Slit-Check Dams for Controlling Debris Flow and Mudflow

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Abstract

The paper presents a rational criterion to design the slit check dams to control mudflows and debris flows. The method here presented is based on the scheme proposed by Armanini & Larcher (2001) for the case of ordinary sediment transport and it is based on the balances of the solid and liquid phases of debris flow and mudflow and on momentum conservation. The theory is compared with the results of some laboratory tests on scale reduced models of debris flows and of mudflows. The experimental results agree quite well with the predictions of the theory.

Keywords: slit check dam, debris flow, mudflow, defence works.

Introduction

Slit check dams are open check dams presenting one or more narrow, vertical openings, going from the dam's base up to the weir Their purpose is mostly to dose the sediment transport rate.

Although slit check dams have been used for many years in torrent restoration as a primary device to obtain a more effective sediment transport control, many aspects of the functioning of these devices are still not sufficiently clear. From a hydraulic point of view, open check dams are very often designed only on the basis of the designer's experience, who imitates similar structures built in analogous situations.

In the active defence strategy against debris flow and mudflows, open check dams are widely employed with the double purpose to reduce the kinetic energy of the flow (debris flow breakers) and to temporary store the debris/mud volume. The first purpose is the target of the check dams located in the upper part of the basins where the available volumes are small: in this case the dynamic impact is the most important design parameter. In the second case the reference parameter is volume retention. The check dams of this type are built in the lower part of the basins, often at the exit of the stilling basins.

A rational criterion for designing the opening of slit check dams to control the bed load and the suspended load has been proposed by Armanini & Larcher (2001). The approach is based on the conservation of the mass of water and sediments and on the energy balance between an upstream section and the section of the opening of the check dam. In the present paper the authors extend the scheme, presenting a theoretical approach to the problem of designing the opening of a slit check dam when the solid transport rate exceeds the typical limits of ordinary sediment transport. In particular two different types of flow are considered, characterized by markedly different features: debris flow and mudflow. In the case of granular debris flow, the interstitial fluid is represented by water and the effects of cohesion can be considered negligible. On the contrary, the mudflow is represented by a homogeneous mixture composed by water and by the finer fraction of sediments with cohesion and yield stress that sharply affect the flow properties.

Effect of a width contraction in a debris/mud channel flow

From the analysis of the mass and momentum conservation equations, it is possible to establish that in steady flow condition, upstream a slit check dam (that is upstream a contraction of the channel cross section) a permanent deposit takes place (Armanini and Larcher, 2001). If the channel is prismatic the flow run in uniform condition over the deposit, with a linear profile of the free surface and of the bed characterized by an equilibrium slope, as shown in Fig.1.

In order to describe the main features of the deposition process, it is useful to assume a long prismatic channel, with a rectangular cross section. Over the deposit the flow occurs in equilibrium with the underlying sediment bed, whereas the upstream reach of the channel is characterized by a slope angle α_0 generally steeper that the deposit slope angle α (Larcher, 2004; Armanini et al., 2005).

While this assumption is quite immediate in case of debris flow, because of its attitude to deposit, the

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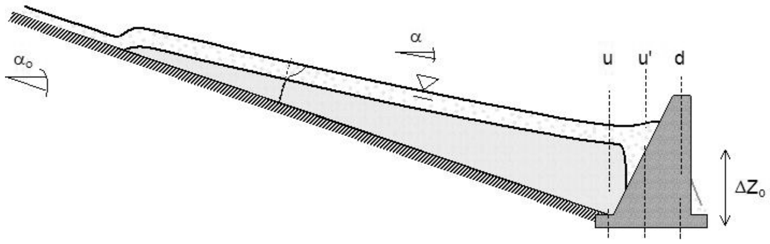

Fig. 1. Steady-state configuration of the flow upstream a slit dam.

same is not obvious for the mudflows. In fact, although the matter has been subjected to several experimental investigations (among the others, O'Brien & Julien, 1988; Major & Pierson, 1992; Coussot, 1997), the issue cannot be considered as completely settled.

