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Flow structure in a compound channel with smooth and rough floodplains

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Abstract: Compound channels are a common configuration of rivers. During extreme events of floods, the momentum transfer due to the velocity difference between the main channel and the floodplains flows generates a complex 3D flow. Accurate estimation of channel conveyance remains a difficult issue. Although several studies have being carried out in the past, a new experimental study aims revisiting some of the previous experiments. Four different flow conditions were tested, corresponding to uniform flows for relative depths (ratio of the water depths in the floodplain and in the main channel) approximately equal to 0.15 and 0.3, for smooth and rough floodplains. The influence of the relative depth and the floodplain roughness is assessed and the flow characteristics are presented. It includes the lateral distributions of streamwise velocity and Reynolds stresses in the horizontal plane. The accuracy of several methods to predict the total cross-section discharge is evaluated.

Key Words: Compound channels, rough boundary, smooth boundary, Reynolds shear stress, flow structure

1. INTRODUCTION

During floods the main channel of rivers may not be enough to convey the total discharge and a compound channel configuration can occur. In these cases, the flow submerges the surrounding fields, called the floodplains. The difference in water depth between the main channel and the floodplains leads to a difference in the streamwise velocity between these subsections. The faster flow in the main channel interacts with the slower flow in the floodplains generating a mixing layer near the interface (van Prooijen et al. 2005). This mixing region reduces the discharge capacity when compared with independent cross sections.

In many cases the floodplains are covered by vegetation, increasing the bottom roughness and the overall resistance. This difference leads to an increase of the velocity gradient between main channel and floodplain flows. Strong lateral shear layers between these regions are observed (Tang and Knight 2009).

Water depths in single channels are accurately estimated since the method proposed by Antoine de Chézy (Myers 1978). This is not the case for compound channels, because of the mixing layer in the interface which creates a 3D flow structure (Shiono and Knight 1991).

The traditional method to study the flood inundation is based in an old approach that simply divides the total cross section with vertical divisions in the interface of the main channel and the floodplains (Chow 1959). New 1D approaches can take into account the interaction between the flows in each subsection. The 2D and 3D methods include some of the characteristics of compound channels. In engineering, due to the amount of data required and the processing time, 1D methods are often preferred. Still, the momentum transfer should be taken into account in 1D modelling (Bousmar and Zech 1999).

Since Sellin (1964) presented the first evidences of the flow characteristics in compound channels that there have been attempts to model it. Knight and Shiono (1996) referred the difficulty of the developed formulas to be applied universally as, in many cases, they had been set based on a reduced amount of data.

This paper intends to improve the knowledge of the flow in compound channels. Firstly, the flow structure in a compound channel is characterized experimentally for smooth and rough floodplains. For each case two different water depths are presented. Secondly, the accuracy of several 1D methods, available in the literature, is assessed for the four flow conditions.

2. ONE DIMENSIONAL METHODS

Modeling the flow in a compound channel as a simple channel by applying a flow resistance formula does not take into account the sub-section velocity difference. Chow (1959) suggested the division of the channel in subsections where velocity and roughness could be considered as uniform. This method, called the Divided Channel Method, is still widely used in commercial models as HEC-RAS (Brunner 2008), ISIS (Knight 2001), SOBEK and Mike 11 (Huthoff et al. 2008).

As pointed out in Knight (2001) this treatment of a compound channel assumes that there is no interaction between the subdivided areas despite the existence of mean velocity discontinuities at the assumed internal boundaries. Therefore the simple division of the channel in subsections is not appropriate for modelling the discharge in compound channels (Knight and Shiono 1996).

Different methods had been proposed with the attempt to model the interaction processes that occur in this type of flows, including the momentum transfer.

According to Knight (2001), these methods can be divided into 5 groups: i) methods that change the sub-area wetted perimeters; ii) methods that make discharge adjustments (with experimental data, for example); iii) methods that include apparent shear stresses on the sub-area division lines; iv) methods where the lines are located at zero shear stress; v) methods that combine different divisions of the channel.

In this work, six methods were used to model the flow in the compound channel. Firstly, the traditional method Divided Channel Method (DCM) has been applied. From the groups presented before, the methods implemented were the Coherence Method (CM) and the Debord Method (DM) from the group ii), the Exchange Discharge Method (EDM) and the Interacting Divided Channel Method (IDCM) from the group iii) and the Weighted Divided Channel Method (WDCM) from the group v).

