Jul 1st, 12:00 AM

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Abstract: In this paper a new method to experimentally evaluate roughness of vegetated river beds is proposed. Unlike the usual method of attaining uniform flow, the new method is based on measurements performed in boundary layer flows, conditions that in flow experiments with plants can be attained easier then in uniform flows. The described method can be applied either in case of smooth or vegetated walls. The friction factor f and Manning's n values so obtained are in agreement with literature results.

Keywords: flow resistance; uniform flow; boundary layer; vegetated river beds.

1. INTRODUCTION

In the past, vegetation on river beds was considered an unwanted source of flow resistance, and for this reason vegetation was commonly removed to improve the water conveyance.

Nowadays, vegetation is regarded as a means for providing stabilization for banks and channels, habitat and food for animals, and pleasing landscapes for recreational use. Therefore the preservation of vegetation is of great relevance for the ecology of water systems.

For this reason, the study of the effects of vegetation on the hydrodynamics of rivers represents one of the most important topics studied by hydraulic engineers today. Characteristics of flows over vegetated surfaces have been deepened with either experimental or numerical methods. Numerous studies have been carried out to examine the flow resistance of the streams on a vegetated bottom, and the main hydrodynamic characteristics of these streams, as the mean flow, the turbulent structures, and sediment transport (Lopez & Garcia 2001, Nezu & Onitsura, 2002, Tsuijimoto 1999, Tsuijimoto & Kitamura, 1990).

2. AIM OF THE PAPER

In a water current, vegetation may be regarded as a kind of bottom roughness that produces a resistance against the flow (Bettess 2003, Kouwen et al. 1969, Kouwen & Unny, 1973, Kouwen & Unny, 1981, Lopez and Garcia, 1997, Stone & Shen, 2002).

The main procedure to experimentally obtain the resistance characteristics of a bottom is to use a very long channel and generate within it a uniform flow. Therefore, long experimental laboratory channels must be realized in order to evaluate resistance characteristics of vegetated river bottoms.

Motivated by this observation, and by authors' experience of boundary layer flow over vegetated beds (De Felice and Gualtieri 2005, Gualtieri and Pulci Doria 2008), a simplified method is proposed that enables determination of the resistance coefficient by investigation of the boundary layer velocity profile. As the channel length necessary to produce a boundary layer is shorter, therefore, shorter laboratory channels are needed, and therefore this method allows easier determination of the resistance coefficient in a uniform flow.

Therefore, in this paper an experimental modelling of a particular kind of bottom roughness is described and a simplified method to evaluate the resistance coefficient is proposed.

3. GENERALITIES ABOUT RESISTANCE FORMULAS AND COEFFICIENTS

In a recent review (Yen, 2002) it is observed that each resistance coefficient can be considered either as cross section coefficient or as reach one, and that the usual resistance formulas can be considered as reach formulas to apply to uniform flows. It is possible to apply them also to not uniform flows, although to a very short reach, practically to a single cross section: therefore, in this case, the resistance coefficient can vary in the subsequent cross sections of the flow.

Referring to the reach formulas, the most frequently used, relating open-channel flow velocity V to resistance coefficients, are the Darcy-Weisbach, Manning and Chézy ones:

$$V = \sqrt{\frac{8g}{f}}\sqrt{RS}\,; \qquad V = \frac{1}{n}R^{2/3}S^{1/2}\,; \quad \mathrm{V} = \mathrm{C}\sqrt{\mathrm{RS}} \tag{1}$$

where f, n, and C are the Weisbach, Manning and Chézy resistance coefficients, and R = hydraulic radius, S = slope, g = gravitational acceleration. In case of cross section formulas, the slope S must be substituted by the head slope **J.** Comparing these formulas, it is possible to obtain the following expressions:

$$\sqrt{\frac{f}{8}} = \frac{n\sqrt{g}}{R^{1/6}} = \frac{\sqrt{g}}{C} \tag{2}$$

Among these resistance formulas, the authors choose the Darcy-Weisbach approach, because it is the most suitable for an exact evaluation of the flow resistances. Within this approach, the resistance coefficient f is related to the equivalent wall surface roughness, that here will be called ε, through the Colebrook-White formula:

$$\frac{1}{\sqrt{f}} = -K\,log\left(\frac{\varepsilon/4R}{a} + \frac{b}{Re\sqrt{f}}\right) \tag{3}$$

with Re Reynolds number defined as Re = 4VR/ν, and ν kinematic viscosity of the fluid.

