The influence of vegetation in compound channels under uniform flow

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Abstract: River floods are one of the most dangerous natural disasters and the study of compound channels is very important to understand the behavior of the flow. In this study experiments were made in a compound channel with vegetation in the floodplains, and rods and shrubs in the interface between the main channel and the floodplain. Velocities, Reynolds stresses and turbulence intensities were measured in the three directions of the flow, with an Acoustic Doppler Velocimeter (ADV). The riparian vegetation decreases the longitudinal velocity in the floodplains and increases in the main channel. The shrubs in the interface between the main channel and the floodplain cause very low velocities in that region. The lateral Reynolds stresses are generally zero along the section and have a negative peak in the interface that decreases when there are no partially submerged vegetation and increases in the other way. The roughness of the channel (synthetic grass) has a massive influence in the flow decreasing a lot the velocities on the bottom of the floodplain until approximately 12 mm of the bottom. The distribution of the boundary shear stress in the section was estimated by three methods: (i) Clauser's method; (ii) modified log-wake law; and (iii) turbulent kinetic energy (TKE). It was observed that the roughness in the floodplains decreases the boundary shear stress in the main channel and near the interface it is not possible to estimate correctly this parameter. The shrubs decrease slightly this variable in the floodplains in the interface.

Keywords: Compound channel, riparian vegetation, shear velocity, boundary shear stress, Clauser's method, modified log-wake law, turbulent kinetic energy

1. Introduction

Characterizing the flow in compound channels is very important to study flood events in low land rivers. There are many more flow mechanisms in compound channels than in single open channels. These mechanisms have been extensively studied by Sellin (1964), Posey (1967) and Zheleznyakov (1971) who found out that the common methods used to predict the flow discharge of the channels are not correct. After those studies, Knight and Hamed (1984) and Knight et al. (1994) focused on the interaction between the floodplain and the main channel flows including the distribution of the Reynolds stresses.

Often, floodplains are covered with vegetation, easily submerged by the flow during floods, which renders them significantly rougher than the main channel. Observations of river banks reveal that lined vegetation, growing along the edge of floodplain, is very common. Such vegetation may be trees or bushes of different kinds and may be spaced in different patterns. A single line of riparian vegetation may be used for bank stabilisation, to promote environmental diversity or landscape amenity purposes (Hubble et al., 2009).

The determination of the boundary shear stress in rough bottom is significantly difficult due to the uncertainty of the roughness layer height. The boundary shear stress is usually estimated through the appropriate analysis of the velocity profile data, through the shear velocity, or by measuring directly the boundary shear stress with appropriate equipment.

2. Background

2.1. Compound channels

It has been found that the structure of the flow in a compound channel includes peculiar features induced by the velocity gradient between the floodplain and the main channel flow. These mechanisms integrate the marked tridimensional features and turbulent flow structure in compound channels. Because of the mentioned velocity gradient, there exists a significant lateral momentum transfer between the main channel and the floodplain. In the bottom of the main channel, there are vortices with helicoidal axis called secondary currents. These vortices are produced by the anisotropy of the turbulence (Shiono and Knight, 1991). In this type of channels, there is also a mixing layer that stabilizes the velocity in the lateral direction of the channel. This layer controls the exchange of momentum and mass between the main channel and the floodplain. The distribution of the mean longitudinal velocity in the cross section increases from the floodplain to the main channel as does the boundary shear stress. The introduction of the vegetation in the channel increases the resistance in the bottom of the channel. That is why the boundary shear stress, \( \tau_B \), that can be related to the shear velocity, \( u_s \), by \( \tau_B = \rho u_s^2 \), where \( \rho \) is the water density, is important to the characterization of the channel flow.

In recent studies in the Flood Channel Facility
(FCF), described by Knight and Sellin (1987), it has been observed that the distribution of the lateral Reynolds stress is approximately zero except in the interface between the main channel and the floodplain. In that region, the lateral Reynolds stresses decrease significantly and that decrease is higher for lower relative depths. Yang et al. (2007) studied the structure of a trapezoidal compound channel for a relative depth of 0.7 for smooth and rough floodplains. It was observed that the roughness of the floodplain decreases the velocity and that the synthetic grass is the type of vegetation that decreases the velocity the most. With rough floodplains the velocity gradient between the main channel and floodplains is higher and the lateral Reynolds stresses decrease in the interface. The velocities in the center of the main channel are higher when there are rough floodplains.