A simple explanation of the calculation of the stage-discharge curves by each method is presented herein.

2.1 Divided Channel Method (DCM)

This method proposes the division of the channel in three sub-sections, namely the main channel and the lateral floodplains. The typical division is through vertical lines, where the total flow is given by the sum of sub-section discharges:

$$Q = \sum_i Q_i = \sum_i K_i R_i^{2/3} A_i S_0^{1/2} \tag{1}$$

in which Q stands for the discharge; K for the subsection roughness coefficient; R for the hydraulic radius; A for the cross section area and S_0 for the slope of the channel. Index i indicates each subsection.

2.2 Coherence Method (CM)

The Coherence Method was developed by Ackers (1993) and it improves the results of the DCM. This method uses two empirical coefficients for the adjustment of the sub-section discharges.

The coherence (*COH*) is the relationship between the discharge obtained by the Eq. (1) but assuming only one section (Single Discharge Method - SCM, average roughness coefficient and velocity for the whole cross section) and the DCM:

$$COH = \frac{Q^{SCM}}{Q^{DCM}} \tag{2}$$

The closest to 1 is this coefficient, the most appropriate is to treat the channel as a single one. When this coefficient is significantly less than 1 it is necessary to apply a different coefficient, called *DISADF* in order to correct the discharge in each subsection. (Fig. 1 presents an example of one flow condition with the division of the flow into 4 regions according to its relative depth (floodplain/main channel water depth ratio).

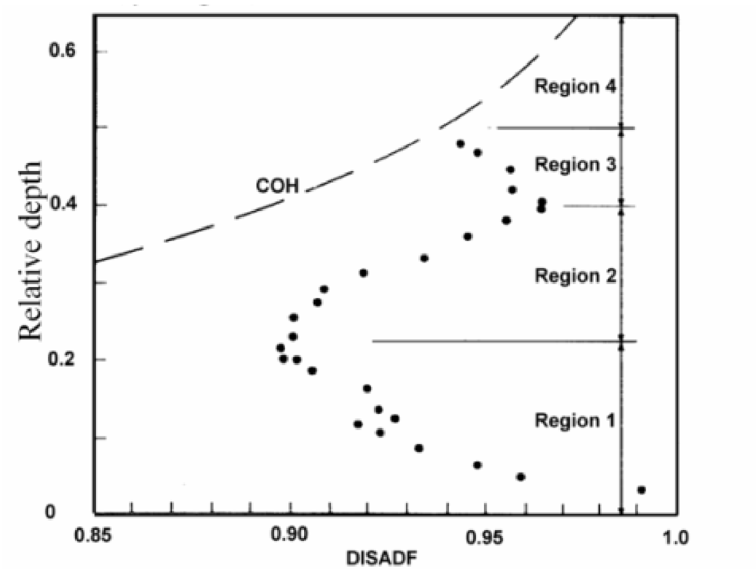

Figure 1. DISADF coefficient.

Ackers (1993) presented the formulas for computing the *DISADF* in each flow region. The discharge is then obtained by the following equations.

$$Q = Q^{DCM} - DISDEF \quad \text{for flow region 1} \tag{3}$$

$$Q = Q^{DCM} \times DISADF \quad \text{for flow regions 2 to 4} \tag{4}$$

in which *DISDEF* is a factor called discharge deficit which calculation procedure can be found, for example, in Wark et al. (1994).

2.3 Debord Method (DM)

The Debord Method proposes a correction of the DCM results based on experimental results conducted with 16 different configurations (Nicollet and Uan 1979). In those tests, the compound channel flow was compared with the flow in the independent sections (vertical separations were placed in the interface). The authors concluded that the most important parameter was the roughness ratio between subsections. The discharges can be computed with the Eqs. (5) and (6).