The values of the constants a and b have been the object of many experimental surveys. In particular, in (Yen 2002) values for a and b, obtained by other researchers either for open channels with different aspect ratios or for wide open channels, are suggested. In Moody-type diagram, relative to circular full pipes, the values K = 2, a = 3.71 and b = 2.51 are used.

According to Marchi (1961) it is always possible to use the values of Moody-type diagram relative to circular full pipes also for open channels flows. This requires adoption of a suitable shape parameter ψ depending on the aspect ratio, so that the Colebrook-White formula becomes:

$$\frac{1}{\sqrt{f}} = -2\,log\,\frac{1}{\psi}\left(\frac{\varepsilon/4R}{3.71} + \frac{2.51}{Re\sqrt{f}}\right) \tag{4}$$

In particular, ψ assumes the value 0.83 for wide rectangular channels. The values of f so obtained are very similar to those suggested by Yen for wide channels.

4. PREVIOUS AUTHORS EXPERIMENTS ON BOUNDARY LAYER FLOWS

The elaborations performed in this paper are based on experimental data described in (De Felice and Gualtieri 2005, Gualtieri and Pulci Doria 2008).

The experiments were carried out in the plant sketched in figure 1. The main device was a channel 4m long and 15cm (b) wide, with variable slope, with plexiglas walls and bottom, coming out from a feeding tank supplied by a circulation pump, which took water from the drain tank downstream. The tank fed the channel through a rectangular adjustable sluice gate.

In the first sections of the channel, a zero piezometric head gradient boundary layer with horizontal free surface stream was generated. The boundary layer thickness increased in the subsequent sections along the channel, till it reached the same value as the height of the circulating flow, at a distance, from the inlet of the channel, depending on the dynamic characteristics of the flow itself (generally at least 50 cm).

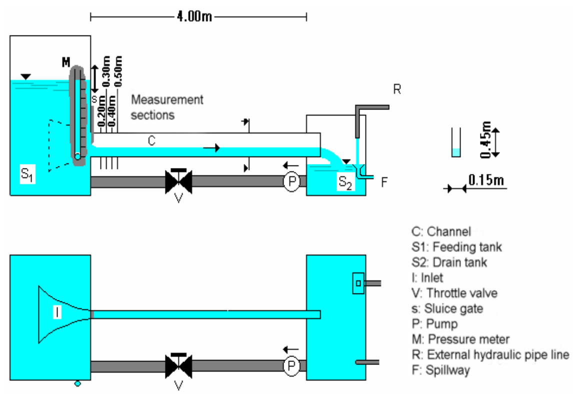

Figure 1. Scheme of the experimental plant.

Measurements were performed either with a smooth or a vegetated bottom. In particular, the vegetation was modelled by means of brass 4mm diameter cylinders with three different heights (5mm, 10mm, 15mm) placed according to two different regular geometries (respectively, a rectangular mesh $5*2.5cm^2$ and a square mesh $2.5*2.5cm^2$), pointed out synthetically as single and double density. Consequently, the projected area of vegetation per unit volume of water in the flow direction (Tsujimoto *et al.*, 1992) were, respectively, $3.2m^{-1}$ and $6.4m^{-1}$. Combinations of three different heights and two different densities produced six different vegetated bottoms.

In all the seven considered flow conditions (smooth bottom and six different vegetated bottoms), the same experimental setting was considered. In particular, the height of the sluice gate was set at 7.49cm so that the height in the vena contracta was at 4.62cm; the load on the vena contracta was at 10.34cm, and the resultant velocity of the free-stream was at 1.424 m/s.

