = \Sigma\, K_i$ and the total $Q = \Sigma\, Q_i$. HEC-RAS software implements the *Flow Distribution Option* in order to compute the lateral velocity distribution V_{di} by dividing the cross-section into a number of slices and then calculating the V_{di} as:

$$V_{di} = \frac{Q_{si}}{A_{si}} \tag{8}$$

where A_{si} = cross-sectional area for each slice; Q_{si} = discharge for each of the slice.

4.2 *CES quasi-2D Model*

The Environment Agency's CES model is based on the LDM (Wark et al, 1988; Shiono and Knight, 1991, Ervine et al, 2000), and it combines the continuity and momentum depth-averaged equations of motion for steady conditions and in the stream-wise component. The general equation of the model for a straight river (sinuosity equal 1.0) is obtained:

$$gS_oY - \frac{f}{8}q^2\sqrt{1+S_y^2} + \frac{d}{dy}\left[\lambda Y\sqrt{\frac{f}{8}}q\frac{\partial(q/Y)}{\partial y}\right] = \Gamma \tag{9}$$

where f = Darcy's friction factor; q = streamwise unit flow rate ($=Y{\cdot}U_d$); U_d = depth-averaged velocity; S_y = lateral bed slope; λ = non-dimensional Boussinesq eddy viscosity; y = lateral horizontal coordinate; and Γ = secondary flow parameter. The first term in eq. (3) is the hydrostatic pressure, the second is the boundary friction term, the third is the turbulence due to lateral shear stress and the last term in the right side represents the secondary circulations. The recommended values for the different variables and the Finite Element Code for solution of Equation 3 can be found in Defra/EA 2003. Once the velocity in each slice, U_d, is obtained, the total discharge, Q_t, in the cross section can be calculated as sum of unit discharges as:

$$Q_t = q(y_i - y_{i-1}) \tag{10}$$

where y_i and y_{i-1} are the horizontal coordinates in transverse direction for both sides of the slice.

4.3 *SRH2D Model*

The two dimensional depth-averaged model SRH-2D is a free-use available numerical code developed by Yong G. Lai, from U.S. Bureau of Reclamation (Lai, 2010). The code is based on the finite-volume approach and it can be assumed that provides an acceptable solution of the 2D equations in a variety of river flows (Lai, 2000; and Lai et al, 2006). The model solves the shallow water equations of flow:

Continuity equation:

$$\frac{\partial H}{\partial t} + \frac{\partial(HU_d)}{\partial x} + \frac{\partial(HV_d)}{\partial y} = 0 \tag{11}$$

Momentum equations in x and y :

$$\begin{aligned}&\frac{\partial(HU_d)}{\partial t} + \frac{\partial(HU_d^2)}{\partial x} + \frac{\partial(HU_dV_d)}{\partial y} = \\ &= \frac{\partial(H\tau_{xx})}{\partial x} + \frac{\partial(H\tau_{xy})}{\partial y} - gH\left(\frac{\partial H}{\partial x} + \frac{\partial z_b}{\partial x}\right) - \frac{\tau_{xb}}{\rho}\end{aligned} \tag{12}$$

$$\begin{aligned}&\frac{\partial(HV_d)}{\partial t} + \frac{\partial(HU_dV_d)}{\partial x} + \frac{\partial(HV_d^2)}{\partial y} = \\ &= \frac{\partial(H\tau_{xy})}{\partial x} + \frac{\partial(H\tau_{yy})}{\partial y} - gH\left(\frac{\partial H}{\partial y} + \frac{\partial z_b}{\partial y}\right) - \frac{\tau_{yb}}{\rho}\end{aligned} \tag{13}$$

where, x and y are horizontal Cartesian coordinates; z_b is bed elevation, t is time; H is water depth; U_d and V_b are depth-averaged velocity components in x and y directions, respectively, τ_{xx} , τ_{xy} and τ_{yy} are depth-averaged stresses due to turbulence as well as dispersion, ρ is the water density, and τ_{xb} , τ_{yb} , are the bed shear stresses. These bed stresses are obtained using the Manning's resistance equation as follows:

$$\tau_x^b = \rho C_f U_d \left(U_d^2 + V_d^2\right)^{1/2}\left[1 + \left(\frac{\partial z_b}{\partial x}\right)^2 + \left(\frac{\partial z_b}{\partial y}\right)^2\right]^{1/2} \tag{14a}$$

$$\tau_y^b = \rho C_f V_d \left(U_d^2 + V_d^2\right)^{1/2} \left[1 + \left(\frac{\partial z_b}{\partial x}\right)^2 + \left(\frac{\partial z_b}{\partial y}\right)^2\right]^{1/2} \quad (14b)$$

$$C_f = \frac{gn^2}{H^3} \quad (14c)$$

where C_f is a friction coefficient that is mainly depending on *n*, the Manning's roughness coefficient. The turbulence stresses are computed with Boussinesq equation as:

$$\begin{aligned} \tau_{xx} &= \rho(\upsilon + \upsilon_t)\left(\frac{\partial U_d}{\partial x} + \frac{\partial U_d}{\partial x}\right) - \frac{2}{3}k \\ \tau_{xy} = \tau_{yx} &= \rho(\upsilon + \upsilon_t)\left(\frac{\partial U_d}{\partial y} + \frac{\partial V_d}{\partial x}\right) \\ \tau_{yy} &= \rho(\upsilon + \upsilon_t)\left(\frac{\partial V_d}{\partial y} + \frac{\partial V_d}{\partial y}\right) - \frac{2}{3}k \end{aligned} \quad (15)$$

where υ is kinematic viscosity of water and υ_t is eddy viscosity. The eddy viscosity is calculated with the k-ε turbulence model (Rodi 1993), and the eddy viscosity is calculated as:

$$\upsilon_t = C_\mu \frac{k^2}{\varepsilon} \quad (10)$$

with two additional equations for the turbulent kinetic energy, *k*, and its dissipation rate, ε. Launder and Spalding (1974) added two transport equations to solve the new two unknown variables.

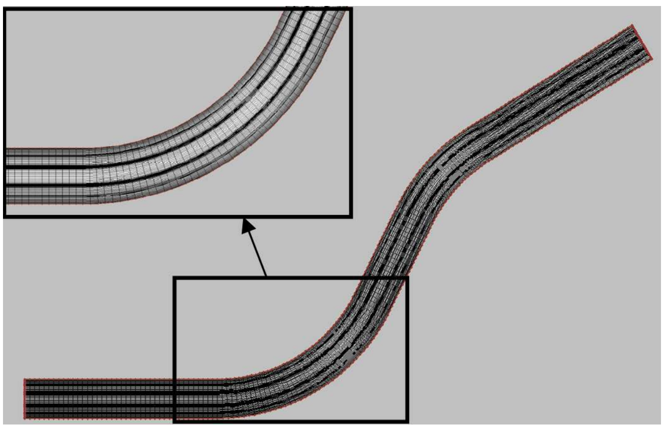

Figure 5. River Main, mesh discretization (15680 elements). The mesh is denser in the main channel banks, inclined bed, and less dense where the bed is flat.

The numerical solution of the SRH2D equations is implemented in a finite-volume method with quadrilateral elements. The standard conjugate gradient solver with ILU preconditioning is used (Lai 2000) for spatial integration in an iterative process. The computational domain is discretized in 15939 nodes and 15680 quadrilateral elements, 99 in cross-stream direction and 160 in streamwise, as it is shown in Fig. 5.

The boundary conditions are total discharge at the upstream section and a unique mean water level for the downstream cross-section. SRH2D calculates a distribution of the velocity along the upstream condition in such a way that the total discharge is satisfied. The approach used in this work is the conveyance distribution, which at the inlet is distributed across the upstream section following the conveyance proportion, eq. (7), so then the velocity in each element is proportional to depth and inversely proportional to Manning's *n*. This approach overestimates velocities in the main channel and underestimates them in the floodplain. These boundary conditions are the same than the 1D model, so the differences in results are only dependent on modelling equations.