$$Q_{mc} = \varphi K_{mc} R_{mc}^{2/3} A_{mc} S_0^{1/2} \tag{5}$$

$$Q_{fp} = \sqrt{1 + \frac{A_{mc}}{A_{fp}}\left(1 - \varphi^2\right)} K_{fp} R_{fp}^{2/3} A_{fp} S_0^{1/2} \tag{6}$$

in which the subscripts *mc* and *fp* stand for main channel and floodplain, respectively and φ stands for the experimental coefficient given by:

$$\varphi = \frac{1}{2}\left[\left(1 - \varphi_0\right)\cos\left(\frac{\pi \; R_{fp}/R_{mc}}{0{,}3}\right) + \left(1 + \varphi_0\right)\right] \quad \text{for } 0 < R_{fp}/R_{mc} \leq 0{,}3 \tag{7}$$

$$\varphi = \varphi_0 = 0{,}9\left(K_{mc}/K_{fp}\right)^{1/6} \qquad \text{for } R_{fp}/R_{mc} > 0{,}3 \tag{8}$$

2.4 Exchange Discharge Method (EDM)

This method takes into account the concept of the apparent shear stress. The basis of this method is the transverse integration of the equation of momentum conservation. After some simplifications and mathematical operations this equation could be written for the main channel and for the floodplains as showed in Eq. 9 and 10, respectively.

$$\rho g A_{mc} S_o + \left(h_{\text{int},rig}\tau_{\text{int},rig} + h_{\text{int},lef}\tau_{\text{int},lef}\right) - \tau_o P_{mc} = 0 \qquad \text{for the main channel} \tag{9}$$

$$\rho g A_{fp} S_o - h_{\text{int}}\tau_{\text{int}} - \tau_o P_{fp} = 0 \qquad \text{for the floodplain} \tag{10}$$

in which ρ stands for the density of water; g for the acceleration due to gravity; h_{int} for the interface height; τ_{int} for the apparent shear stress in the main channel and floodplain interface; τ_0 for the bottom shear stress and P for the wet perimeter. Subscripts "*rig*" and "*lef*" stand for right and left, respectively.

The mixing layer model by Smart (1992) is the base of EDM to model the momentum transfer due to turbulence. Therefore the Eq. (11) is used for calculating the apparent shear stress (Bousmar and Zech 1999).

$$\tau_{\text{int}} = \frac{1}{2}\psi\rho\left(U_{mc} - U_{fp}\right)^2 \tag{11}$$

in which ψ stands for an experimental parameter and U stands for the sub-section average velocity. EDM also models the momentum transfer associated with the geometry like converging main channels, which is out of the scope of this work.

2.5 Interacting Divided Channel Method (IDCM)

Huthoff et al. (2008) developed this method based on the apparent shear stress concept (Eq. 9 and 10). The authors used the formulation of van Prooijen et al. (2005) in order to model the momentum transfer in the interface, obtaining the Eq. (12).

$$\tau_{\text{int}} = \frac{1}{2}\gamma\rho\left(U_{mc}^2 - U_{fp}^2\right) \tag{12}$$

in which γ corresponds to a coefficient obtained from experimental results collected in the literature.

2.6 Weighted Divided Channel Method (WDCM)

The Weighted Divided Channel Method was developed by Lambert and Myers (1998) and it is based on the analysis of the velocity distributions in the main channel and in the floodplains. This method corrects the DCM results by weighting the velocities obtained with vertical and horizontal divisions between the subsections (Eqs. 13 and 14).

$$U_{mc} = \xi U_{mc}^{DCM-V} + (1-\xi) U_{mc}^{DCM-H} \tag{13}$$

$$U_{fp} = \xi U_{fp}^{DCM-V} + (1-\xi) U_{fp}^{DCM-H} \tag{14}$$

in which superscripts *DCM-V* and *DCM-H* stand for the results of DCM with vertical and horizontal divisions, respectively and ξ for the weighting coefficient for the WDCM (see Lambert and Myers, 1998 for further details).

3. EXPERIMENTAL DATA ACQUISITION

3.1 Experimental setup and equipment

The experiments were conducted in a compound channel located in the National Laboratory for Civil Engineering, in Lisbon. The experimental facility consists in a 10 m long, 0.4 m wide and 0.1 m high main channel in the centre of two symmetrical 0.7 m wide floodplains. The longitudinal slope of the flume is 0.0011 m/m. The transition between the subsections is made by banks with 45º slope. Fig. 2 shows a schematic top view and cross-section of the flume.