In many applications, mudflows are treated as a Bingham or Herschel-Bulkley homogenous fluids. This approach prevents the possibility of analyzing separately the behaviour of the solid phase and the one of the interstitial fluid, and erosional and depositional mechanisms at the bed cannot be foreseen. Within this frame, equilibrium conditions (deposit upstream the check dam) are supposed not to take place and mudflow is always treated as a rigid-bed flow. Within the mudflow, the finer fractions of the grain distribution are associated to water in forming the interstitial fluid. In fact, the role of fine sediments, because of their dimensions and, in some cases, of their electrochemical properties, may be different from the role of the coarser ones: they can perfectly be mixed with water, forming a homogeneous interstitial fluid, typically characterized by high viscosity and cohesive behaviour (yield stress). According to recent rheological findings (Quemada, 1998), the fine sediments can built up *structural units* that are dispersed in the laminar flowing medium. The structural units are subjected to *viscous resuspension* (Schaflinger et al., 1990, Armanini et al., 2003). This new concept has been adopted to better characterizing the dynamics of mudflows and viscous debris flows in laminar conditions. Usually, the dispersed phase is considered neutrally buoyant in the fluid whereas in our experiments, as in many natural phenomena, such as mudflows and viscous debris flows, and industrial processes, heavy particles immersed in a lighter fluid bear both a shear-induced particle-migration and a flux due to gravity. When these two mechanisms balance each other, the particles are steadily suspended.

If the channel slope is great enough, to allow a flow regime at which it corresponds a transport capacity larger than the sediment discharge fed from upstream, we observe the formation of a deposition layer on the channel bed and a new slope (smaller than the channel slope) such that the new transport capacity exactly balances sediment discharge. The new bed and the new free surface slopes are independent of the channel bed. It is obvious that in these cases the flow tends to become asymptotically uniform. In these equilibrium conditions, the velocity (and its gradient normal to the mean flow direction) decrease with continuity to zero as the mobile bed level is approached.

As reported in Fig. 2, an equilibrium layer on the bottom is formed, over which the suspension mixture flows. Particle concentration close to the bottom reaches its maximum values and the effective suspension viscosity becomes infinite. Near to the free surface, where the shear stress tends to vanish, an undeformed plug layer has been observed. The plug layer has to be always expected where the shear stress becomes smaller than critical yield stress.

It is interesting to note that a similar event occurs in the horizontal planes also just upstream the contraction induced by the slit dam: in this case however the plug forms laterally.

In other words, the flow of mud through a sharp narrowing (Fig. 3) develops, contrarily to water flow, constant rigid zones in the corners (called *dead zones*) in which the induced shear stress is smaller than yield stress and flow velocity is zero. By this rheological mechanism the initial sharp narrowing becomes, in stationary condition, gradual.

For quantifying the gradual self-formed narrowing, some series of experiments were carried out for different values of solid concentration and width of slit and the results are summarized in Figure 4. B and Y are defined in eq. (1) in which B_0 is the channel width, b is the slit width and $B(x)$ is the local width of the natural gradual narrowing at a distance x from the slit (x_0 is the total length of the gradual narrowing).

$$B = \frac{B(x) - b}{B_0 - b} \qquad Y = \frac{x}{B_0 - b} \tag{1}$$

The development of the *dead zones* affects the morphology of the mobile bed near the narrowing, in fact respect to granular flow, the deposit tends to disappear near the narrowing where the dead zone has formed.

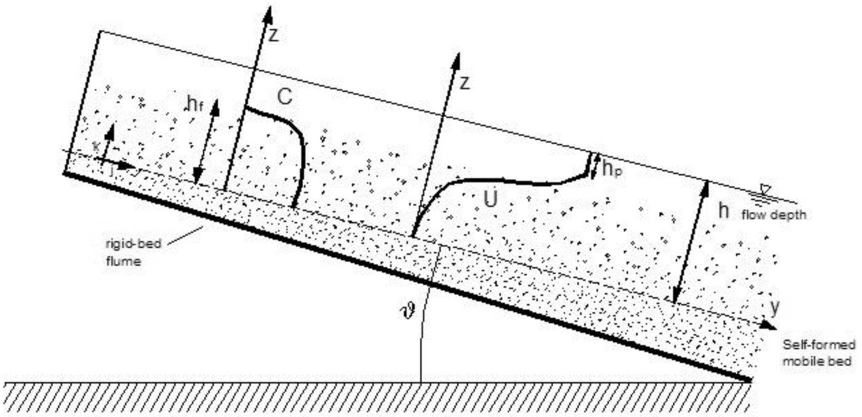

Fig. 2. Schematic diagram of the free surface mudflow.

