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Hydraulic parameters in channels with wall vegetation and gravel bed

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Abstract

In this experimental study, field observations and laboratory experiments have been carried out to assess the impacts of the vegetated channel walls and aspect ratio on flow velocity profiles, shear stress distribution and roughness coefficient of channel. Results show that the presence of vegetation cover on channel wall causes deviation of the Reynolds stress distribution from the linear one under uniform flow condition. It is also noticed that the Reynolds stress distribution is influenced by the aspect ratio (*W/h*). The vegetation cover planted on the flume wall considerably influences the location of the $u_{\max}$. With the changes of the position of $u_{\max}$, the zero Reynolds stress position also changes. The Manning coefficient *n* of channel with gravel bed and vegetated walls have been assessed and calculated by using the boundary layer characteristics method.

Key Words: Aspect ratio, Experiment, Reynolds stress, Roughness coefficient, Vegetated channel wall, Velocity profiles

1 Introduction

In practice, to determine river sediment transport and flow resistance, both shear stress and velocity are normally required. In natural channels, vegetation does usually exist. The vegetation in a channel has considerable influence on water resources management such as estimation of roughness coefficient, simulation of water level along a river reach, etc. The momentum exchange between the stationary water near river bank and high velocity in the central zone of channel is very important for any restoration projects including stabilization of river bank. Contaminants transport associated with sediment in coarse-bed channels is another important problem at many sites where tailings remain void of vegetation (Yi et al., 2008). Also, vegetation can be used in a beneficial manner to control pollution and erosion, and to provide more physical resistance to surface runoff (Liu et al., 2008; Wang et al., 2008a; Sui et al., 2009). Conceptually at least, vegetation can also obstruct the movement of the contact portion of the bed load and hinders or stabilizes the bed forms (Vanoni, 1977). Generally, vegetation is a controlling factor in interaction of flow, erosion and geomorphology in rivers and may cause difficulties in hydraulic design. Masterman and Thorne (1992) pointed out that bank vegetation to be a key factor in reducing flow velocity. Thus, vegetation occupies a pivotal place in the river mechanics and fluvial geomorphology. Recently, some research works have been reported regarding the velocity profiles and the turbulent characteristics of vegetated channels. Lopez and Garcia, 2001). However, the impact of bank vegetation depends on many factors such as flow condition, distance from the bank vegetation to measuring point and vegetation density. As claimed by Wang et al. (2008b), the effect of vegetation on soil erosion in a range of river environments seems to remain unexplored.

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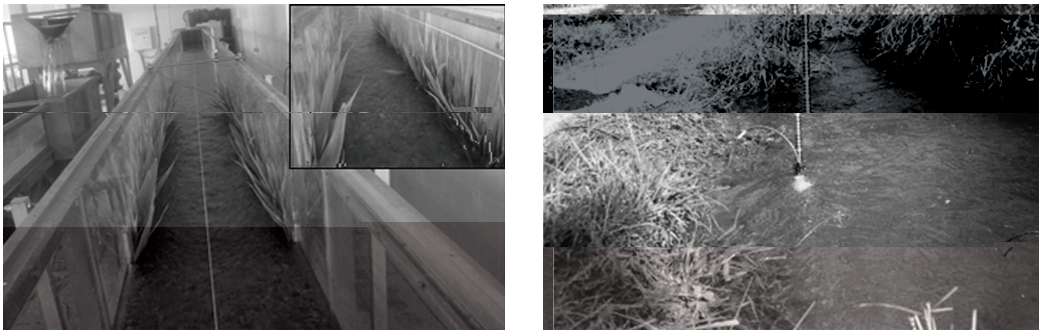
In view of the importance of vegetation in channels, researchers have carried out some investigations regarding vegetative roughness coefficient by using different methods, as reported by Chow (1959), Dawson and Charlton (1988) and Fisheich (1994). Recently, researchers have used real vegetation in their studies regarding the roughness coefficient of vegetated channels (Fathi-moghadam and Kouwen, 1997; Werth, 1997; Rahmeyer, 1998; and Wu et al., 1999).

The objectives of this study are: 1) to investigate the effect of aspect ratio (*W/h*, where *W* is width of channel or flume and *h* is flow depth) on velocity profiles and Reynolds stress distributions in channel or flume which has a gravel channel bed and vegetated river bank; 2) to evaluate Manning roughness coefficient by using the boundary layer characteristics method and to compare our results with the application of the Manning equation and those reported by other researchers.