Moreover, it was necessary to ensure the zero value of piezometric head gradient of the boundary layer, in each one of the seven considered flow conditions. This corresponded to hold the free surface of the current horizontal, at least in the first 50cm where the boundary layer developed. Therefore, it was necessary to suitably incline the channel, taking into account the vegetation type, whose possible increase in height and density would generate a corresponding increase of head losses, which would need to be balanced by a suitable increase of channel slope. The chosen slopes' values are reported in both the subsequent tables 3 and 4. The test sections were set at 20, 30, 40 and 50 centimetres from the channel inlet. In each test section two measuring verticals were considered, differently positioned with respect to the cylinders. The first one was set at the centre of either a rectangular or a square mesh. The second one was set along a cylinder row and at the centre of the lateral side of the same rectangular or square mesh. It is clear that in the case of a smooth bottom there was no need of such a second measurement location.

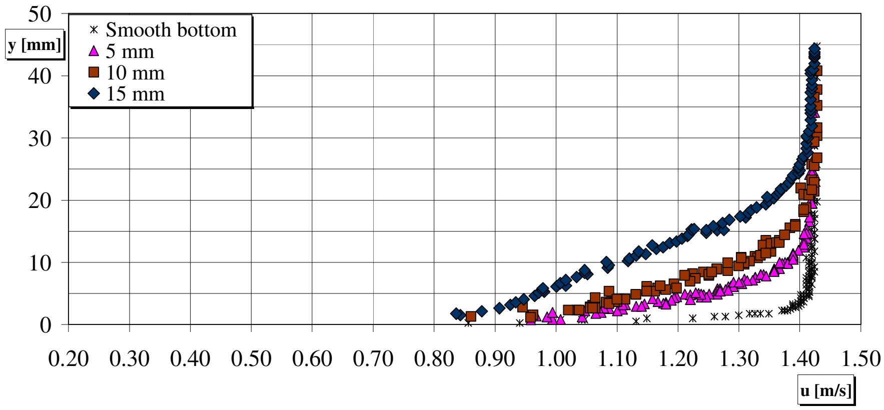

Figure 2. Local mean velocity distributions in test section n.1, central vertical (single density).

Either in the case of smooth or vegetated bottom, in each test section, along the chosen verticals, instantaneous velocities were measured in 20÷30 experimental points through a suitable LDA system, and in each point the local mean velocity was obtained.

Finally, in the figures 2 and 3, examples of the experimentally obtained local mean velocity distributions are reported.

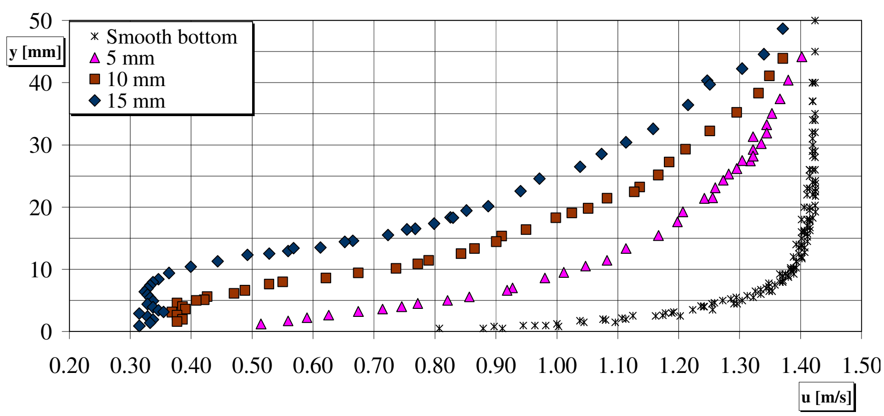

Figure 3. Local mean velocity distributions in test section n.4, lateral vertical (double density).

5. CONTROL OF FLOW RATE EXPERIMENTAL VALUES

Once the experimental local mean velocity distributions had been obtained, some preliminary elaboration of the data were carried out.

First of all, the local mean velocity distributions were completed through two further values of the velocity, respectively relative to the bottom and to the free surface, which could not be obtained directly through the LDA measurements. These values were defined through suitable extrapolation techniques.