The effects of rigid and partly submerged vegetation along the interface between the main channel and the floodplain were studied by Sun and Shiono (2009) and Terrier et al. (2010). In Mediterranean climates, like part of Portuguese climate, the wood strata of riparian vegetation is dominated by ash (Fraxinus angustifolia Vahl), alder (Alnus glutinosa), black poplar (Populus nigra L.) and willows (Salix atrocinerea) (Ferreira et al. 2005). Typical values of spacing ratio, L/D, L being the distance between elements and D the diameter of the foliage, between 8 and 16 are frequently presented in the literature (cf. Landcare Notes 1998, cited by Terrier et al. 2010). Shiono et al. (2009) presented a survey for spacing ratios in three rivers in Japan and found typical values around 12 to 16. Terrier (2010) found that the average tree diameter along the right bank of river Rhone in the area of Lyon was 0.5 m with an average spacing ratio of 10. Sun and Shiono (2009) studied the effect of cylinders located in the floodplain’s edge of a trapezoidal compound channel and observed that they affect the flow characteristics significantly. Without cylinders (or rods) in the interface, the normalized boundary shear stress (\(1 - \rho g h i / \tau_0\)) shows a peak that occurs in the rod area and increases as the water depth decreases (\(g\) – gravity acceleration; \(h\) – water depth; \(i\) – channel slope). The magnitude starts to decrease from the floodplain edge towards the floodplain wall and its sign becomes positive near the wall. This variation is mainly associated with momentum transfer from the main channel to the floodplain and with the secondary currents, as explained by Shiono and Knight (1991). For the rod cases, the boundary shear stress is always lower than \(\rho g h i / \tau_0\) for non-disturbed flow. The main differences between the two cases are near the rod and on the floodplain.

After Sun and Shiono (2009), Terrier et al. (2010) carried out experiments in the same flume but with brushes along the edge of the floodplain. They found out that with this type of foliage, increasing the vegetation density, leads to a decrease of flow discharge. They observed that the lateral distributions of boundary shear stress follow the distributions of the depth-averaged velocities. Near the shrub area, the boundary shear stress decreases. The authors observed also a significant difference between the case with shrubs in the floodplain edge and the case with no shrubs. The differences between relative depths are almost insignificant.

Sanjou and Nezu (2011) observed the decrease in the velocity of the flow near the cylinders, on the interface, and large scale longitudinal vortices in both the main channel and the floodplain. The authors found negative and positive lateral shear stresses appearing behind the cylinders. This effect is significantly different from what happens in compound channel flows without cylinders in the interface. It suggests that momentum exchanges between the main channel and the floodplain are suppressed by the cylinders.

This study was carried out after a sequence of studies already made in the same compound channel at the National Laboratory of Civil Engineering (LNEC), Lisbon, Portugal. The experiments were made for rough and smooth floodplains and with rods and shrubs in the interface between the main channel and the floodplain. Fernandes (2013) compiled and analyzed the results of all the experiments carried out earlier. The author obtained results for velocities, Reynolds shear stresses, turbulent intensities and boundary shear stress for different relative depths. In Table 1 and Table 2 the flow conditions are presented, measured under uniform flow. The measurements were made in a longitudinal position of the compound channel, where the boundary layer and mixing layer were completely developed. The boundary shear stress was measured only for smooth floodplains and for rough floodplains without riparian vegetation in the interface. The results corroborated the literature mentioned before almost entirely. The boundary shear stress has a decrease in the main channel and an increase in the edge of the floodplain near the interface in rough floodplains that confirms the significant influence of the strong Reynolds shear stress near the interface.

<table>
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<tr>
<th>Table 1 - Flow conditions without riparian vegetation</th>
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2 of 12
Because of the roughness of the bottom and the turbulence generated by the vertical elements in the interface, it was difficult to measure the boundary shear stress for the other cases. To complete the information, some methods were used in this study to estimate the lateral distribution of \( \tau_0 \).

### 2.2. Methods for estimating shear velocity or boundary shear stress

#### 2.2.1. Introduction

There are various methods to estimate the boundary shear stress. Most of those methods are based in the concept that there is a constant shear layer within the water column where shear stress only varies slightly from bottom stress \( \tau_0 \). Even though a constant shear layer does not exist in open-channel flow, these methods have been successfully applied in open-channel flow studies. Nezu and Nakagawa (1993) studied various methods for rough beds in open-channel flow.