2 Experimental set up

Two types of experiments have been carried out: 1) experiments in the field and 2) experiments in laboratory. In both types of experiments, the channel/flume bed was covered by gravel and the channel bank or flume wall have been planted by vegetation, as shown in Fig. 1.



(a) (b)

Fig. 1 View of test sites in laboratory flume (a) and field (b)

2.1 Field experiments

Three selected natural channels are located in central Iran, near the city of Isfahan. In each channel, a 30 m-long channel reach has been chosen for collecting data. Flow discharge is provided by a traditional Qantas. The median grain size of bed material has been determined by the Wolman method (1954) as d_{50}= 41 mm with the standard deviation of $(d_{84}/d_{16})^{0.5}$ =1.3, as shown in Fig. 2. Along each studied channel reach, six cross sections with a distance of 5 m to adjacent cross section have been set up for measurements. At each cross section, three velocity profiles have been measured. One velocity profile has been measured in the central part of the channel, and other two velocity profiles have been measured along channel bank in a distance of D/W=0.2 from channel bank (where, D is the distance from the channel bank). Each velocity profile consists of 14 measurements of flow velocities from the channel bottom (2.5 cm from the channel bed) to the water surface (2 cm beneath water surface).

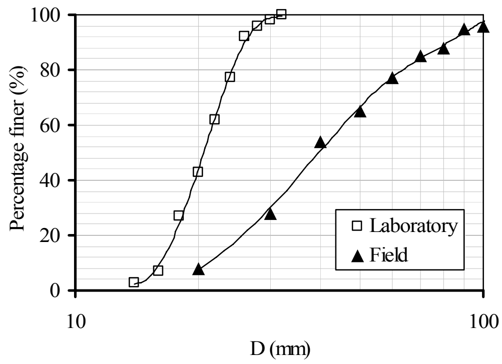


Fig. 2 Grain size distribution of bed material

A current meter made in Vale Port Company has been used to measure the point velocities. The diameter of the sensor of this current meter is 5 cm. Therefore, the closest point to the channel bed for measurement of flow velocity was 2.5 cm above the channel bed. The sampling duration of each measurement is 1 minute. Velocities have been measured at each measurement point thrice, with the average of these three values taken as representative of the actual velocity for that point. Bed slope was measured by using a Theodolit and as division of difference in elevation of two consecutive points along a channel reach by the distance. The discharge was calculated by using the continuity equation $Q=AV$ (where, A is the cross sectional area and V is the weighted velocity in each section).

2.2 Laboratory experiments

Laboratory experiments have been conducted in a flume of 20 m long, 0.6 m wide and 0.6 m deep. An ADV has been used in laboratory to measure velocity profiles. This ADV has a sampling frequency of 200 Hz. At a distance of 11.5 m from the upstream entrance cross section, the flume walls have been covered by vegetation called Arundo donex. The channel bed was covered by gravel which has median grain size of d_{50} = 21 mm with a standard deviation of less than 1.3. The bed slope was constructed by manual deposition of different thickness of gravel along the flume. The obtained bed slope of 0.005 was kept constant in this study. The flow condition was uniform along the flume using adjustment of a tail-gate at the end of the flume. Along the flume, velocity profiles have been measured at 12 m, 12.9 m and 14 m from the upstream entrance cross section. At each cross section, three velocity profiles have been measured at a distance of 5 cm, 12 cm and 30 cm from the flume wall. Each velocity profile consists of 15 to 20 measurement points for velocities, and seven out of these measurements have been carried out to measure flow velocity near channel bed ($y/h<0.2$, where, y is distance from the bed). At each measurement point, the sampling period is 2 minutes. Flow discharge was 65 liter/s and three runs have been carried out under different water depth of 80 mm (“Run 1”), 110 mm (“Run 2”) and 150 mm (“Run 3”).

