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# A NEW SIMPLE METHOD FOR THE DETERMINATION OF THE TRIGGERING OF DEBRIS FLOWS.

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## ABSTRACT

We present the theory of a new method for assessing the debris flow occurrence in alpine basins. It is based on a simplified but consistent hydrological modeling. Our approach assumes that at the origin of the debris or soil mobilization there is surface runoff which affects points otherwise stable. The model complements the use of other simple models as SHALSTAB or SINMAP which investigate instead soil slips due to subsurface flows. As SHALSTAB and SINMAP, our model PEAKFLOW-SF uses the indefinite slope stability model, makes a wise use of digital elevation models, keep for granted that there is sediment enough to be transported and finally it has been implemented into a GIS system.

## 1. INTRODUCTION

In the recent years, a simple methods to determine the spatial patterns of shallow landslide potential (SHALSTAB) has been developed by Dietrich et al. (1992;1995), Montgomery and Dietrich (1994), Wu and Sidle (1995) and Pack and Tarboton (1997, SINMAP). With minor differences, these models combine a steady state shallow subsurface flow model with an infinite slope stability model (Skempton and DeLory , 1957) to map the distribution of relative slope instability. Instability is inferred to be greatest where the calculated ratio of effective precipitation to ground transmissivity is the smallest. This cohesionless model can effectively be treated as parameter free by using digital elevation model (DEM) data over wide areas to delineate regions where the majority of shallow landslides occur. Because of its simplicity, it has been adopted by private timber companies in the Pacific Northwest, is being used by state agencies and many professional. It has also been used in Canada (Pack and Tarboton, 1997) and Italy (Borga et al., 1998; Casadei and Farabegoli, 1998).
Dietrich et al.(1995) proposed also model for predicting the spatial distribution of soil depth, permitting the use of a slope stability model that explicitly accounts for both cohesion (due to soil properties and vegetation) and the vertical structure of the saturated conductivity.
For its theoretical foundation SHALSTAB is not able to predict soil failure derived from surface runoff. Hydraulic head in fact increases due to the presence of water on the terrain surface and the safety factor can decrease under the critical value of unity even in those locations which, according to SHALSTAB, are stable. Based on the GIUH theory and some recent results by (Rigon et al, 2003), we will show that it is actually possible to include in a sound theory of simplicity comparable to SHALSTAB these runoff effects. We warn the

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reader that in the following, we will skip all the complexities implied by the runoff generation processes on which we refer elsewhere (Rigon et al, 2003) and that no simple mathematical treatment of the flood process is possible without considering constant and uniform rainfalls (e.g. Rigon et al, 2003). However, these rainfalls can be obtained as function of the event duration and the frequency (return period). Let the depth be:

$$p(t, t_p \mid t_r) \equiv pH(t_p - t), \qquad (1)$$

where $H()$ is the step function (namely $H$=1 for $0 \le t \le t_p$ and $H$=0 otherwise), $p$ [mm] is the rain intensity relative to the duration $t_p$ [hours]. Then, we will also assume that the Intensity-duration-frequency (IDF) curves are in the form of a power law (e.g. Burlando and Rosso, 1996) as follows:

$$p = a(t_r) t_p^{-m} \qquad (2)$$

where $a$ is parameters function of the return period of the event and $0 \le m \le 1$ is a constant.

## 2. Simplified peak flow hydrology

### 2.1 The peak flow and the GIUH

It is known that the instantaneous discharge derived within the framework of the IUH is given by [e.g. Rinaldo and Rodriguez-Iturbe, 1996]:

$$Q(t) = A_T \int_0^t GIUH(t - \tau) p(\tau) d\tau \qquad (3)$$

where τ is the time passed from the precipitation beginning $p$ to the effective precipitation intensity τ and $A_T$ is the area of the basin contributing to the flood. The integral:

$$S(t) = \int_0^t GIUH(\tau) d\tau \qquad (4)$$

is known in literature as S-hydrograph and since for $t \to \infty$ $S(\infty)=1$, the product $A_T$ $S(t)$ is assumed to represent the portion of basin contributing to the flood at the time $t$. The GIUH is generally a parametric model whose parameters (usually omitted to simplify in the notation) have been estimated on the basis of the comparison with real events and we will assume it can be used even to evaluate the discharges of a single hillslope.
If the basin is assumed to be affected by a time-constant incident precipitation for the duration $t_p$ as assumed in (2) the discharge at any time is:

$$Q(t) = \left[ \begin{array}{ll} pS(t) & 0 \le t \le t_p \\ p(S(t) - S(t - t_p)) & t > t_p \end{array} \right. \qquad (5)$$

where $S(t)$ is the S-hydrograph [*Chow et al.*, 1988].