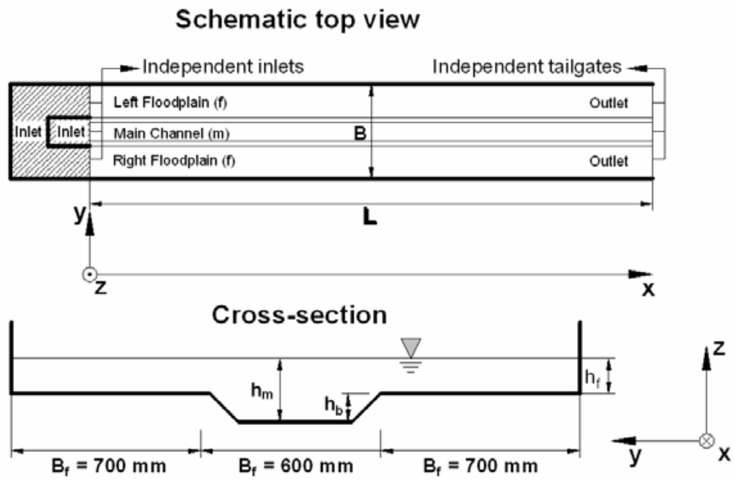

Figure 2. Schematic top view and cross-section.

The channel bottom is made of polished concrete. The rough experiments were done with the floodplains covered by commercial synthetic grass. The roughness of the two channel bottoms (polished concrete and synthetic grass) were estimated by the results of experiments with flow in a single channel. The Manning roughness coefficients for the polished concrete and for the synthetic grass are 0.0095 s.m$^{-1/3}$ and 0.017 s.m$^{-1/3}$, respectively. The equivalent sand roughness k_s of the synthetic grass is 0.00617 m and the average value of $u^* k_s/\upsilon$ (where u^* is the friction velocity in the centre of the channel and υ is the cinematic viscosity) is equal to 178 corresponding to a hydraulically rough boundary ($u^* k_s/\upsilon$>70). The equivalent values for the polished concrete are k_s = 0.15 mm and $u^* k_s/\upsilon$ = 4.6 corresponding to a hydraulically smooth boundary.

Following the recommendations of Bousmar et al. (2005), separate inlets for the main channel and for the floodplains were installed in order to avoid the mass transfer between subsections. These discharges were monitored by two flowmeters and controlled by two different valves. Honeycomb diffusers and polystyrene plates were located at the beginning of the flume to stabilize the flow.

The flow regime is subcritical and the water depths were controlled by three independent tailgates located at the downstream end of the channel.

Water levels were measured with three point gauges. Two of them were fixed at the upstream and at the downstream sections of the flume and the other one was place on a movable trolley allowing the measurement of the water depth in the entire channel. Streamwise velocity measurements were made using a Prandtl tube with a 3.2 mm external diameter. The difference between static and dynamic pressures was measured with a differential pressure transducer.

The 2D and 3D velocity components were measured by an Acoustic Doppler Velocimeter, namely a side looking vectrino (herein called ADV vectrino). The acquisition time was 180 s for each measurement, with a sampling frequency of 100 Hz. The sampling volume is a 7 mm long and 6 mm diameter cylinder. Taking into account the results obtained by the Prandtl tube measurements, namely the symmetry of the discharge, the measurements with the ADV vectrino were done only in one half of the cross-section.

3.2 Experimental procedure

For the presented compound channel, the exact distribution of the discharge in the main channel and the floodplains was not known a priori. The procedure used to obtain a uniform distribution started with the distribution given by the Weighted Divided Channel Method (Lambert and Myers, 1998). With this first discharge distribution, the water levels were controlled with the tailgates in order to achieve a uniform water depth along the channel. When the water depth was constant, the discharge distribution at the measurement section (7.5 m from upstream) was compared with the upstream distribution. If the upstream and downstream discharge distributions match within less than 0.1 l/s, the uniform regime was considered to have been achieved. Otherwise the measured discharge distribution was imposed upstream and the procedure was repeated in an iterative process.

The streamwise velocities were measured with the Prandtl tube in 45 verticals with 5 or 6 points each for the floodplains and main channel, respectively (Fig. 3).

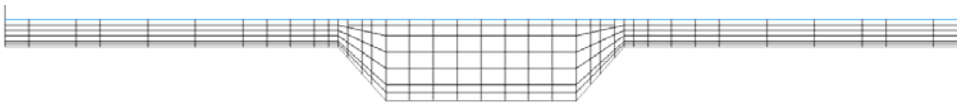

Figure 3. Mesh for the velocity measurements.