The second preliminary elaboration was to control, in each test section and in each flow condition, the flow-rate experimental values per unit width q_u. This control was performed starting from the integral definition of the flow rate per unit width and transforming it in a summation:

$$q_u = \frac{Q}{b} = \int_h V dh \, . \cong \sum_{0}^{h\,max} V_{mean} \cdot \Delta h \qquad (5)$$

The summation was carried out with reference to each height step bounded by two following experimental points (included the bottom and the free surface ones) and the term V_{mean} represents the mean value of the superior and inferior velocity values in each summation step. The result of the summation represents directly the unit flow rates in case of smooth bottom, while in the cases of vegetated bottom instead, the unit flow rates were obtained for each vegetated bottom, through a mean between the results of the summation relative to central and lateral measurements.

The results of such a calculation are reported in the table 1. In the same table, the column of theoretical flow-rates represents the values computed directly multiplying the theoretical velocity in the vena contracta (1.424m/s) by the vena contracta cross section suitably reduced of the cross section relative to the cylinders. Moreover, also a column with the mean experimental flow-rates among the four test sections of each considered flow condition are shown.

In the last column of the table 1, the ratios between experimental flow rates and theoretical ones are reported. The maximum difference appears to be 2.5%: it can be attributed to experimental discrepancies. Therefore, the goodness of the fit between experimental and theoretical flow-rate values is sufficiently proved.

Flow-rate values are expressed per unit width, they show that the influence of lateral walls can be neglected similar to the wide channel case. Consequently in following calculations equation (4) with the Marchi's coefficient 0.83 will be considered.

Table 1. Experimental and theoretical unit flow-rates

		Experimental flow rates Q m^3/s				Theoretical flow-rates Q (m^3/s)	Experimental flow-rates means Q (m^3/s)	Ratio
		sec.1	sec.2	sec.3	sec.4			
Smooth bottom		0,00984	0,00988	0,00986	0,00983	0,00985	0,00985	1,000
Single Density	5mm	0,00981	0,00975	0,00974	0,00970	0,00968	0,00975	1,007
	10mm	0,00981	0,00974	0,00969	0,00975	0,00951	0,00975	1,025
	15mm	0,00958	0,00941	0,00931	0,00943	0,00934	0,00943	1,010
Double Density	5mm	0,00972	0,00966	0,00957	0,00956	0,00968	0,00962	0,994
	10mm	0,00952	0,00941	0,00924	0,00948	0,00951	0,00941	0,990
	15mm	0,00917	0,00895	0,00890	0,00939	0,00934	0,00910	0,975

6. BASIC METHODOLOGY TO OBTAIN THE f FRICTION FACTOR STARTING FROM BOUNDARY LAYER MEASUREMENTS

In the following paragraphs the methodology to obtain the f friction factor starting from boundary layer measurements will be described. In fact, as the available stream is a boundary layer one, the flow is not uniform and the f friction factor values that can be obtained are the cross section ones, and define a cross section f(s) function. In subsequent paragraph it will be shown how to obtain the uniform flow reach value of f friction factor. This one will be subsequently used to obtain the equivalent roughness ε and/or the Manning coefficient n of the considered kind of bottom.

Let us suppose to know the function f(s) relative to a certain kind of vegetated bottom and to the specific stream flowing over. Taking into account the circumstance that also the currents' height h(s) and the flow velocity $V(s) = q_u/ h(s)$ are functions of the same distance, it will be possible to write the Darcy-Weisbach equation (1) in the following way, where $\overline{V}$ is the mean velocity in cross section:

$$J(s)=\frac{f(s)}{4h(s)}\frac{\left[\overline{V}(s)\right]^2}{2g} \tag{6}$$

as in an wide rectangular cross section of height h the hydraulic radius coincides with h. Now it is also possible to write:

$$J(s)=-\frac{dH(s)}{ds}=-\frac{d}{ds}\left\{\alpha(s)\frac{\left[\overline{V}(s)\right]^2}{2g}\right\} \tag{7}$$

where α is the Coriolis coefficient, which in boundary layers is also a function of s. In fact, the hydraulic head H is the sum of the piezometric head and the kinetic head: the first one is constant in all our boundary layers currents as their free surface are always horizontal. Consequently, equations (6) and (7) give:

$$f(s)=-\frac{8gh(s)}{\left[\overline{V}(s)\right]^2}\frac{d}{ds}\left(\alpha(s)\frac{\left[\overline{V}(s)\right]^2}{2g}\right)=-\frac{4h(s)}{\left[\overline{V}(s)\right]^2}\frac{d}{ds}\left\{\alpha(s)\left[\overline{V}(s)\right]^2\right\} \tag{8}$$