Three types of methodologies were considered: (i) based on the velocity profile; (ii) based on the vertical Reynolds stress profile and (iii) based on the turbulence intensities. The estimates obtained by the methods can be compared to the boundary shear stress for 2D uniform flows, based in the momentum equation

\[
\tau_0 = \rho g h i
\]  

#### 2.2.2. Velocity profile

The velocity profile can be divided in two regions: (i) inner region and (ii) outer region. The inner region is also subdivided in three layers: (i) viscous layer, (ii) buffer layer and (iii) logarithmic layer that occupies most of the inner region (Nezu and Nakagawa 1993). The inner region is the region where the turbulent structure of the flow is directly influenced by \( \tau_0 \) and, in case of rough boundaries, by the equivalent sand roughness, \( k_s \). Sometimes the velocity profile is divided in a different way where the inner and outer region has an overlap, and that overlap region is the logarithmic region. The distribution of the velocity in the logarithmic region can be written as,

\[
\frac{u}{u_*} = \frac{1}{k} \ln \left( \frac{z u_*}{v} \right) + B
\]  

for smooth (equation 2) and rough (equation 3) bed, respectively. Here \( u_* \) is the shear velocity, \( k \) is the von Kármán constant (that is usually 0.41 (Nezu and Nakagawa, 1993)), \( z \) the vertical position, \( v \) is the fluid kinematic viscosity and \( B \) an integration constant. The values of \( B \) are normally \( 5.1 \pm 0.96 \) for smooth beds (Cardoso et al. 1989) and \( 8.5 \pm 0.2 \) for rough beds (Song, 2004).

For rough boundaries, two types of flows were proposed by Perry et al. (1969): (i) -k, where the flow interacts with the bed and (ii) \(-d\), where the flow does not interact with the water inside the roughness. The logarithmic law for rough boundaries can also be written in the form,

\[
\frac{u}{u_*} = \frac{1}{k} \ln \frac{z-d}{z_0}
\]  

where \( d \) is the zero-plane displacement \((z-d=z')\) and \( z_0 \) is the characteristic bed roughness height. The last one indicates the position where the logarithmic law is zero and it depends on the roughness of the bed (Nikora et al. 2001). The characteristic roughness height and the equivalent sand roughness can be related by

\[
\frac{z_0}{k_s} = a^b
\]  

For high Reynolds numbers, \( k = 0.40 \) and \( B = 8.5 \) the equation (5) is equal to 0.0333. In the Figure 1 is shown a schematic velocity profile for rough beds. Normally, in the literature, the logarithmic layer limit is \( z/h=0.20 \) to 0.30.

#### Figure 1 - Velocity profile for rough bed

**Clauser’s method**

This method adopted by Clauser (1956) can be applied for rough or smooth boundaries applying the equations (2) and (3). The basis is to do a linear regression in the form \( u = M \ln z + B' \) to the equations mentioned being,

\[
\frac{u}{u_*} = \frac{1}{k} \ln \left( \frac{z u_*}{v} \right) + B
\]  

Equation (7) is applied for smooth beds and equation (8) for rough beds. Equation (6) is applied for both cases. For rough boundaries there is one more unknown variable, the zero plane displacement, \( d \). For that reason, the values of \( d \) may be estimated by iteration until \( B = 8.5 \).
Pokrajac et al. (2006) and Ferreira et al. (2012)

This two methods are based in the same equation. Accepting wall similarity in the sense of Townsend (1976), the equation (4) is valid. Monin and Yaglom (1971) suggested that the boundary zero could be inferred by fitting the data to a logarithmic profile and forcing the slope to be 1/k ≈ 1/0.41. Differentiating the log formula for u with respect to z (Nikora et al. 2002),

\[
\frac{du}{dz} = -\frac{k}{u} (z - d) \tag{9}
\]

Pokrajac et al. (2006) fits a line to the experimental data. The slope is k/u- and the intercept gives the position of the zero-plane d. Ferreira et al. (2012) suggests two scenarios for flow independent of k: one, where the constants B and ks are subjected to a best fit procedure, and another one, where the constant B is considered equal to 8.5 and ks is calculated by a roughness function.

**Modified Log-wake law (MLWL)**

Guo and Julien (2008) proposed a modified log-wake law for open channels associated with the velocity dip phenomenon.

\[
\frac{u_u}{u} = \frac{1}{\delta} \ln \left[ \frac{u_{\text{max}}}{u} \right] + \frac{2}{\delta} \cos \frac{\pi}{2} - \frac{1 - \xi^2}{3k} \tag{10}
\]

where ξ = z/δ, δ is the distance to the bed where the velocity is maximum. This equation is valid for the entire velocity profile. Since the MLWL reduces to a parabolic law near the surface (Guo and Julien, 2003), it is assumed that, for data with ξ ≥ 0.6,

\[
u = ax^2 + bx + c \tag{11}
\]

\[
\delta = -b \tag{12}
\]

\[
u_{\text{max}} = c \tag{13}
\]

Applying equation (10) to data where ξ ≤ 0.2 gives the law of the wall (equation 14),

\[
u = \frac{U}{k} \ln \xi + BU_k \tag{14}
\]

\[
u = \nu_{\text{max}} = \frac{U}{k} \left( 2/3 - \frac{1}{3} \right) \tag{15}
\]