3 Results and discussion

3.1 Velocity distribution

As shown in Fig. 3, the measured three velocity profiles in the flume have an identical shape. The flow is fully developed in the measuring channel reach of the flume from 12 m to 14 m from the upstream entrance cross section. As Fig. 4 indicates that, under flow condition of “Run 1”, the maximum velocity (u_{max}) on velocity profile near flume wall (5 cm from flume wall) occurs at the water surface. With the decrease in the aspect ratio of W/h=5.4 (“Run 2”), $u_{\max}$ on velocity profile near flume wall (5 cm from flume wall) occurs under water surface at a depth of y/h=0.49. Also, at distances of 12 cm and 30 cm from the vegetated wall, the position of $u_{\max}$ is at a water depth of y/h=0.68. The reason for this change in position of $u_{\max}$ is the diminishment of secondary current power by getting far from the vegetated walls. This statement can be justified by considering “Run 1” with an aspect ratio of W/h=7.5 and $u_{\max}$ occurs at the water surface. On the other hand, even if W/h=7, the effect of the secondary currents can be observed near the banks in the natural channel, and the $u_{\max}$ near bank is under the water surface. Consequently, the aspect ratio of W/h=5 should not be considered as the criterion for determining whether or not a secondary current occurs, especially for the natural channels. For least aspect ratio of W/h=4 (“Run 3”), $u_{\max}$ occurs under water at a water depth of y/h=0.46 and the difference in flow velocity between $u_{\max}$ and flow velocity near water surface is 50%. Kironoto and Graf (1995) pointed out that this difference for flow over gravel-bed flume with glass walls was 4%. The reason for different behaviors between “Run 1” and “Run 3” can be attributed to the impacts of aspect ratio, vegetation cover and distance from the flume wall. For “Run 3” where $u_{\max}$ occurs under water surface, the vertical and lateral components of velocity are very active. Consequently, at water surface, the lateral component of flow velocity (v) moves from the flume walls toward the central axis of the flume and in the vertical direction, the velocity component is forced to shift from the water surface toward the flume bed.

Some researchers claimed that the aspect ratio is an important factor affecting the formation of the secondary currents in channel (Sui et al., 2008). A small aspect ratio will definitely cause a very important impact on the formation of the secondary currents in an open channel. According to the findings of Nezu et al. (1989) using a smooth bed and Graf and Altinakar (1998) using a gravel bed, when the aspect ratio

W/h larger than 5, the secondary currents will not be formed in a channel. This means, when *W/h*> 5, u_{max} should occur at the water surface. However, our results indicate that even if *W/h*>5, the secondary currents are still active due to the effect of the vegetation on channel walls. The measured u_{max} occurred under the water surface.

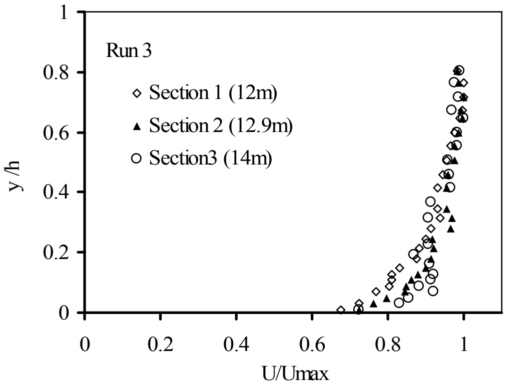


Fig. 3 Measured velocity profiles in flume at a distance of 30 cm from the flume wall

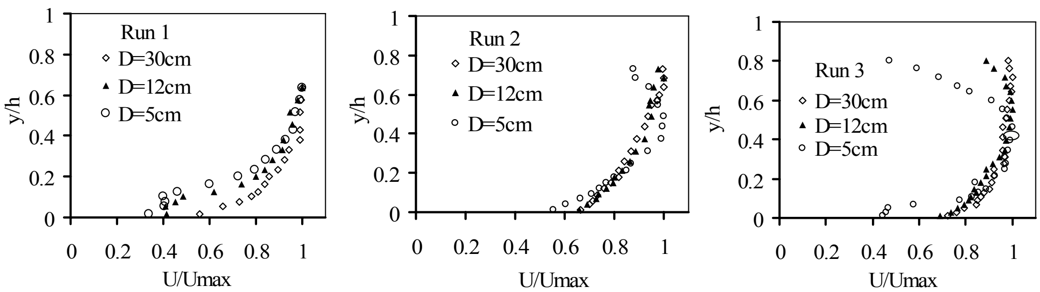


Fig. 4 Velocity profiles measured at different distance from flume wall

In the natural channels, as shown in Fig. 5, even though *W/h*=7, u_{max} of velocity profiles in the central axis is located at water surface. However, instead of locating at water surface, u_{max} of velocity profile measured at the left side of the central axis is located at a water depth of *y/h*=0.53; and u_{max} of velocity profile measured at the right side of the central axis is located at a water depth of *y/h*=0.48. The difference between u_{max} and the measured velocity closest to water surface is 12%. Therefore, one can say that the vegetation cover on channel/flume walls considerably influences the location of the u_{max} even for *W/h*>5 due to development of the secondary currents.