Remembering now that $\overline{V}=q_u/h$ is the mean velocity in cross section and that consequently $\overline{V}$ is variable with s, whereas q_u is constant, rearranging of terms yields:

$$\begin{aligned} f&=-\frac{4h^3}{q_u^{\,2}}\frac{d}{ds}\left(\alpha\frac{q_u^{\,2}}{h^2}\right)=-4h^3\frac{d}{ds}\left(\alpha\frac{1}{h^2}\right)=-4h^3\left(\frac{1}{h^2}\frac{d\alpha}{ds}-\frac{2\alpha}{h^3}\frac{dh}{ds}\right)=\\ &=4h^3\left(\frac{2\alpha}{h^3}S-\frac{1}{h^2}\frac{d\alpha}{ds}\right)=8\alpha S-4h\frac{d\alpha}{ds} \end{aligned} \tag{9}$$

In previous equation, for sake of simplicity, the functional dependence on s has been discarded. Now, as it will be shown afterwards, a particular cross-section can be identified where the condition $d\alpha/ds = 0$ holds. Flow in this cross-section most closely resembles uniform flow (α=const). In this section we assume that the simplified resistance equation, $f = 8\alpha S$, is valid. The corresponding resistance coefficient f is assumed to be a representative reach-value of a uniform field, having the same flow rate and water depth as the aforementioned selected channel cross-section.

At this point the situation is the following one. The f friction factor value relative to the uniform flow can be computed through the simple formulation $f = 8\alpha S$ utilized in the suitable cross section where $d\alpha/ds = 0$. In order to obtain the unknown abscissa of this cross section and also the unknown value of α in the same cross section, it is necessary to known the analytical expression of the α(s) function. The value of α at the cross-section will be obtained by interpolation of α-values determined from velocity profiles in the test sections.

7. CALCULATION OF α VALUES, α(s) FUNCTIONS, AND f FRICTION FACTOR VALUES

In the case of smooth bottom, as the velocity is the same along any horizontal line of the cross section, the expression of α can be easily transformed in an equation holding integrals of only one parameter (instead of surface integrals) which at last can be computed in the same way that in equation (5), i.e.:

$$\alpha = \frac{\int_0^{h\,max} V^3 dh}{h\overline{V}^3} = \frac{\sum_0^{h\,max} \frac{(V^i{}_c{}^3 + V^{i+1}{}_c{}^3)}{2} \cdot \Delta h}{h\overline{V}^3} \tag{10}$$

where V^i_c is the measured velocity at measurement point i.

In contrast, in the case of our model of vegetation, the flow velocity is variable along any horizontal line of the cross section, and therefore α can not be calculated as easily. To solve this difficulty, we make use of the available velocity distributions in the test sections in the following way.

First of all, if we consider any horizontal line in a cross section, it happens that the horizontal velocity distribution has an oscillating behaviour between a minimum value corresponding to lateral measurements and a maximum value corresponding to central measurements. Among the possible functions, the sine function is the most suitable that can represent this oscillating behaviour. Consequently, we can start from this equation:

$$\alpha = \frac{\int_0^{h\,max} \overline{V^3_{mean}} dh}{h\overline{V}^3} \tag{11}$$

where the symbol $\overline{V^3_{mean}}$ is defined as the cubic mean of the velocity values along an horizontal line. In order to compute this term taking into account a sinusoidal velocity distribution between a minimum value (V_l) and a maximum value (V_c), it is necessary to start from the following representation of the horizontal velocity distribution: $V = V_0 + \Delta V \sin x$, where:

$$V_0 = \frac{V_c + V_l}{2} \qquad \Delta V = \frac{V_c - V_l}{2} \tag{12}$$

Consequently, it is possible to write in the generic point of the considered horizontal line:

$$\begin{aligned} \overline{V^3_{mean}} &= \frac{1}{2\pi}\int_0^{2\pi} (V_0 + \Delta V \sin x)^3 dx = \\ &= \frac{1}{2\pi}\int_0^{2\pi} \left(V_0^3 + 3V_0^2 \Delta V \sin x + 3V_0 \Delta V^2 \sin^2 x + \Delta V^3 \sin^3 x\right) dx = V_0^3 + \frac{3}{2} V_0 \Delta V^2 \end{aligned} \tag{13}$$