The measurements of the 2D and 3D velocity components were performed for the uniform flow in the same verticals as presented in Figure 3 with 3 points in the floodplains and 7 points in the main channel. For the present work, four flow conditions have been adopted, varying the relative depth and the floodplain roughness. Table 1 presents these experimental conditions (FP and MC stand for floodplain and main channel, respectively).

Table 1. Experimental conditions.

Flow reference	MC depth, h_m (m)	Relative depth, h_r (–)	FP bottom	Discharge (l/s) MC	FP	Total
hr015s	0.1172	0.15	Polished concrete	38.2	6.6	44.8
hr03s	0.1422	0.3	Polished concrete	54.2	26.4	80.6
hr015r	0.1192	0.15	Synthetic grass	35.1	3.7	38.8
hr03r	0.1450	0.3	Synthetic grass	42.3	16.6	58.9

4. FLOW AND TURBULENCE STRUCTURE

4.1 Streamwise velocity

The streamwise velocity was measured with the Prandtl tube in the positions presented in the mesh of Fig. 3. In Fig. 4 the isolines of the normalized streamwise velocity (u/u_m, where u_m represented the cross section average velocity) are presented for the two relative depths ($h_r = 0.15$ and $h_r = 0.3$) and for the case of smooth and rough floodplains.

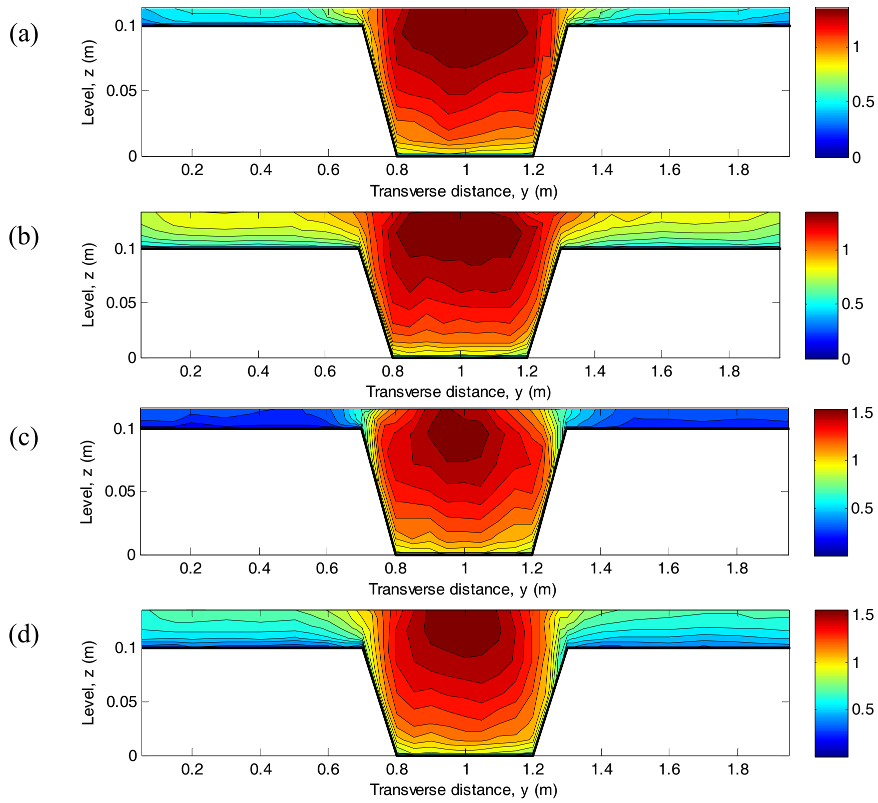

Figure 4. Normalized streamwise velocity (u/u_m) (a) hr015s; (b) hr03s; (c) hr015r and (d) hr03r.

All the plots in Fig. 4 show the influence of the floodplain in the main channel flow namely the decrease of the velocity in the vicinity of the floodplains. The opposite occurs in the floodplain flow where an increase of the velocity is observed due to the presence of the main channel flow.

Two different comparisons can be made concerning the results presented in Fig. 4: i) the influence of the relative depth and ii) the influence of the floodplains roughness.