The slope of (equation 14) gives the shear velocity and the intercept gives the Coles wake parameter, II. The authors also suggest an equation to apply to flow measurements.

\[
u = \frac{U}{k} \left( \frac{U}{z_0} - \frac{1}{3} \left( \frac{\xi - z_0}{\delta - z_0} \right)^3 \right) \frac{2\nu_{\text{u}}}{k} \sin \frac{\pi (z - z_0)}{2(\delta - z_0)} \tag{16}
\]

Assuming k = 0.41 there are four or five fitting parameters for smooth and rough boundaries, respectively: u*, z0, δ, II and d. The parameters can be fitted using a non-linear optimization program.

**2.2.3. Turbulence intensities**

**Turbulent kinetic energy (TKE)**

The absolute intensities of velocity fluctuations can be used to infer bed stress through TKE.

\[
\text{TKE} = \frac{1}{2} (u^2 + v^2 + w^2) \tag{17}
\]

Simple linear relationships have been formulated in turbulence models. Soulsby and Dyer (1981) observed that the average ratio of shear stress to TKE is constant.

\[
|\tau| = C_T \rho \text{TKE} \tag{18}
\]

where C_T is a proportionality constant and can be assumed =0.20. Kim et al. (2000) suggested C_T = 0.21. By assuming linear relationships between TKE and the velocity fluctuations, the bottom stress can also be related to a variance component such as (Kim et al. 2000),

\[
|\tau| = C_T \rho w^2 \tag{19}
\]

where C_T = 0.9 (Kim et al. 2000).

**Wall similarity method (WS)**

This method implies that an extended depth range exists where turbulent energy production and dissipation are nearly in equilibrium and diffusion is negligible, independent of flow and bed roughness conditions. In the equilibrium layer (0.15 < z/h < 0.60) a balance exists between production and dissipation (Townsend, 1976). The vertical flux of TKE is approximately constant in the equilibrium layer. Hurther and Lemmin (2000) showed that the vertical flux of TKE normalized by the cube of the shear velocity is given by,

\[
\frac{1}{2\rho} \left( u^2 + v^2 + w^2 \right) w = F_k \tag{20}
\]

F_k was found to be a constant with a mean value of 0.3 for the range 0.15 ≤ z/h ≤ 0.60 (Lopez and Garcia, 1999; Hurther and Lemmin, 2000).

**Turbulence intensities**

Universal equations to determine the variations of the turbulence intensities were proposed for open channels. The equations suggested by Nezu and Nakagawa (1993) are based in the same assumption as the wall similarity where the turbulent energy production and dissipation are nearly in equilibrium and diffusion is negligible in an intermediate region of the flow. For smooth channels the equations are:

\[
\frac{\sqrt{u^2}}{u} = 2.30e^{-z/h} \tag{21}
\]

\[
\frac{\sqrt{v^2}}{u} = 1.63e^{-z/h} \tag{22}
\]

\[
\frac{\sqrt{w^2}}{u} = 1.27e^{-z/h} \tag{23}
\]

For rough bottoms Kironoto and Graf (1994) suggested:

\[
\frac{\sqrt{u^2}}{u} = 2.04e^{-0.5z/h} \tag{24}
\]

\[
\frac{\sqrt{w^2}}{u} = 1.14e^{-0.75z/h} \tag{25}
\]
2.2.4. Vertical Reynolds stress profile, $\tau_{xz}$

The longitudinal shear stress at a distance $z$ from the bottom, for uniform and 2D flows, can be obtained by the equilibrium of forces applied in that direction in a control volume (Figure 2).

\[
\tau = \gamma(h-z)J = \frac{y}{h}J \left(1 - \frac{z}{h} \right) \tag{27}
\]

In turbulent flows the shear stress due to the dynamic viscosity, is

\[
\tau_l = \mu \frac{du}{dz} \tag{28}
\]

and the component due to the velocity turbulence intensities is defined by,

\[
\tau_t = -\rho u' \tilde{w}' \tag{29}
\]

where $u' \tilde{w}'$ is the mean of the product between $u'$ and $w'$. For values higher than $\delta'$ (viscous layer thickness) the total shear stress, $\tau$, is approximately equal to $\tau_l$ and for values lower than $\delta'$ the total shear stress is nearly $\tau_l$ (Figure 3).