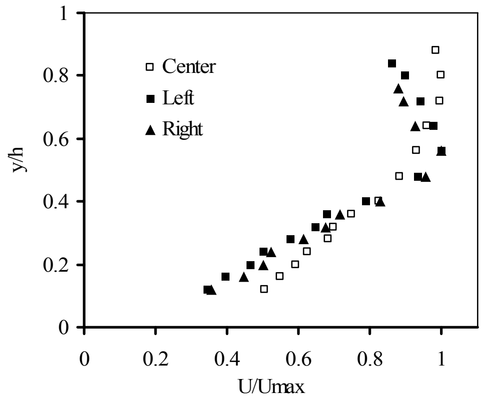


Fig. 5 Velocity profiles measured in the field

As pointed out by Afzalimehr and Dey (2009), the log law is valid for flow in a flume with gravel bed and vegetated flume wall. The results of this study indicates that, for an aspect ratio of W/h=7, the upper limit of the log law is near the water surface, y/h=0.85. However, approaching to the flume walls, the valid upper limit of this log law decreases to y/h=0.3, as shown in Fig. 6. Additionally, under flow condition of "Run 1", this limit is only y/h=0.1. Under flow condition of "Run 3", when W/h decreases to 4, the log law is applicable between y/h=0.3 and y/h =0.8 in the central axis, as shown in Fig. 7. The lines in both Figs. 6 and 7 display the log law fitness to the velocity points near the bed.

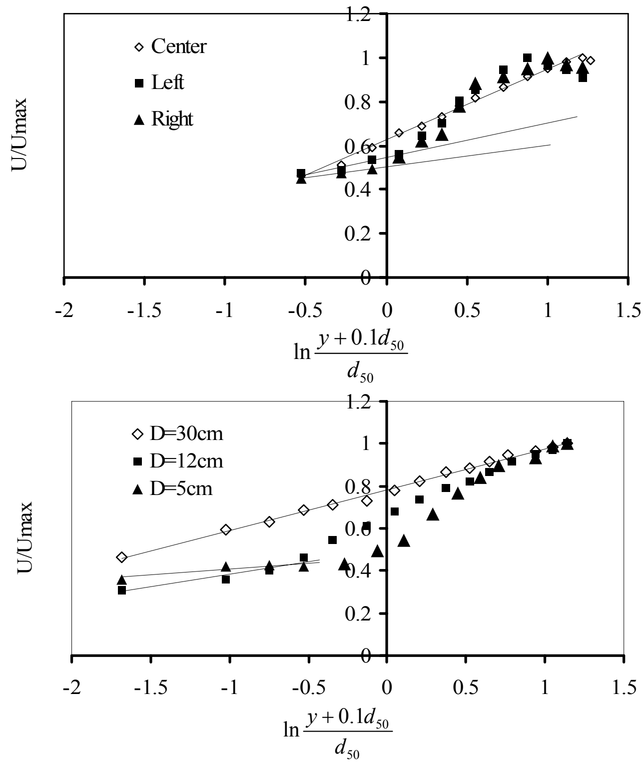


Fig. 6 Logarithmic velocity profiles fit for field observation (top) and laboratory (bottom) with W/h = 7.5

3.2 Shear stress distribution

For uniform flow, the shear stress distribution can be expressed by the following equation:

$$\frac{\tau}{\rho} = -\overline{u'w'} = u*^2\left(1-\frac{y}{h}\right) \tag{1}$$

The linear Reynolds stress distribution has been reported by Kironoto and Graf (1995) for uniform flow over gravel-bed streams. However, the shear stress distribution in a channel with vegetated channel differs from the findings of Kironoto and Graf (1995). Results of our study show that the Reynolds stress distribution is deviated from the linear distribution even for the uniform flow, as shown in Figs. 8 and 9. Furthermore, the Reynolds stress distribution depends also on the aspect ratio. This means, when W/h=7.5, except near flume bed, Eq. 1 can be used to describe the shear stress distribution.