One of the main goal of hydrological analysis is the estimation of the maximum discharges, which can be obtained in a catchment after a precipitation of assigned mean intensity or of intensity derived from the local IDF curves depending on an assigned return period. The maximum discharge is obtained for the time, $t^*$, determined by solving the equation: $dQ/dt=0$ which reads [*Henderson*, 1963].

$$GIUH(t) = GIUH(t - t_p) \tag{6}$$

The meaning of (6) was detailed in *Rigon et al.*, 2003 and it is briefly described also in Figure (1). In particular, for $t > \tau_c$ ( where $\tau_c$ is the positive time after which $GIUH(t)=0$ and is called concentration time) $S(t)$ is a constant, while $S(t-t_p)$ is an increasing function of $t$; therefore, the solution of (6) will have to be found in the interval $t_p<t^*<\tau_c$.

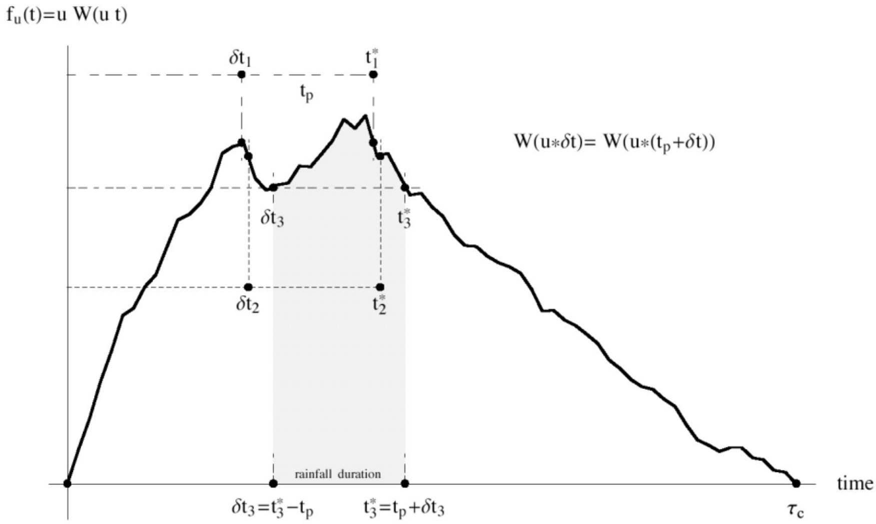


**Figure 1:** *After Rigon et al, 2003. The solutions of equation (6) are given by those pairs of points at a distance $t_p$ from one another, where the width function assumes the same value. In the case of the width function in this figure, there are three solutions and the third one corresponds to the maximum discharge. The gray area under the curve measures the basin area contributing to the peak flow and it is independent of the celerity value, u; in fact different values of u correspond to a change of coordinates that does not affect the integral. The base-time of the GIUH is the concentration time $\tau_c$, while $\delta t_3$ is the delay separating $t_p$ from $t^*$.*

The peak discharge generally happens with a delay ($\delta t=t^*-t_p$) with respect to the end of the effective precipitation and depends on the characteristics of the GIUH. Henderson's equation (6), in general, cannot be solved analytically. However, particular hydrograph can be solved. For instance, for the Nash hydrograph [Nash, 1957] it results:

$$t^* = \frac{t_p}{1-\left(\exp\left(-t_p/k\right)\right)^{1/(n-1)}} \tag{7}$$

where $n$ is the number of reservoirs and $k$ the expected residence time in a single reservoir. It is easy to note that $t^*$ is a function always greater than $t_p$ and increasing with $n$.

For $t_p=\tau_c$, the (6) has only one solution, while for $t_p \rightarrow 0$ the solutions tend to the number of maxima relative to the GIUH, since the hydrograph tends to the instantaneous hydrograph. For intermediate values of $t_p$, the number of solutions is monotonically decreasing with $t_p$ and, in general, only the number of maxima separated by less than $t_p$ in the GIUH produces a relative maximum in the overall hydrograph [*Rigon et al.*, 2003].

Then a simple ordering procedure lets us determine the time corresponding to the maximum discharge. The delay $\delta t$ of such peak over the duration $t_p$ is a decreasing function of $t_p$ which reduces to zero for $t_p=\tau_c$.

As a result, the peak discharge, $Q_p$, can be then estimated by using the equation (5) once $t^*$ is substituted to $t$.

## 2.2 Use of the intensity-duration-frequency curves to determine the maximum discharges with assigned return time

The peak discharge at the basin closure, $Q_p$ (5), is a function of the rainfall duration $t_p$ (through $p$). Since $p(t_p|t_r)$ is a decreasing function of $t_p$ and $S(t^*)=S(t_p+\delta t)$ is a monoton increasing function, there is a particular duration, $t^*_p$, which maximizes the peak discharge and which must be lesser than or equal to the concentration time, $\tau_c$. This duration, if the hydrograph is time-invariant, can be determined by solving the equation $dQ_p(t^*)/dt_p=dQ(t_p+\delta t,t_p)/dt_p=0$, where $\delta t=\delta t(t_p)$ is a function of $t_p$ only.