Substituting now equation (12) in equation (13), it is possible to obtain, through simple calculations:

$$\begin{aligned} \overline{V^3_{mean}} &= V_0^3 + \frac{3}{2} V_0 \Delta V^2 = \left(\frac{V_c + V_l}{2}\right)^3 + \frac{3}{2}\left(\frac{V_c + V_l}{2}\right)\left(\frac{V_c - V_l}{2}\right)^2 = \\ &= \frac{V_c{}^3}{2} + \frac{V_l{}^3}{2} - \frac{3}{16}(V_c - V_l)\left(V_c{}^2 - V_l{}^2\right) \end{aligned} \tag{14}$$

Now, this expression must be inserted in equation (11) and this one must be suitably integrated obtaining:

$$\alpha = \frac{\int_0^{h\,max} \overline{V^3_{mean}} dh}{h \cdot \overline{V}^3} = \frac{\int_0^{h\,max} \frac{V_c{}^3}{2} dh + \int_0^{h\,max} \frac{V_l{}^3}{2} dh - \int_0^{hmax} \frac{3}{16}(V_c - V_l)\left(V_c{}^2 - V_l{}^2\right) dh}{h \cdot \overline{V}^3} \tag{15}$$

Finally, we transform integrals in summations obtaining:

$$\alpha = \frac{\sum_{0}^{h_{\max}} \frac{(V^{i}{}_{c}{}^{3} + V^{i+1}{}_{c}{}^{3})}{2} \frac{\Delta h_i}{2} + \sum_{0}^{h_{\max}} \frac{(V^{i}{}_{l}{}^{3} + V^{i+1}{}_{l}{}^{3})}{2} \frac{\Delta h_i}{2} - \sum_{0}^{h_{\max}} \eta_j \Delta h_j}{h \cdot \overline{V}^3} \tag{16}$$

with:

$$\eta_j = \frac{3}{16}\left(V_{cj} - V_{lj}\right)\left(V_{cj}{}^2 - V_{lj}{}^2\right) \tag{17}$$

where j represent the generic height between two following measurement points. Finally, equations (16) and (17) allow calculation of α-values in all the test sections of all the seven considered flow conditions. These values are reported in table 2.

The values from the table 2 are also presented in Figure 4. In this figure it is shown that the experimentally obtained α values can be interpolated through second order polynomials which start all from the same s_0 section placed at about 0.10m from the leading edge of the boundary layer. Up to this section, the α-value may be considered equal to unity.

Table 3 shows the results of the aforementioned analysis: the second order polynomial curves coefficients a, b, c; the slope of the current S; the abscissa s_0 of the test section where dα/ds = 0; the height of the test section where dα/ds = 0.

Table 2. Coriolis α values for every flow conditions and in each test section

		Final α values			
		sec.1	**sec.2**	**sec.3**	**sec.4**
Smooth bottom		1,0081	1,0069	1,0127	1,0161
Single Density	**5mm**	1,0289	1,0416	1,0507	1,0590
	10mm	1,0470	1,0790	1,0998	1,1132
	15mm	1,0884	1,1411	1,1796	1,1833
Double Density	**5mm**	1,0343	1,0369	1,0458	1,0402
	10mm	1,0743	1,1027	1,1298	1,1071
	15mm	1,1371	1,1890	1,2178	1,1749