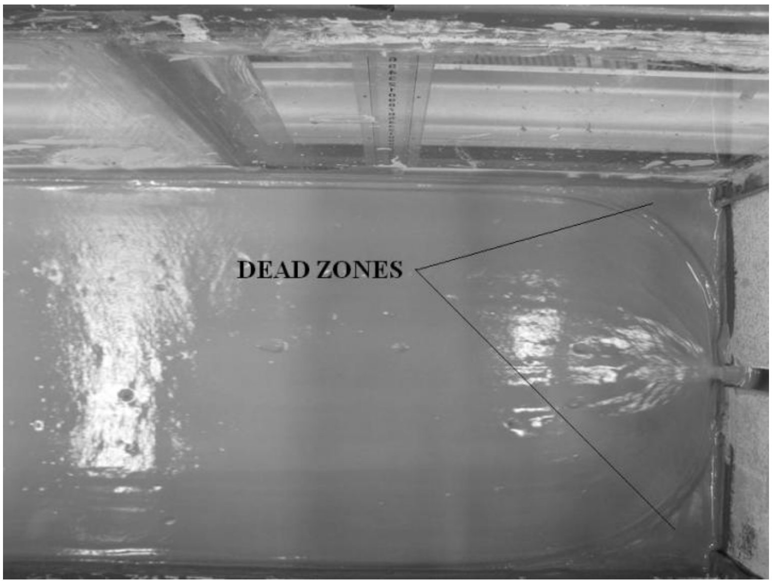

Fig. 3. Picture plan view of a Mudflow through slit check dam (schematizing as a sudden channel narrowing).

The profile of the mobile bed within the natural gradual narrowing can be obtained solving the following system of equations that represent the conservation of liquid and solid discharge and of energy:

$$\begin{cases} \dfrac{\partial}{\partial x}\left(B(x)Uh\right) = 0 \\ \dfrac{\partial}{\partial x}\left(\alpha B(x)U^n\right) = 0 \\ \dfrac{\partial}{\partial x}\left(h + z + \dfrac{U^2}{2g}\right) = 0 \end{cases} \tag{2}$$

In the set of equations (2), U is the average velocity of the mudflow, h is the depth of flow, z is the height of the mobile bed and n is the exponent of the mudflow sediment transport law (value usually included between 1.1-1.2). Expressing the shape of the gradual self-formed narrowing by a power law (eq. 3), the profile of mobile bed can be obtained from the solution of the equation (4):

$$B(x) = B_0 - (B_0 - b)\left(\frac{x}{x_0}\right)^m \tag{3}$$

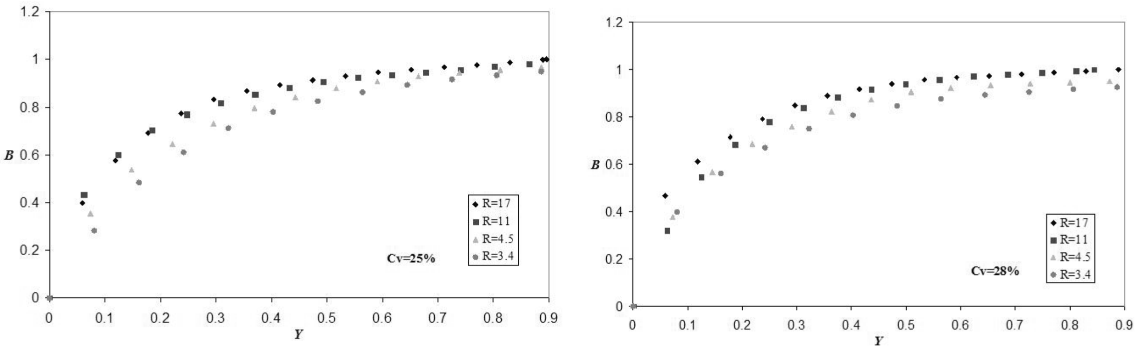

Fig. 4. Experimental behaviour of the normalizing constriction of the net section B(x) in flow direction for different values of restriction ratio R=B0/b.