3. Experiments

In this study, the experiments were carried out in a flume in the National Laboratory of Civil Engineering (LNEC) in Lisbon, Portugal. The compound channel is 10 m long and 2 m wide. The trapezoidal main channel, with 0.1 m bank full height, 0.4 bottom width, 0.6 bank full width and a side slope of 45º, is made of polished concrete. The floodplains are 0.7 m wide roughened with a 5 mm high synthetic grass. The bottom slope is 0.0011 m/m. From previous experiments on the same flume but in single open channel, Manning Strickler coefficient, $n$, is equal to 0.0092 m$^{-1/3}$/s, for the polished concrete, and 0.0172 m$^{-1/3}$/s, for the synthetic grass.

A schematic representation of the compound channel is presented in Figure 4. Following the recommendations of Bousmar et al. (2005), separated inlets for the main channel and for the floodplains were established. The supply is ensured by an elevated constant level reservoir. For each subsection (i.e., main channel and the left and right floodplains), the flow rate is controlled by a valve and monitored by an electromagnetic flow meter (uncertainty of 0.3 l/s). At the end of the flume, three independent tailgates (one per subsection) were used to adjust the water level in the flume.

Vertical elements were used to simulate riparian vegetation at the edge of the main channel. Two different types of vertical elements were used. The first consists of rods partly-submerged without foliage, with 6 mm diameter and 45 mm height (see Figure 4d). The second type simulate shrubs with a highly dense foliage crown, 90 mm diameter, with spherical shape located radially around the center on top of the vertical rods (see Figure 4b and 4c). The arrangement of the vegetation elements is defined by the spacing ratio, L/D, equal to 15. The spacing between elements is 90 mm leading to a situation where the crowns of adjacent elements with foliage touch each other in the longitudinal direction.

Three types of cases were studied, all of them with rough floodplains: (i) without vertical elements, (ii) with rods in the interface and (iii) with shrubs in the interface, for two relative depths, 0.20 and 0.30 (Table 2). Velocity measures were made with a 3D sideloooking ADV. The acquisition time is 3 min in each measurement position, with a sampling rate of 100 Hz. The sampling volume is a 7 mm long and 6 mm diameter cylinder, and the transmit length was set to 1.8 mm. The velocity time series were despiked with the filter proposed in Goring and Nikora (2002). Only correlations higher than 70% were considered. To align the ADV probe with the longitudinal direction, the pitch angle was slightly modified to obtain a depth averaged spanwise velocity, $V$, equal to 0 near the lateral wall in the floodplain. For the measurements near the water surface, only 2D velocity components (in
plane \(xy\) were obtained because the receivers could not remain inside water to measure correctly. The measurements were made 7.5 m downstream the flume inlet, where the flow is uniform for the study flow depths. In the cross section, 22 vertical profiles were measured, with more verticals measured in the regions where the velocity components change the most (interface). Since it is needed to use the profiles to estimate the boundary shear stress, 11 to 12 points were measured in the floodplains per vertical, increasing the number in the lateral of the channel with the water depth. Considering the symmetry of the channel, measured verticals located only in the left half of the channel.

4. Results and discussion

4.1. Velocities and lateral shear stress in cross section

The Figure 5 presents the patterns of the streamwise velocity normalized by the cross section averaged velocity, \(U_m\) for all the measured scenarios. The cross section averaged velocity was obtained by the integration of the depth-averaged velocities in the cross section, to ensure mass conservation. For the integration of the streamwise velocity, \(u\), a third-order polynomial was fitted to the profile. Figure 6 shows the depth-averaged velocities in the cross section for the flow cases with rough floodplains and no vertical elements in the interface for \(h_r=0.20\) and \(h_r=0.30\) (HR020R and HR030R, respectively), rough floodplains and rods in the interface for \(h_r=0.20\) and \(h_r=0.30\) (HR020RR and HR030RR, respectively) and rough floodplains and shrubs in the interface for \(h_r=0.20\) and \(h_r=0.30\) (HR020RS and HR030RS, respectively).

The patterns show that, in the floodplain, for all flow cases, there is a part of the flow with very low velocities, which means that the synthetic grass almost stops the flow and influences a significant part of the vertical (until \(=0.12\) m). This strong decrease of the velocity in the floodplain due to the synthetic grass was never shown in previous studies. The velocity gradient between the main channel and the floodplains is higher when there are rods and shrubs in the interface. In the presence of vertical elements in the interface, the velocity in the main channel becomes higher than in the cases without riparian vegetation and the maximum velocity region is not observed near the surface of the main channel flow, but moves towards its bottom. This effect may be a result of the secondary currents induced by the riparian vegetation and to the lateral shear generated at the rods and shrubs. This effect is more noticeable in the case of shrubs. In that case the velocity becomes almost zero near the interface.