5 MODELLING APPROACH

The first step in a river modelling work once the topography and geometry is defined is to identify the hydraulic variables involved. The most important one is the hydraulic resistance to flow, defined in terms of Manning's roughness coefficient in this work. In order to reduce the uncertainty explained in previous sections about the Manning's *n* value, this is calibrated with the 800 m longitudinal water level profile in the 10.5 m3/s inbank discharge. The HEC-RAS 1D model is used for iterating different Manning's *n* and to obtain the bankfull *n* by fitting the computed water profiles with the field profiles. Figure 6 shows the computed profile obtained with a mean Manning's *n* = 0.041 in the main channel, which is the roughness that best fits the field data. According to the variation of roughness with depth, the Manning's *n* in the bed should be 0.045 and 0.030 in the banks. These values will be used as the main channel roughness coefficients in the overbank discharges.

The water levels obtained with HEC-RAS for the two overbank discharges (20.1 and 51.3 m3/s) are shown in Fig. 7. The bank stations are located on the top of the main channel (DCM) or on the top of the floodplain walls (SCM) and two separated solutions are obtained. The results illustrate the main differences between both methods. The DCM gives lower water levels than the SCM for the same discharge. In general the water levels measured in the field works are found between the two numerical solutions (DCM and SCM). Some small discrepancies appear, probably due to roughness variation with depth and/or local changes in slope or section area.

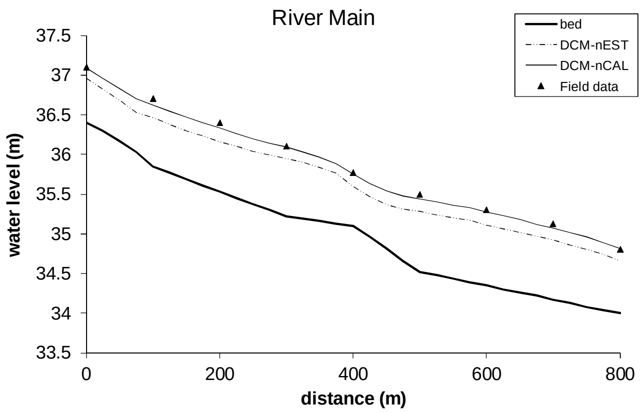

Figure 6. Field (9 full dots) and computed water surface profiles by using HEC-RAS with the estimated (DCM-nEST) and calibrated Manning coefficients (Manual) for the inbank flow.

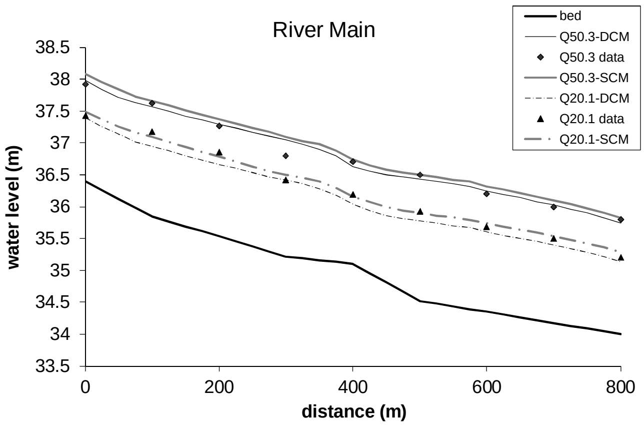

Figure 7. Computed water surface profiles by using HEC-RAS (DCM and SCM) for the two overbank discharges. Comparison with field data (Myers and Lyness, 1994).

Previous studies in compound channel flows demonstrated that DCM and SCM are not providing good results under uniform flow conditions. In this paper, Fig. 7 demonstrates that for gradually variable flow conditions DCM and SCM maintain discrepancies in water levels with real data and give different results. In terms of velocity distribution across the section, the SCM gives a uniform velocity for the whole section and the DCM is not providing a real distribution. For overbank flow the velocities given by HEC-RAS are different to the real distribution, especially in the main channel. Fig. 8 shows the velocity distribution obtained with HEC-RAS for the inbank flow Q10.5 and for the overbank flow Q51.3, together with the values measured for Q51.3. The model overestimates velocities in main river banks.