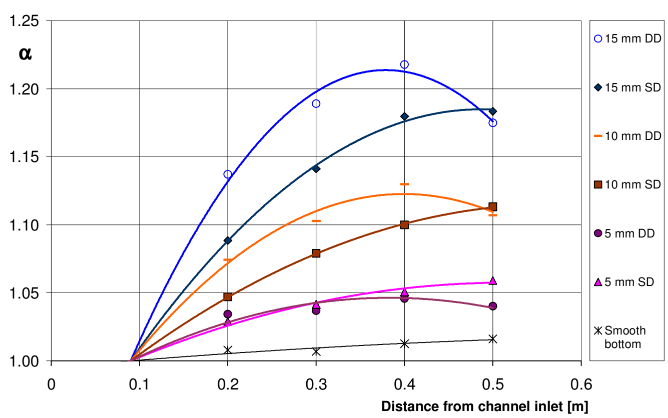

Figure 4. Values of parameter α and interpolating functions

Once the maximum values of α for each flow condition have been obtained, it is finally possible to use equation (9) to obtain the f values relative to the seven different considered bottoms. The results of such calculations are reported also in table 3.

Table 3. Interpolating curves coefficients for α values, consequent maximum values of same α and f correspondent values for each considered bottom.

		$\alpha=as^2+bs+c$ a	b	c	S	s_0 [m]	h_0 [m]	α $[s=s_0]$	f=8Sα $[s=s_0]$
Smooth bottom		-0,03630	0,05930	0,99500	0,0025	0,817	0,04814	1,01922	0,02038
Single Density	5mm	-0,31910	0,32820	0,97310	0,0092	0,514	0,05083	1,05749	0,07783
	10mm	-0,49900	0,55930	0,95280	0,0160	0,560	0,05507	1,10952	0,14202
	15mm	-1,17120	1,14140	0,90680	0,0227	0,487	0,05716	1,18489	0,21518
Double Density	5mm	-0,54110	0,41410	0,96710	0,0115	0,383	0,05050	1,04633	0,09626
	10mm	-1,28530	1,02520	0,91820	0,0205	0,399	0,05428	1,12263	0,18411
	15mm	-2,56320	1,94120	0,84610	0,0295	0,379	0,05727	1,21363	0,28642

8. FINAL CALCULATIONS OF n AND ε VALUES

Finally, starting from the obtained f values, and employing the aforementioned methodologies, it is possible to obtain both the n Manning coefficients and the equivalent sand roughness ε corresponding to the seven considered bottoms. In particular the procedures are the following ones.

For the n coefficients, it is simple to use equation (2). For the ε values it is necessary to invert equation (4), with the shape parameter $\psi = 0.83$. The results of such calculations are reported in table 4. For the smooth bottom case, the value of ε = 0.14mm, and Manning's n = 0.0097 are consistent with values for plexiglass roughness cited in literature (Ven te Chow, 1988). As expected, both the values of ε and n in case of vegetated bottom follow a clear trend of growth either with increase of density or with increase of cylinders height.

Table 4. Bed's slopes and final f values and for each flow condition, and consequent equivalent sand roughness and n Manning parameter.

		f=8αi $[s=s_0]$	S	h_0 [m]	$\varepsilon/4h_0$ $[s=s_0]$	ε $[s=s_0]$	n
Smooth bottom		0,02038	0,0025	0,04814	0,000720	0,000139	0.00972
Single Density	5mm	0,07783	0,0092	0,05083	0,049685	0,010102	0,01917
	10mm	0,14202	0,0160	0,05507	0,145103	0,031961	0,02624
	15mm	0,21518	0,0227	0,05716	0,257374	0,058847	0,03250
Double Density	5mm	0,09626	0,0115	0,05050	0,075319	0,015215	0,02129
	10mm	0,18411	0,0205	0,05428	0,210463	0,045692	0,02980
	15mm	0,28642	0,0295	0,05727	0,358251	0,082069	0,03751

Finally, in figure 5, the obtained roughness of each bottom is reported as a function of the cylinders height and density, including also the case of smooth bottom. The experimental points are very well aligned along straight lines which join themselves in a single point on the abscissa axis. Probably the ε value is almost zero if the cylinders height is no more than 2 or 3mm, due to the presence of the laminar substratum. The two straight lines show an increasing trend with the cylinders height, more accentuated in case of double density with respect to single density. Besides the 3mm cylinders' height ε values can be computed, in relation to the two vegetation densities, through the formulas reported into the diagram.