The cross section distribution of the lateral shear stresses, \(\tau_{xy}=-\rho u'v'\), is presented in Figure 7. Figure 8 shows the distributions of the depth-averaged lateral Reynolds stresses, \(\tau_{m,xy}\), in cross section. The depth-averaged lateral shear stresses were obtained by calculating the mean values of \(\tau_{xy}\) for each vertical. For all flow cases the lower shear region is located near the interface. In other parts of the channel the lateral shear stress is approximately zero. The presence of riparian vegetation affects significantly the lateral shear stress distribution. With rods and shrubs the shear layer width and maximum value are higher for higher relative depths. The opposite happens without riparian vegetation. For rods, the spreading of the shear region towards the bottom of the main channel is similar to what happens without vertical elements in the interface. With shrubs the spreading is towards the surface. In the interface, on the side of the floodplain the values become positive mostly for the case with shrubs. At some point between the high and the low value of the lateral shear stress (in the interface) this variable becomes zero, which means that the vegetation suppresses the momentum exchange between the main channel and the floodplain. This means that the lateral shear stress is not due to the momentum exchange between the main channel and the...
Figure 5 – Distribution of the streamwise velocity, u, scaled by the cross section averaged streamwise velocity, U_

Figure 6 – Depth-averaged streamwise velocity, U, scaled by the cross section averaged streamwise velocity, U_

Figure 7 – Cross sectional distribution of the lateral shear stress

Figure 8 - Depth-averaged lateral shear stress
floodplain but a consequence of the riparian vegetation on the velocity field.

4.2. Boundary shear stress

4.2.1. Applicability of methods

A preliminary study of the methods mentioned in 2.2 was made with a higher number of measured positions (23 positions) in the center of the floodplain for the case HR030RR. The objective of this study consisted on the applicability assessment of the methods for estimating the boundary shear stress and other parameters of the flow. The results of the velocity shear stress, $u_*$, constant B and zero-plane displacement, $d$, are presented in Table 3 for all methods. The MLWL 1 and MLWL 2 refer to the application of the method of the modified log-wake law with the equation (10) and (16) respectively. The TKE-W is the turbulent kinetic energy method corresponding to equation (19). The results are shown in velocity shear stress because most of the methods estimate that parameter.

| Table 3 - Velocity shear stress, constant B and zero-plane displacement |
|-----------------------------|-----|-----|
| Method                     | $u_*$ (m/s) | $B$ | $d$ (m) |
|-----------------------------|-----|-----|
| Momentum equation, $\omega_*=\sqrt{g/\rho h}$ | 0.0213 | - | - |
| Clauser (1969)             | 0.0245 | 8.54 | 0.0126 |
| Pokrajac et al. (2006)     | 0.0239 | 8.69 | 0.0129 |
| Ferreira et al. (2012)     | 0.0274 | 8.50* | 0.0132 |
| MLWL 1                     | 0.0229 | - | 0.0130 |
| MLWL 2                     | 0.0238 | 8.75 | 0.0134 |
| $\tau_{xz}$                | 0.0190 | - | 0.0120* |
| TKE                        | 0.0230 | - | 0.0120* |
| TKE-W                      | 0.0477 | - | 0.0120* |
| WS                         | 0.0187 | - | - |
| Turb. Intensities $u'$     | 0.0304 | - | 0.0130* |
| Turb. Intensities $w'$     | 0.0402 | - | 0.0130* |

*Value imposed for the method, not calculated

Except the vertical Reynolds stress method, all methods give higher estimates of $u_*$ than the momentum equation, $\omega_*=\sqrt{g/\rho h}$. The TKE-W method gives an over estimated value comparing to Clauser, TKE, MLWL and Pokrajac et al. (2006). The values of the zero-plane displacement, $d$, are over 2 times the height of the roughness, and about 30% of the water depth. This means that the roughness in the floodplains has a big influence in the velocity profile for shallow water. Being $d$ so high we are in the presence of a $-d$ flow type, in the sense of Perry et al. (1969), where the flow circulates over the roughness elements without interacting with the flow inside them.

The linear adjustments made for the Pokrajac et al. (2006) method and vertical Reynolds stresses methods had a correlation of $R^2=0.76$ and 0.72, respectively. Knowing this, the two methods mentioned seem inappropriate for fewer measured positions. The results of the wall similarity method do not form a constant layer as it was mentioned in the literature. The method of Ferreira et al. (2012) was difficult to apply because of all the variables unknown.

So, for the distribution of the boundary shear stress in the cross section the methods of Pokrajac et al. (2006), Ferreira et al. (2012), WS, vertical Reynolds stress, TKE-W and turbulence intensities will not be considered. Because the determination of the velocity shear stress by the MLWL 1 method is done the same way as the Clauser’s method, the results of the boundary shear stress estimated by the MLWL 1 method are not presented.