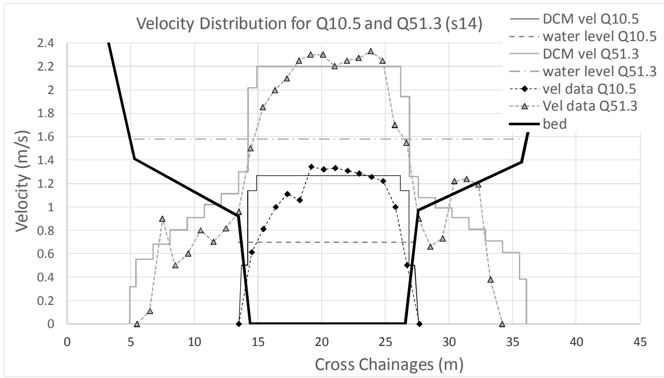

Figure 8. Field measured and 1D computed velocities in section s14 for inbank and overbank discharges, Q10.5 and Q51.3.

In order to understand the flow behaviour in this gradually varied flow river, the SRH2D was applied to the computational domain in Figure 5. The way the 2D model estimates total energy is a combination of bed friction (through a roughness coefficient) and turbulence stresses (through a dissipation coefficient). This is an important advantage with respect to 1D modelling that only includes bed friction losses and the velocity distribution only depends on water depth and Manning's coefficient. The results obtained with SRH2D model are shown in Figs 9 and 10. The water level profiles obtained with 2D modelling (k-ε turbulence model) are lower than those obtained with the DCM for all the discharges. The Manning's coefficients are the same in both models, as well as the boundary conditions. The difference in water levels between 2D and 1D solution are smaller for overbank flows than for the inbank one. This result confirms the conclusions by Moreta (2014) who demonstrated that for uniform flow in straight compound channels, 2D modelling gives lower water levels than 1D modelling if the roughness and boundary conditions are the same.

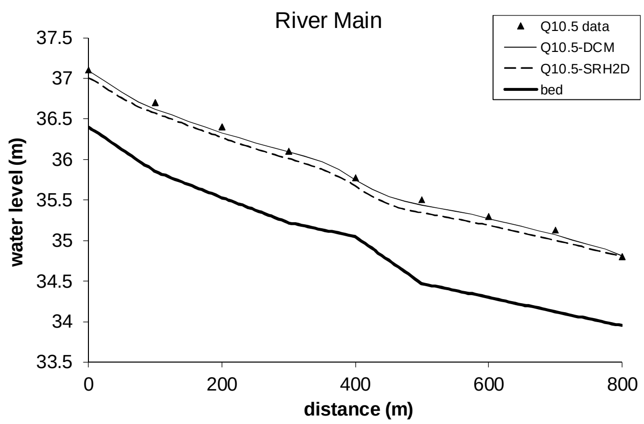

Figure 9. Water level field data and 2D computed water surface (*W.S.*) compared with 1D values (DCM). Inbank discharge Q10.5.

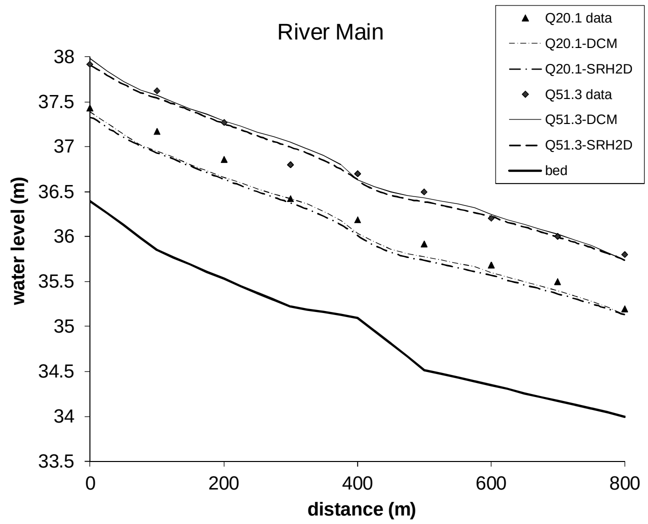

Figure 10. Water level field data and 2D computed water surface (*W.S.*) compared with 1D values (DCM). Overbank discharges Q20.1 and Q51.3.