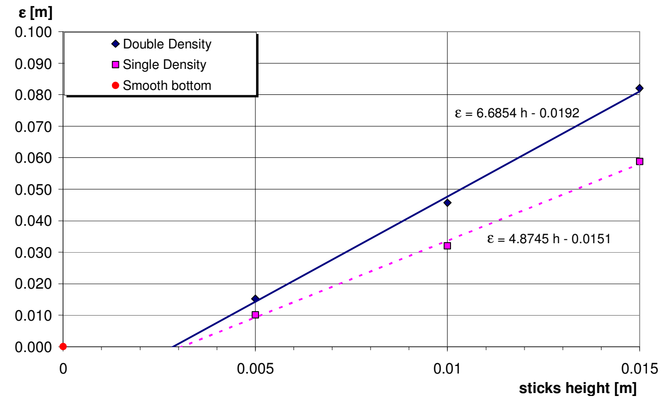

Figure 5. Roughness values of the different bottom and their interpolating functions.

9. COMPARISON TO DATA FROM LITERATURE

As it has been already stressed, the very little value of roughness obtained in the case of smooth bottom represents a first confirmation of the validity of the considered procedure. But, in any case, another literature control has been performed with the Lopez and Garcia experimental data (1997). In particular, to perform such a comparison, a graph of Lopez and Garcia has been considered. In this graph, in the abscissa, the projected area of vegetation per unit volume of water in the flow direction and in the ordinate the Manning's n values, are reported. In the cases of this papers' measurements, the projected area of vegetation per unit volume of water in the flow direction parameter is worth $3.2m^{-1}$ in the case of single density and $6.4m^{-1}$ in the case of double density. Consequently, six experimental points have been obtained, relative to both two densities' values and three cylinder' heights. For each one of these six different conditions, the values of the Manning's n reported in table 4 have been represented through a point into the Lopez and Garcia graph, so obtaining the final sketch of figure 6.

The obtained results are qualitatively comfortable. The Manning' s n value depends mainly on the bottom roughness, so that, with equal cylinders density, the n coefficient must grow with the height of the cylinders. Figure 6 shows that for both cylinders densities, higher cylinders yield higher Manning's n values, and that the obtained roughness values from the current study remain lower than the Lopez and Garcia interpolating line which corresponds to 10cm cylinder height. Referring to the flow depth, the use of equations (2) and (4) can show that for a given ε value, the Manning's n value must slowly decrease with it. Now, the flow in (Lopez and Garcia, 1997) is deeper than in authors experiments; consequently, the effect of the flow depth would be to shift the Lopez and Garcia line from the authors points.

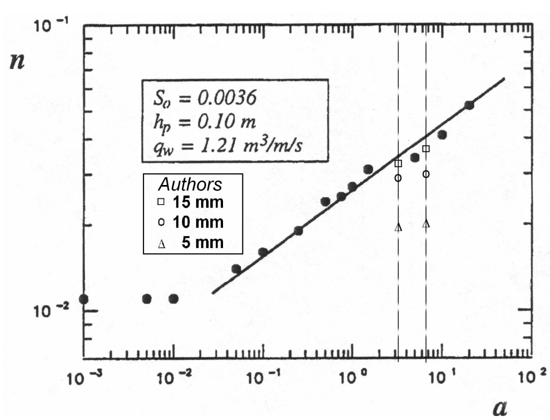

Figure 6. Comparison between Lopez and Garcia data and authors data

10. CONCLUSIONS

The aim of this paper was to present a simplified procedure of determining the resistance coefficients of water currents, and in particular of vegetated bottom current. The proposed procedure allows the estimation of the f friction factor of a uniform flow through experimental measurements carried out on a boundary layer stream. Consequently, instead of requiring a very long experimental channel, the whole procedure can be carried within the boundary layer of the flow in a shorter experimental channel and the results can be extrapolated to the uniform flow.

The roughness values obtained with the described procedure are reasonable either in case of smooth or vegetated bottom.

In a historic period in which vegetated bottoms will attain more and more importance in engineering practice, the possibility of obtaining such results through the use of simplified experimental facilities can play a positive role in the field of environmental hydraulics.

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