Yang et al. (2006) obtained the following equation under steady non-uniform flow condition, by simplifying the Navier Stokes equation

$$-\frac{\overline{u'w'}}{u_*^2} = \left(1-\frac{y}{h}\right) + b\frac{y}{h} + \frac{u\,w}{u_*^2} \tag{2}$$

where, u' and w' are velocity fluctuations in flow direction and vertical direction respectively, u_* is shear velocity, y is distance from the bed, h is flow depth and b is a parameter related to flow non-uniformity. b can be expressed as

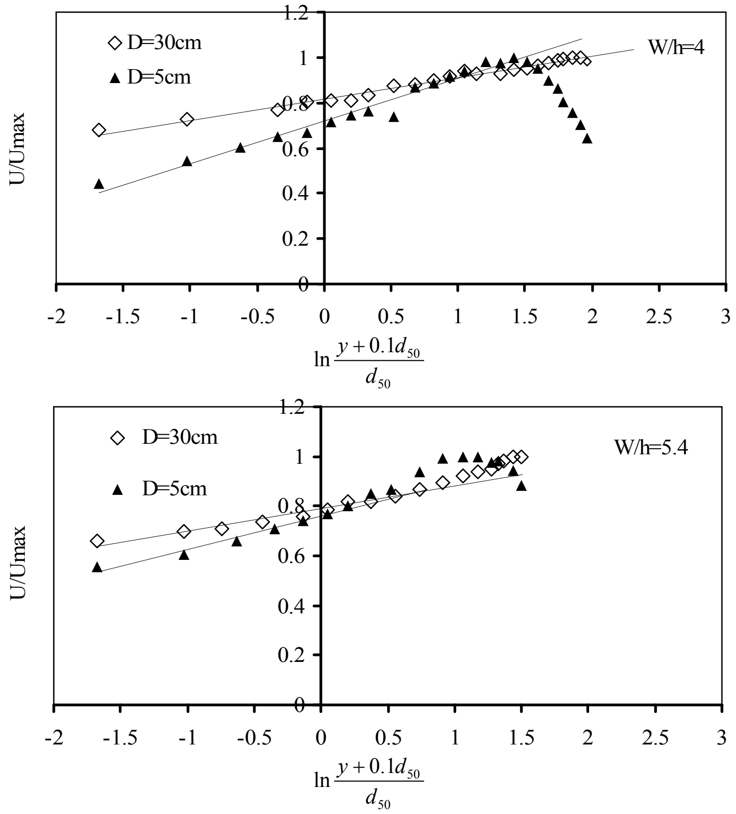


Fig. 7 Arithmic velocity profiles fit for "Run 2" (bottom) and "Run 3" (top) in laboratory

$$b = -\left(\frac{u_h}{u_*}\right)^2 \frac{dh}{dx} \tag{3}$$

where, u_h is flow velocity at water surface in the flow direction. Under condition of uniform flow, b=0. Consequently, Eq. 2 can be simplified as following

$$-\frac{\overline{u'w'}}{u_*^2} = \left(1-\frac{y}{h}\right) + \frac{u\,w}{u_*^2} \tag{4}$$

According to Eq. 4, non-zero velocity components may be responsible for the deviation of the Reynolds stress distribution from the linear one.

As shown in Fig. 9, there is a good agreement between our results and those determined by Eq. 4 developed by Yang et al. (2006). The deviation of the Reynolds stress distribution from the linear case in this study may be attributed to the effect of the vertical velocity component (w). On the other hand, as shown in Fig. 10, for flow condition of "Runs 2" and "Run 3" which have deeper flow depths and less W/h than those of "Run 1", the Reynolds stress distribution can be described by a polynomial of the second degree. This difference in Reynolds stress distribution under flow conditions of "Run 2" and "Run 3" from those under flow condition of "Run 1" may be attributed to increased effect of the secondary currents due to small aspect ratio. Under this flow condition, the maximum Reynolds stress is near the vegetated flume walls and closed to the channel bed at y/h=0.1. As pointed out by Yang et al. (2006), the "Zero Reynolds stress" is located at the position of $u_{\max}$. Therefore, the closer to the vegetated flume wall, the closer the position of $u_{\max}$ to the gravel channel bed, and the closer the position of zero Reynolds stress to the gravel channel bed.