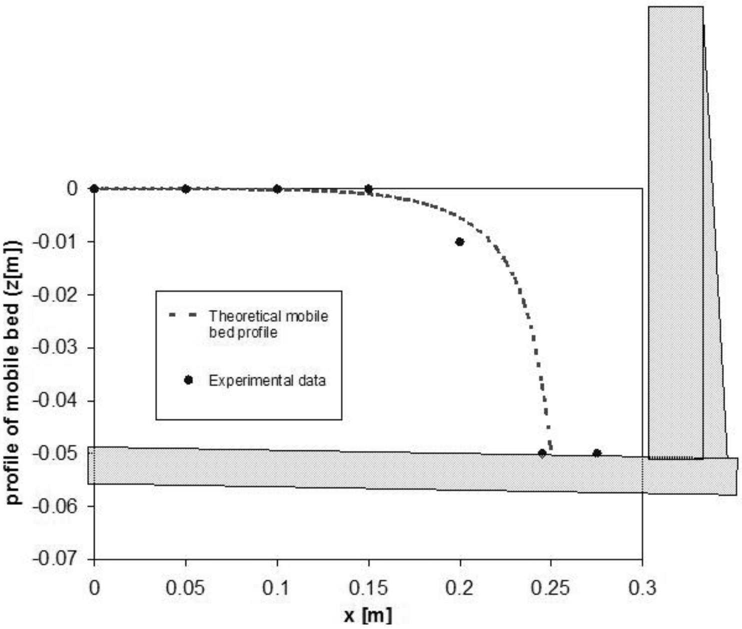

Fig. 5. Comparison between theoretical and experimental bed profile (Bo=34 cm, b= 3 cm, x0=27 cm, m=5.5, n=1.17, Q=3.6 l/s).

$$\frac{\partial z}{\partial x} + m(b-B_0)\left(\frac{x^{m-1}}{x_0^m}\right)\left[\begin{array}{c} h_v b^{\frac{n-1}{n}}\frac{1-n}{n}\left(B_0-(b-B_0)\left(\frac{x}{x_0}\right)^m\right)^{\frac{1-2n}{n}} + \\ -\frac{Q^2}{g h_v^2 b^{\frac{2n-2}{n}}}\left(B_0-(b-B_0)\left(\frac{x}{x_0}\right)^m\right)^{\frac{-2-n}{n}} \end{array}\right] = 0 \qquad (4)$$

Fig. 5 contains the comparison between the theoretical self-formed bed profile and the experimental one ($C_v = 25\%$, $B_0 = 34$ cm, $b = 3$ cm, $x_0 = 27$ cm, $m = 5.5$ and $Q = 3.6$ l/s) and they matched quite well. The experimental data show clearly that the deposit disappears near the slit-check dam and also for mudflow the erosion-depositional mechanisms can not be neglected for a correct design of these structures.

Slit check dams for debris flows and mudflows

In case of ordinary sediment transport, the flow regime in the slit occurs according to two different schemes (Armanini & Larcher, 2001): if the slit is relatively wide, the critical velocity inside the restriction is too small in order to guarantee the conservation of the sediment discharge through the dam, therefore a supercritical flow takes place in the slit; on the contrary, if the slit is narrow enough, the critical velocity in

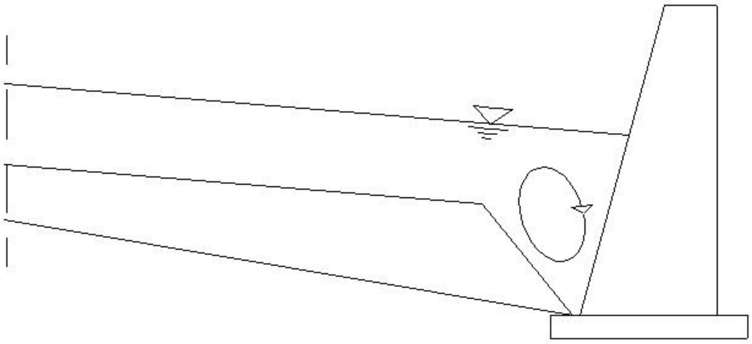

Fig. 6. Scheme of the vertical vortex upstream the check dam.