However 2D modelling has some advantages over 1D modelling. First, the changes in main channel and floodplain sinuosity are taken into account, second, it considers internal energy losses due to flow turbulence and third, consequently the velocity direction and distribution must be better simulated. In Figures 11 and 12 the velocity distribution obtained with 1D (DCM) and 2D models for the inbank, Q10.5, and overbank, Q51.3, discharges are compared with field measurements. The velocities given by SRH2D improve slightly the velocities obtained by DCM.

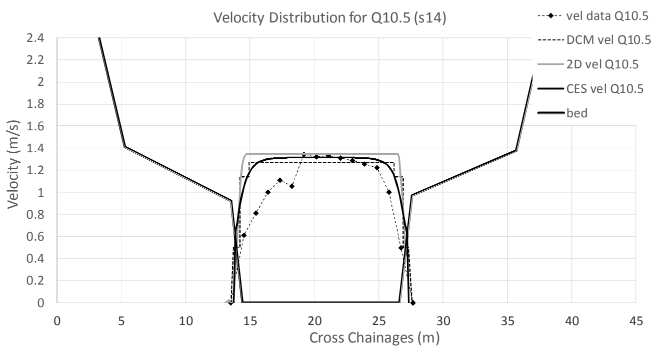

Figure 11. Velocities of field data (Martin and Myers, 1991) and computed with 1D (DCM), 2D (SRH) and CES for overbank discharge Q51.3.

6 IMPROVING 1D MODELLING

In order to improve 1D modelling (with DCM), the results obtained with CES are discussed in this paragraph. The first step is that for straight river channels with moderate roughened floodplains, the water profiles obtained by 1D model are better than the 2D model. However, the distribution of depth-averaged velocity can be obviously improved. The CES is applied to section 14, using the same bed slope and Manning's coefficient of roughness than in 1D modelling. CES precise a water level to estimate the velocity distribution and total discharge. The water depth used for estimating the velocity is that obtained from the 1D modelling. Figures 11 and 12 show that the velocity distribution obtained with CES fit better with the data than the distribution given by 2D model.

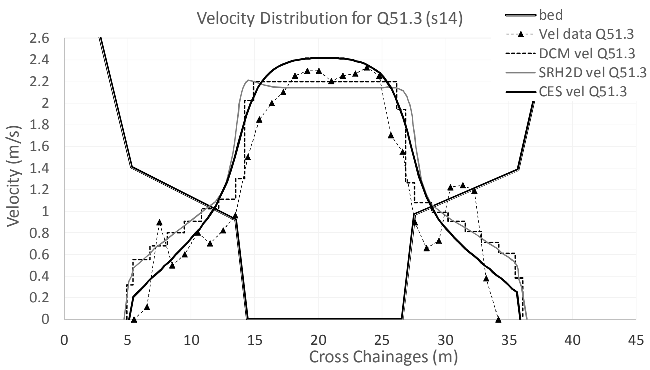

Figure 12. Velocities of field data (Martin and Myers, 1991) and computed with 1D (DCM), 2D (SRH) and CES for overbank discharge Q51.3.

7 CONCLUSIONS

The numerical analysis of this work is based on previously published field data and illustrates some of the problems that affect common 1D numerical model in reproducing overbank flow. HEC-RAS model is not able to yield an accurate velocity distribution across the section of a straight compound channel. Secondly, the comparison between the field data and the SRH2D model shows the need to take into account that the Manning's coefficients valid for 1D modelling are not enough accurate for 2D simulations. Therefore, some uncertainties rising from the use of 2D models can provide uncertain results respect to better predictable estimations obtained by 1D modelling.

The analysis and comparison of flow velocities measured in field works and computed by numerical models has shown that the prediction of accurate velocity distributions in compound channel flow is a major challenge in numerical modelling. Typical 2D finite volume codes based on k-ε turbulence model trend to under predict main channel and floodplain interaction. These 2D models slightly improve the depth-averaged velocities obtained with 1D model for the straight river case analysed herein. In order to better simulate velocities, the CES based on Lateral Distribution Method is proposed for comparison. The CES gives a better representation of momentum interaction between main channel and floodplains and of the velocity distribution across the section. This methodology has been contrasted with field river data under gradually varied conditions, confirming the results of some previously published works on the topic under differente conditions (Weber and Menendez, 2004, and Vionnet et al, 2004).

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