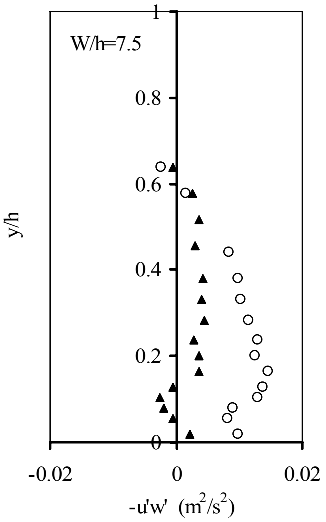


Fig. 8 Shear stress distribution for "Run 1" with W/h = 7.5 in laboratory

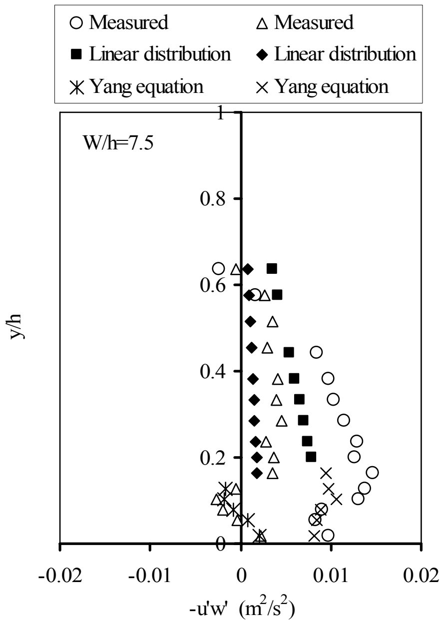


Fig. 9 Comparison between measured shear stress distribution and those calculated by Yang equation for "Run 1" with W/h = 7.5

3.3 Roughness coefficient

Hydraulic resistance of a natural channel with vegetated channel walls significantly affects the hydraulic slope of the flow in the channel. The resistance of a channel is commonly described by Manning's roughness coefficient (n), Chezy's resistance factor (C), or the Darcy-Weisbach friction factor (f), among which the Manning's n is widely used in the computation of open-channel flow (Chow, 1959). By using the boundary layer characteristics method developed by Afzalimehr and Anctil (2000), shear velocities have been calculated at first. Then, the Manning coefficient "n" was determined. The calculated Manning coefficient by using boundary layer characteristics method have been compared with that determined from the classic Manning equation. As shown in Fig. 11, the calculated Manning coefficients by using these two methods are in good agreement.

Results indicate that both the aspect ratio and the relative submergence also affect the Manning roughness coefficient. In this study, the impacts of the aspect ratio (W/h = 4–7.4) and the relative submergence (h/d_{50} = 4.6–8.6) on the Manning roughness coefficient have been investigated.

The depth averaged velocity has been calculated by using following Equation,

$$u = \frac{1}{h}\int_0^h u\,dh \tag{5}$$

The depth average velocity (u) has been weighted (u_m) by using three velocity profiles at each cross section. Similarly, the shear velocity (u_*) has also been weighted (u_{*m}) at each cross section. Consequently, Manning coefficient can be determined as following,

$$n = \frac{u_{*m}\left(\frac{g}{R_h^{1/3}}\right)^{1/2}}{u_m} \tag{6}$$

The shear stress has been calculated by the boundary layer characteristics method developed by Afzalimehr and Anctil (2000) as follow