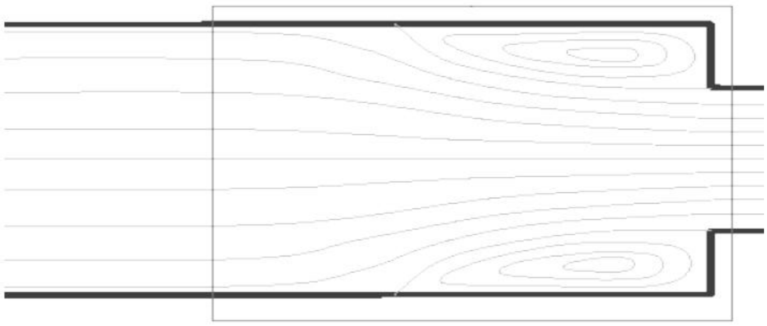

Fig. 7. Scheme of the recirculations upstream the check dam.

the slit is higher than the transport velocity (the uniform flow velocity corresponding to the equilibrium slope) and in the slit there will be a critical flow condition (Froude number = 1). In this case the solid discharge is smaller than the transport capacity, but is conserved assuming that the bed inside the slit is non-redouble.

A series of experiments has been performed in order to check analogies and differences between the ordinary sediment transport-case and the debris flow-case. The experiments have been carried out in a flume with the length of 6 m and the width of 10 cm, endowed with a recirculating system for the grain-liquid mixture. The check dam was placed at the end of flume. Six different slit widths have been tested: 1.0, 1.5, 2.0, 2.5, 3.0 and 4.0 cm. The solid phase was constituted by well sorted sand characterized by a mean diameter of 1.9 mm, a density of $\rho_s = 2932$ kgm^{-2} and a friction angle $\phi = 36°$C. The interstitial fluid was water. This facility allows quite simply to get the required steady-uniform conditions in a sufficiently long reach.

In all the experiments inside the slit the critical flow conditions occurred, therefore the analysis was restricted to the narrow-slit case defined by Armanini & Larcher (2001) for ordinary sediment transport.

In this situation the depth h_s and the average velocity U_s of the flow inside the slit depend just on the total discharge (Henderson, 1966) and they can be expressed as:

$$h_s = \sqrt[3]{\frac{Q^2}{gb}} \qquad U_s = \sqrt[3]{\frac{Qg}{b^2}} \tag{5}$$

in which Q is the discharge and b is the width of slit.

The above equations represent the critical conditions, corresponding to the minimum of the energy, and are widely used for pure water, but it is easily to demonstrate that they can be extended also to debris flows.

Moreover, the mass conservation provide the relation between the discharge and upstream velocity, $Q = B_o U_u h_u$, where B_o, U_u and h_u are respectively the width, the mean velocity and the flow depth in the upstream undisturbed section 'u' (Fig.1).