The increase of the water depth from $h_r = 0.15$ to $h_r = 0.3$ (comparing Fig. 4a with Fig. 4b and Fig. 4c with Fig. 4d) leads to a reduction of the interaction between these flows due to the decrease of the velocity gradient between the subsections. Regarding the isovels, there is a slightly difference between the two relative depths. Nevertheless, the overall distribution is similar.

The influence of the floodplain roughness on the isovels distribution is more evident. A lateral shift of the maximum velocity zone is observed for both relative depths. In average the velocity gradient is higher for the rough floodplain boundary.

4.2 Reynolds stresses

The 3D velocity measurements, obtained with ADV vectrino, allow the calculation of turbulence intensities and Reynolds stresses.

Reynolds stresses were calculated by $\tau_{xy} = -\rho u'v'$, where u' and v' are the fluctuation velocities for the streamwise and spanwise directions, respectively.

In the Fig. 5 the isolines of the Reynolds stresses τ_{xy} are presented. They reveal the interaction between the flow in the floodplain and in the main channel. For all cases, the higher Reynolds stresses values are observed near the interface.

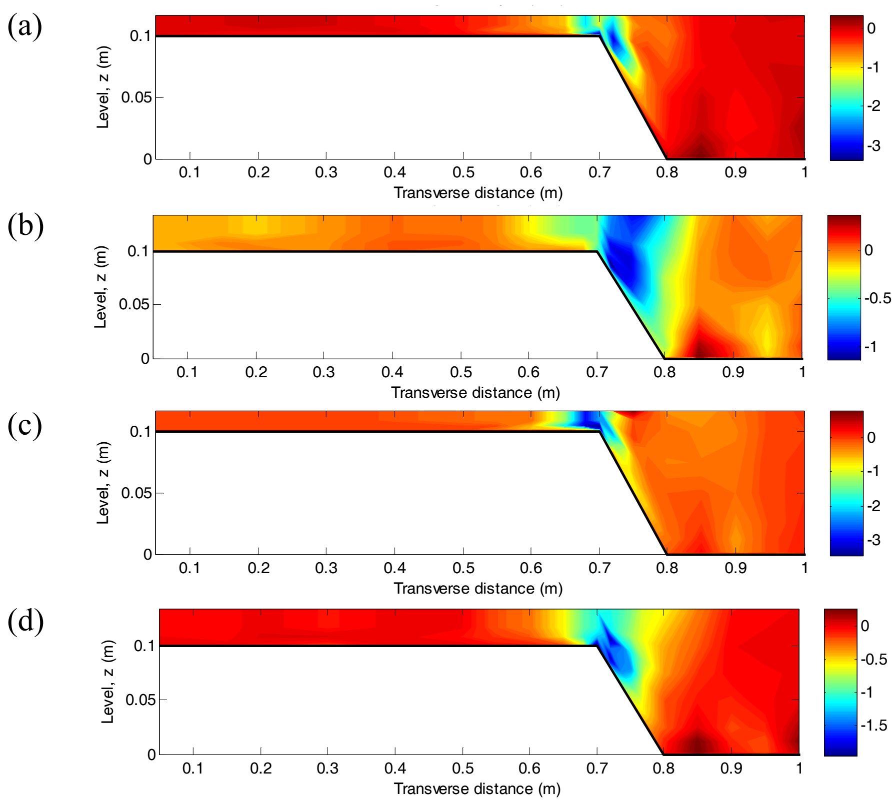

Figure 5. Reynolds stresses cross-section distribution (τ_{xy} in Pa). (a) hr015s; (b) hr03s; (c) hr015r and (d) hr03r.

The depth averaged Reynolds stresses are presented in the Fig. 6.

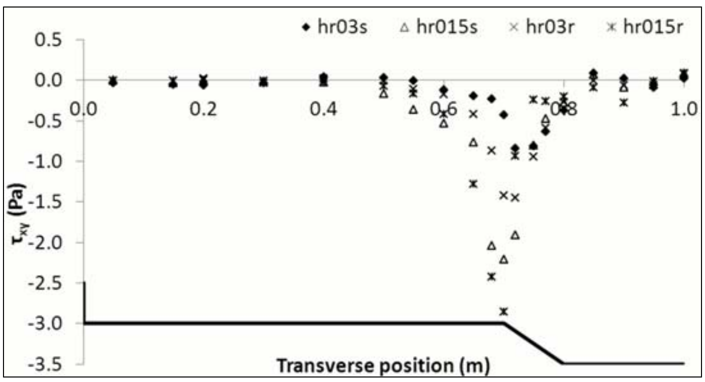

Figure 6. Depth averaged Reynolds stresses.