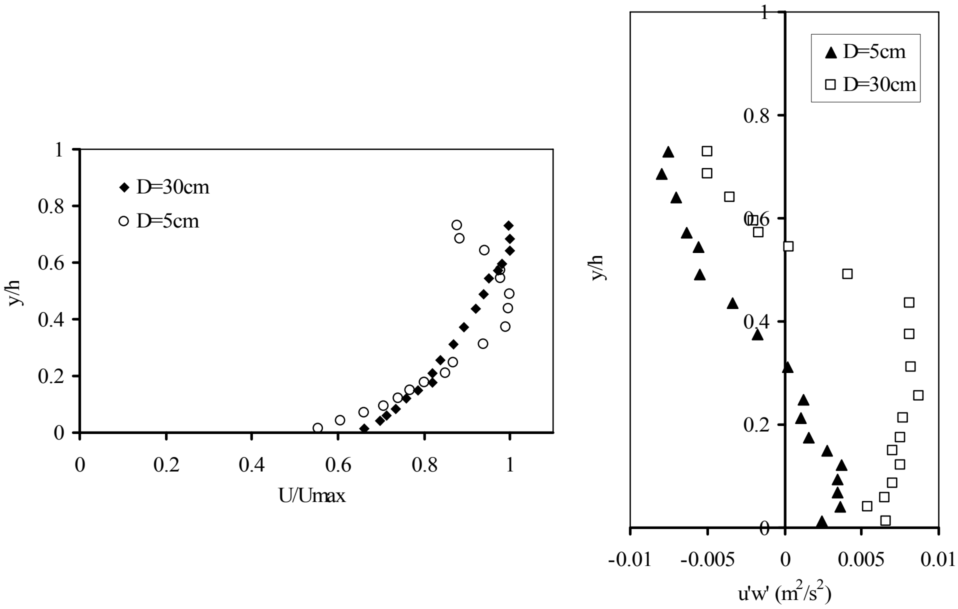


Fig. 10 Shear stress distribution (right) and velocity profile (left) for "Run 2" in laboratory

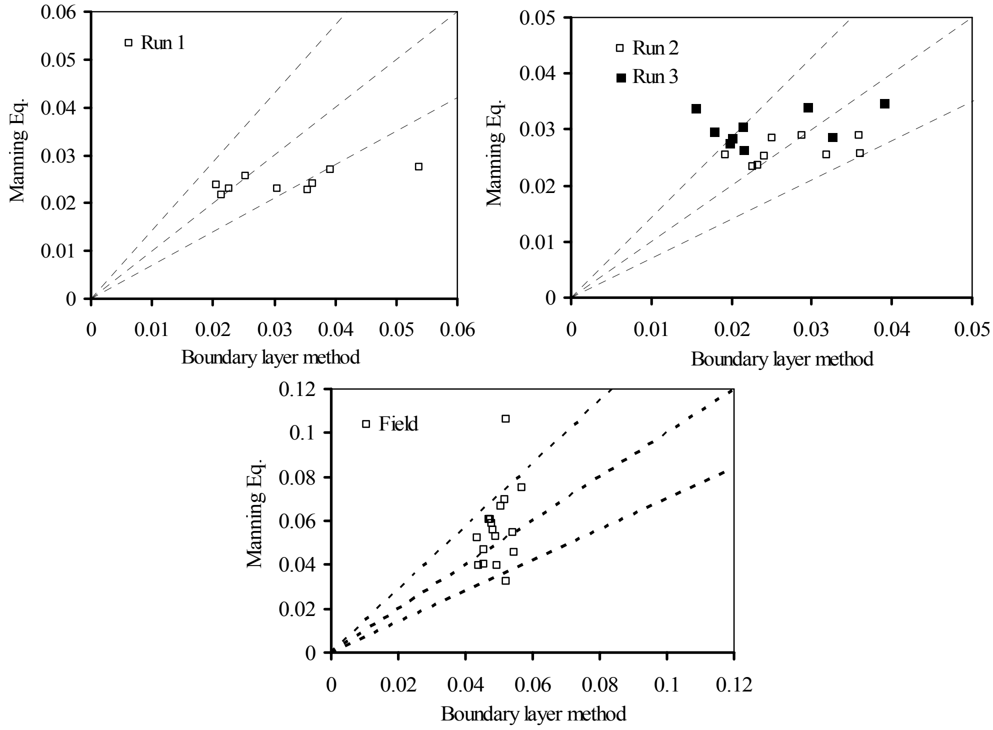


Fig. 11 Comparison of the values of "n" calculated by Manning equation with those determined by developed "boundary layer method"

$$u_* = \frac{(\delta_* - \theta)\, u_{max}}{c\,\delta_*} \tag{7}$$

where, δ_* is the boundary layer thickness, θ is the momentum thickness, u_{max} is the maximum velocity and c is a constant which depends on flow structure and roughness. Constant c in Eq. 7 has been suggested to be 4.8 for meandering river (Afzalimehr and Singh, 2009), and 4.4 for straight coarse-bed rivers (Afzalimehr and Rennie, 2009). To determine the constant "c" in Eq. 7, the shear velocity has been calculated at first by using Reynolds stress method (Kironoto and Graf, 1995). Then, constant d (Kironottfh4a63f

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