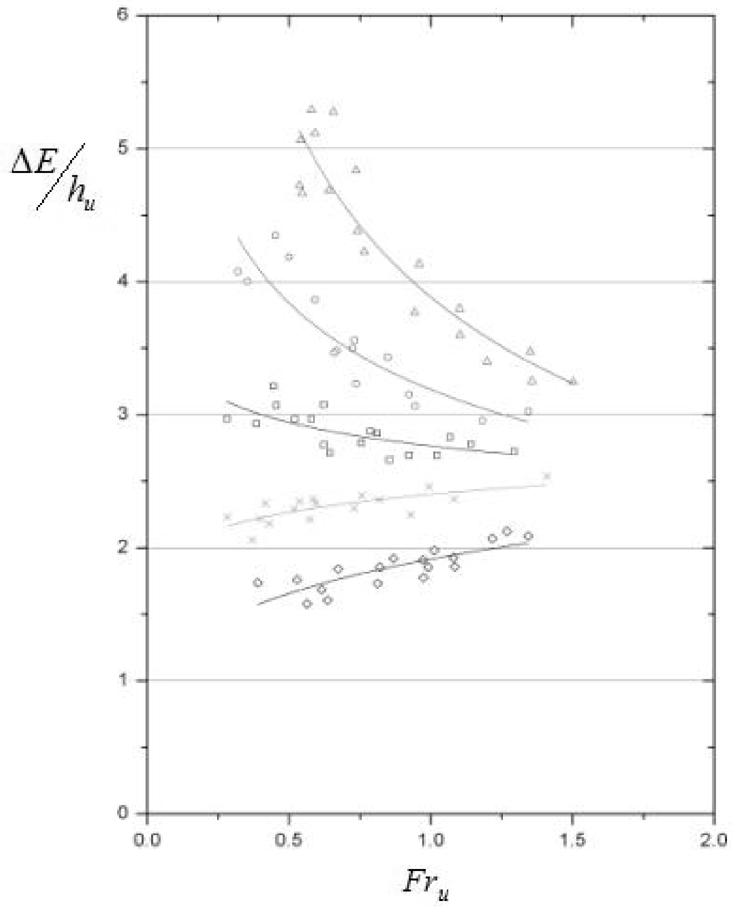

Fig. 8. Total energy dissipation.

The energy balance between the upstream section 'u' and the slit section provides the following relation:

$$\Delta z_0 + h_u + \frac{U_u^2}{2g} = h_s + \frac{U_s^2}{2g} + \Delta E \tag{6}$$

in which Δz_0 is the height of the deposit in the undisturbed section 'u' upstream the dam and ΔE is the loss of energy due to the flow contraction upstream the slit.

The combination of the two equations gives the height of the deposit in the flowing compact form:

$$\frac{\Delta z_0}{h_u} = \frac{3}{2}(Fr_u R)^{2/3} - 1 - \frac{Fr_u^2}{2} + \frac{\Delta E}{h_u} \tag{7}$$

in which $R = B_0/b$ is the restriction ratio and $Fr_u = U_u/(gh_u)^{0.5}$ is the Froude number of the undisturbed upstream flow.

The energy loss ΔE is one most critical parameters of the above equation. The energy loss occurs because of the presence of a strong contraction induced by the slit. In the case of ordinary sediment transport (bed load or suspended load) this effect can be ascribed to the vertical vortex that forms between the downstream end of the deposition and the check dam, as represented in Fig. 6.

This energy loss was analytically derived, and experimentally verified, by Armanini & Larcher (2001), that obtained the following expression:

$$\frac{\Delta E}{h_u} = \frac{Fr_u^2}{2}\left[1 - \frac{2}{3}\left(Fr_u\ R\right)^{-2/3}\right]^2 \tag{8}$$

During the laboratory investigation with debris flows another important dissipative effect has been observed, this is the loss energy induced by the formation of two symmetric recirculation zones upstream the check dam, as represented schematically in Fig. 7.