For all experiments the depth averaged Reynolds stresses are approximately zero for values of y < 0.5 and for y > 0.85. For both floodplain roughness, the values of the depth averaged Reynolds stresses near the interface increase from relative depth 0.15 to 0.3. The same occurs when smooth and rough floodplain results are compared. Along with the depth-averaged Reynolds stresses increase, a spreading of the shear layer towards the floodplain is observed.

5. ACCURACY OF THE 1D METHODS

As referred above, several 1D methods have been developed to deal with the complexity of the compound channel flows. In the present work, the accuracy of these methods is assessed by comparing their predictions with the experimental results (Table 1).

The assessment of the accuracy by each method is based on the calculation errors computed by Eq. (15).

$$\text{Error}\,(\%) = 100 \times \left(Q^{Measured} - Q^{Calculated}\right) / Q^{Measured} \tag{15}$$

in which $Q^{Measured}$ and $Q^{calculated}$ stands for the measured and calculated discharges, respectively.

The complete results of these errors are presented in Table 2.

Table 2. Errors obtained by applying the different 1D method.

Flow reference	DCM	CH	DM	EDM	IDCM	WDCM
hr015S	-7%	0%	-2%	9%	-1%	4%
hr03S	-5%	3%	0%	4%	1%	6%
hr015R	-21%	-9%	-8%	12%	-11%	-2%
hr03R	-32%	-12%	-15%	1%	-15%	9%

These results are presented graphically in Fig. 7.

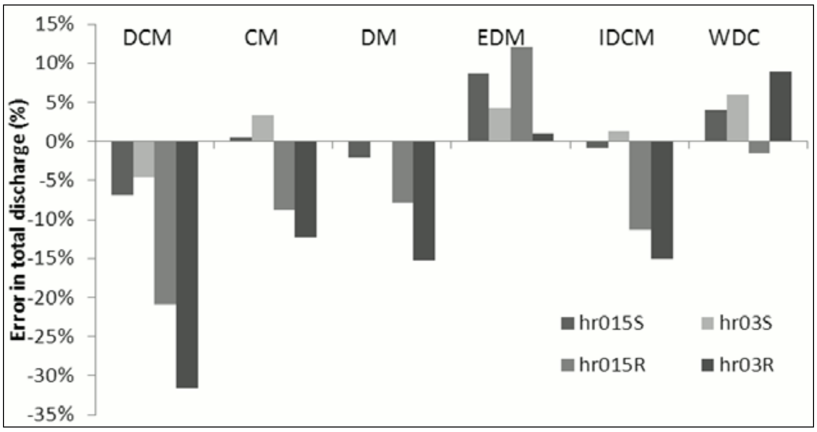

Figure 7. Error in the calculation of the discharges.

The DCM, assuming a simple division between the sub-sections of the entire channel, without considering the interaction between the subsections, leads to an over estimation of the discharge up to 7% when the channel has the same roughness along the entire perimeter. This over estimation is even bigger with rough floodplains with errors of up to 32%. The velocity reduction that the flow of floodplains causes in the main channel flow is not accounted in the main channel discharge. The errors decrease with the relative depth for smooth floodplain and increase with rough floodplains.

All alternative methods allowed the improvement of the DCM results for the total discharge calculation. The results of the smooth floodplains conditions reveal good agreement with the experiments. When the bottom has different roughness important discrepancies still occur.

6. CONCLUSIONS

The present work had two separate parts: an experimental study on the compound channel flow structure and the assessment of several one dimensional methods available in the literature.

The influence of the floodplain roughness on the streamwise velocities and on the Reynolds stresses is clear. The synthetic grass increases the floodplain and the main channel resistance to flow. The gradient of velocities increases when the floodplain is covered by the synthetic grass which leads to an increase of the Reynolds stresses.

Regarding the test of 1D methods, for smooth floodplains, good results on flow conveyance have been obtained by methods that take into account the momentum transfer. With the rough floodplains, the errors are relevant.

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