It was possible to calculate also the equivalent sand roughness, $k_s$, from the methods of Pokrajac et al. (2006) and MLWL 2, assuming $B=8.5$. The results obtained were 6.3 mm from the first method and 6.1 mm from the second. These results are close to 6.8 mm, which is the value suggested by Fernandes (2013) for the same bed roughness. The Coles wake parameter was also estimated by the methods MLWL 1 and MLWL 2, with the results 0.387 and 0.07, respectively.

Since this study is in a compound channel, the distribution of the boundary shear stress in the cross section is difficult to estimate because of uncertainty about the validity of the logarithmic law and difficulties in measuring near the interface. So in that area the results cannot be guaranteed to be right. When the density of vegetation in the interface region is higher the uncertainty area is larger, comparing rods to shrubs. In the case of rods the uncertain region has a width of $\approx h$ from the interface to the floodplain and ends at the toe of the inclined margin of the main channel. For the case of shrubs the uncertainty area is wider in the floodplain, where the width $\approx h$ is ads to the shrubs’ width.

4.2.2. Zero-plane displacement

This variable was estimated by Clauser’s method and the MLWL 2. For the first one, the zero-plane displacement was calculated by changing the value of $d$ until $B$ became equal to 8.5 for each vertical. For the MLWL 2 method $d$ was calculated by nonlinear regression. Until approximately $y=0.60$ m the values of $d$ are almost constant and near the interface, $y\geq0.60$, the results start to spread, increasing in some cases and decreasing in others. Near the interface, it is not possible to fix a value of the zero-plane displacement. The MLWL 2 gives less scattered values than the Clauser’s method. No relation between the results has been found.

| Table 4 - Zero-plane displacement considered for all cases |
|-----------------------------|-----|-----|-----|
| Case | Rough floodplains | Rough floodplains and rods | Rough floodplains and rods |
| $h_r(\cdot)$ | $d (mm)$ | | |
| 0.20 | 12.0 | 14.0 | 14.0 |
| 0.30 | 12.0 | 12.0 | 13.0 |

The definition of the zero-plane displacement is important because the other methods need to assume a value of $d$ to estimate the boundary shear
stress. Table 4 shows the values of \( d \) used each case to estimate the boundary shear stress through some methods. These results were obtained by the mean of \( d \) values in the non-disturbed area of the flow.

### 4.2.3. Boundary shear stress distribution

The results of the boundary shear stress for each method (Clauser’s, TKE and MLWL 2) and for all the cases are presented in Figure 9. The Clauser’s method results presented were estimated considering constant values of \( d \) (Table 4). The TKE method was not applied for the lower relative depths because it uses the vertical component of velocity and, for those relative depths, the measurements of \( w' \) cannot be guaranteed to be correct. In the figures are presented the shrubs region and the uncertainty region (that includes the shrub region). The results are compared with the boundary shear stress estimated by \( \rho g h i \) and \( \rho g i R \) (R - hydraulic radius), in the main channel.

Far from the interface, the values in the floodplain are very close to those derived on the basis of the momentum equation \( \tau_0 = \rho g hi \). There is no tendency between methods unless in the main channel where the TKE gives results higher than the other two methods and in the floodplain near the interface where this methods also gives higher estimates. In the center of the main channel, the results tend to a constant value, but, in this case are much, lower than those given by \( \tau_0 = \rho g hi \). The shear velocity in the main channel is much closer to the estimates of \( \tau_0 = \rho g Ri \), which makes sense because this section of the channel is not very wide and the flow cannot be completely 2D like in the floodplain.

In the main channel the tendency is to decrease towards the interface, for the Clauser’s method and MLWL 2. The TKE method gives opposite results. This situation happens in all cases.

In the presence of rods and in the case without...
riparian vegetation in the interface, approximately the same lateral distribution of the boundary shear stress is obtained. With shrubs in the interface, \( \tau_0 \) decreases near the interface instead of increasing like in the other cases. These distributions are in accordance with Terrier et al. (2010).

The differences between \( \tau_0 \) and \( y_i \) are higher for the case with shrubs (\( \approx 16\% \) for \( h=0.20 \) and \( \approx 23\% \) for \( h=0.30 \)). This large difference can be balanced by the shear stress generated in the shrubs foliage. When comparing the methods, the Clauser’s method gives averaged cross section boundary shear stresses with higher differences and the TKE method gives the lowest. In the main channel, the errors decrease from \( \approx 50\% \) to \( \approx 30\% \) in respect with \( y_i \) and \( y_{ri} \).

The distribution of the boundary shear stress without vertical elements of vegetation in the interface is approximately the same as the measurements by Fernandes (2013). There are some differences in the floodplain where the author measured higher values than \( \rho g_{hi} \), and in this study the channel slope results and the methods estimates are almost equal in most cases.