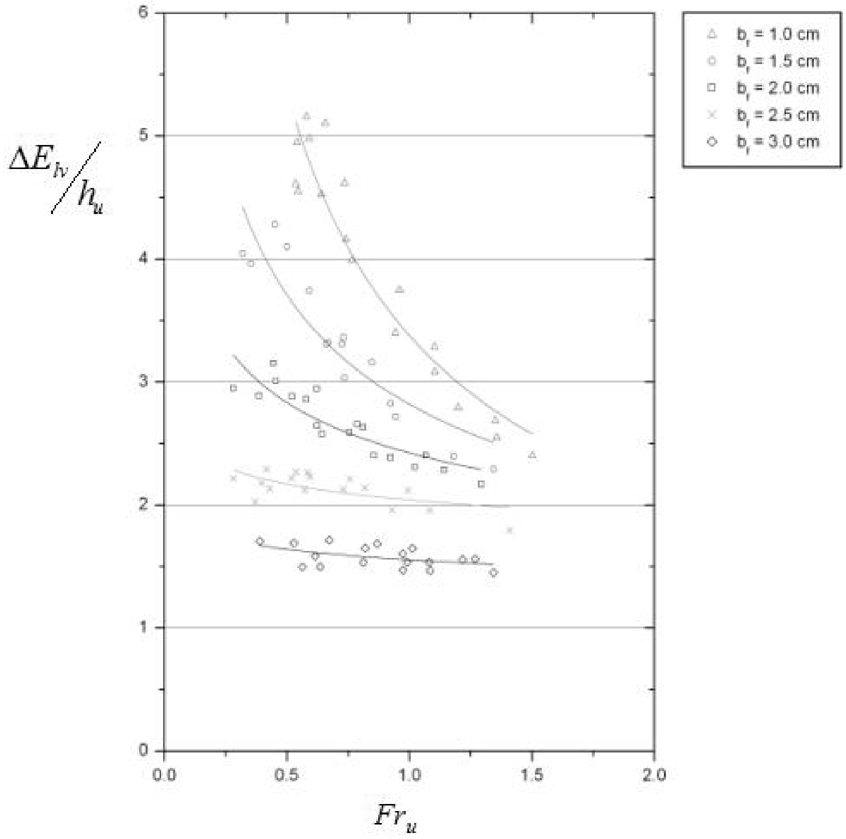

Fig. 9. Energy dissipation induced by lateral vortices.

This effect was experimentally observed to be inversely proportional to the Froude number and on the contraction ration R (Fig. 8 and Fig. 9).

From the experimental data the energy dissipation induced by lateral vortices (ΔE_{lv}) was computed with a best fitting technique, and the following relationship was found:

$$\frac{\Delta E_{lv}}{h_u} = \ln\left(\frac{R^{3/2}}{1.2}\right) Fr_u^{0.26-0.1R} \tag{9}$$

Combining the dissipative effect of the vertical vortex (eq. 9) and the one of the lateral recirculations represented by equation (8), it is possible to obtain the total energy dissipation determined by the presence of the check dam as a function of the Froude number (Fr_u) of the debris flow over the deposit and of the slit width, as represented in Fig. 9. The following analytical expression is obtained, that can be later substituted in equation (7).

$$\frac{\Delta E}{h_u} = \frac{Fr_u^2}{2}\left[1 - \frac{2}{3}(Fr_u R)^{-}2/3\right]^2 + \ln\left(\frac{R^{3/2}}{1.2}\right) Fr_u^{0.26-0.1R} \tag{10}$$

In this way an expression is obtained (eq. 11) that can be utilized for designing the width of the slit of slit check dams to be employed as a countermeasure against debris flow.

$$\frac{\Delta z_0}{h_u} = \frac{3}{2}\left(Fr_u R\right)^{2/3} - 1 - \frac{Fr_u^2}{2}\left\{1 - \left[1 - \frac{2}{3}(Fr_u R)^{-2/3}\right]^2\right\} + \ln\left(\frac{R^{3/2}}{1.2}\right) Fr_u^{0.26-0.1R} \tag{11}$$

Conclusions

The paper have presented a description of the functioning of slit check dams in controlling debris flows mudflows and both from the experimental and theoretical point of view. The authors have extended the existing scheme proposed by Armanini & Larcher (2001) for the case of ordinary sediment transport, presenting an approach to the problem of designing the opening of a slit check dam when the solid transport rate exceeds the typical limits of ordinary sediment transport, as it is often observed in steep torrents.

For the debris flow case, an expression that can be utilized for designing the width of the slit of slit check dams to be employed as a countermeasure against debris flow has been obtained. The important dissipative effects have been identified during the experimental activity, that are induced by the formation of two symmetric recirculation zones upstream the check dam and by the dissipative effect of the vertical vortex that forms between the downstream end of the deposition and the check dam. For the mudflow case, experiments shows that in particular conditions also mudflow can take place in equilibrium with a bed constituted of the same material, showing a typically two-phase behaviour, characteristic of granular materials. In this frame, the adoption a priori of one-phase models for the description of mudflow dynamics, very common in the scientific community, can be considered incorrect, because this approach prevents the possibility of analyzing separately the behaviour of the solid phase and the one of the interstitial fluid, and erosional and depositional mechanisms at the bed are neglected. Furthermore, the flow of mud through the slit check dam develops, contrarily to water a debris flow, constant rigid zones in the corners (called “dead zones”) in which the induced shear stress is smaller than yield stress and flow velocity is zero. The new shape of the narrowing affects the profile of the self-formed mobile bed near of the check dam.

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