When comparing this distribution with the distribution of the boundary shear stress with smooth floodplains, a significant increase closer to the interface is observed. This increase is higher with lower relative depth flows and when the floodplains are rough.

There is not a method that is clearly better than the other, mostly because the floodplains have rough beds and low flow depths which give results in high uncertainty when applying the methods. Since the TKE method gives a boundary shear stress distribution different from the other methods and from the other studies, it is considered that this method fails for this particular case of compound channels.

In the margins of the main channel, the estimates of \( \tau_0 \) highly decrease and it was considered that these results can be more accurate analyzing profiles normal to the bottom of the channel (\( y=0.20 \) to \( y=0.40 \) m), instead of vertical profiles, since the logarithmic law can be valid in that direction. The results showed that the boundary shear stress is generally higher, except for the shrub cases, where the results do not change. Figure 10 shows these results for the Clauser's method. In that figure, in the main channel the line corresponds to \( \tau_0 \) calculated by \( \rho g_{ri} \) and in the floodplains by \( \rho g_{hi} \).

5. Conclusions

In this work the influence of the riparian vegetation in compound channels was assessed. The velocities were measured which rendered it possible to assess the velocities and Reynolds stress in the compound channel. Methods for estimating the boundary shear stress were applied with the results of velocity and turbulence intensities.

The introduction of the riparian vegetation in the interface increases the velocity in the main channel and decreases it in the floodplains for the same discharge. The synthetic grass decreases significantly the velocity until 30\% of the water depth in \( h=0.30 \) and 50\% for \( h=0.20 \), having a large influence in the flow. The shrubs decrease significantly the velocity near the interface suggesting that the elements suppress the momentum exchange between the main channel and the floodplains.

The lateral Reynolds stresses near the interface have a negative peak that without the vertical elements is higher for lower relative depths but in the case if rods or shrubs the peak is higher for higher relative depths. What suggests that the lateral shear stress is not due to the interaction between the main channel and the floodplain but a consequence of the influence that the vertical elements have in the velocity.

The TKE-W turbulence intensities profile and WS methods could not be applied to the channel. Also the Pokrajac et al. (2006), Ferreira et al. (2012) and vertical Reynolds stress profile methods were difficult to apply in vertical with a small number of measured points.
The zero-plane displacement, d, has constant values far from the interface. The median values are 12 to 14 mm, which are far higher than the roughness of the bed (5 mm).

In the interface region, the TKE method gives different boundary shear stress distributions than the Clauser’s method and the MLWL 2 and from other studies in this area. So this method seems to fail the assessment of $\tau_0$ in that area.

By introducing vegetation in the floodplains, the boundary shear stress in the main channel decreases and increases in the floodplain edge, near the interface. The case with shrubs decreases the boundary shear stress in the interface region as does the longitudinal velocities. However, near the interface the results are uncertain so it cannot be guaranteed to be correct. In the floodplains $\tau_0$ is equal to $\rho g h_i$ and in the main channel it is lower than that value, being closer to $\rho g R_i$. The boundary shear stress in the main channel is practically independent of the water depth. In the floodplain this parameter is higher for higher relative depths being the opposite near the interface.

Analyzing profiles normal to the bed in the main channel margins, the estimates of $\tau_0$ are higher than in vertical profiles, except for the case with shrubs.

**Notation**

$a, b, c$ fitting parameters of the parabolic law  
$B$ constant of the log law  
$d$ zero-plane displacement  
$h$ water depth  
$h_r$ relative depth  
$h_{fp}$ floodplain depth  
$h_m$ main channel depth  
$i$ channel slope  
$k$ Von Kármán constant  
$L/D$ spacing ratio  
$P$ wetted perimeter  
$u$ time averaged streamwise velocity  
$U$ depth-averaged streamwise velocity  
$\bar{u}$ shear velocity  
$U_m$ cross section averaged streamwise velocity  
$v$ cartesian coordinate, lateral direction  
$x$ cartesian coordinate, longitudinal direction  
$z$ cartesian coordinate, vertical direction  
$z_0$ bed’s characteristic roughness height  
$\gamma$ fluid weight  
$\delta$ distance to the bed where the velocity is maximum  
$v$ kinematic viscosity  
$\xi$ normalized distance  
$\Pi$ Coles wake strength  
$\rho$ water density  
$\tau_0$ boundary shear stress  
$\tau_{m,xy}$ depth-averaged lateral shear stress  
$\tau_{xy}$ lateral shear stress  
$\tau_{xz}$ vertical shear stress

**References**


Pokrajac, D., Finnigan, J. J., Manes, C., McEwan, I. and Nikora, V.