The result of the derivation is (Rigon et al, 2003):

$$\frac{p'\left(t_p \mid t_r\right)}{p\left(t_p \mid t_r\right)} = \frac{GIUH\left(t_p+\delta t\right)}{S\left(t_p+\delta t\right)-S(\delta t)} \tag{8}$$

where in the left member there is the ration of the derivative (with respect to $t_p$) of the IDF curve on the IDF curve and on the right side there is the ratio of the GIUH (evaluated at time $t^*$) on the basin area contributing to the peak. If (2) is used in (8) it becomes:

$$m = \frac{t_p IUH\left(t^*\right)}{S\left(t^*\right)-S(\delta t)} \tag{9}$$

which implies that the critical duration of precipitation is independent of the return time. the solution of (9) gives $t^*_p$, the critical duration for which (at fixed return period), the maximum allowable liquid discharge is obtained as:

$$Q_{MAX}(t_r) = p(t_p^* \mid t_r) C(t^*, t_p^*) A_T \tag{10}$$

where $C(t^*, t_p^*) \equiv S(t^*) - S(t^* - t_p^*)$. Equation (10) is similar to the well known equation of the rational method [e.g. *Chow*, 1988]; however, the runoff coefficient $C$ depends on the effective fraction of contributing area and it is not evaluated, in general, for the concentration time but for the time to peak $t^*$ and derived from geomorphological arguments and not by (direct) calibration.

## 3. Factors of debris flow initiation

Two factors mainly cause the debris-flow generation: one has a meteorologic nature, the other a morphologic nature. Indeed the analysis of events occurred in different conditions has showed that the debris flows occur after a rainfall of exceptional intensity following a rainfall of long duration [Honglian and Xiangxing Cai, 1968; Cannon and Ellen, 1988; Armanini and al., 2003]. Yet, besides the weather event, there must also be the presence of a deep enough debris accumulation. In fact, the frequency of debris-flow events does not coincide with the frequency of the intense rain events. Hampel [1968] distinguishes between watercourses on the rock and those flowing on the alluvial erodible deposits. The former require a gradual inter-event debris accumulation and are temporarily stable after the event, the latter can be destabilized by an event and this can cause a period of high activity, followed by the return to a state of inactivity since, after the flow has occurred, a long enough period is needed for the material to accumulate again.

A third type of mechanism is when the debris flow is originated by a landslide. At last, a possible source of debris flow is represented by the collapse of retention structures existing along the river mainstream. In the present paper we do not deal with the two last types of mechanisms.

From the analysis of the historical events on the Alps, but also in other similar places like Canada, China and Japan, we deduce that the debris flows concern basically the catchments of modest area: generally lesser than 5 $km^2$ an this makes the assumption of constant and uniform rainfall used in this analysis more acceptable.

In this paper we assume that the sediment is already available and we just focus on the characteristics of the floods which can destabilize it.

### 3.1 Stability of a granular cluster

The theory of the stability of dissolved-material clusters for the study of the debris-flow triggering has been introduced by Takahashi in 1978. This theory is based on the balance of the forces acting on a cluster for different wet conditions.

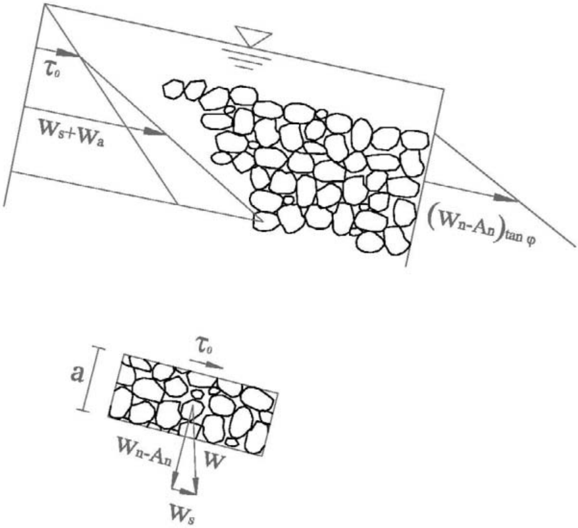


**Figure 2**: *Scheme of forces acting on a granular cluster submerged by water. $h_0$ is the surface runoff depth ,$\tau_0=h_0\gamma_w\sin\theta$ is the bottom shear stress, $W_s=C^* a\gamma_s\sin\theta$ is the tangential componeent of the weight, $W_a=(1\text{-}C^*)a\gamma_w\sin\theta$ the tangential component of the weight of the water,$W_n=C^* a\gamma_s\cos\theta$ is the normal component of the granular material weight, $A_n=\text{-}C^* a\gamma_w\cos\theta$ is the buoyancy.*

The cluster will remain in a condition of equilibrium as long as the ratio between the normal and the tangential component of the destabilizing forces is lesser than the tangent to the friction angle of the material Φ:

$$\frac{g\sin\alpha\left(h_0\rho + C^* a_L\rho_s + \left(1-C^*\right)\rho a_L\right)}{C^* a_L g\cos\alpha\left(\rho_s-\rho\right)} \leq \tan\phi \tag{11}$$

With reference to figure 2, let us consider a granular cluster composed of non-cohesive particles, with density $\rho_s$ and uniform granulometry; α is the cluster inclination angle and $C^*$ is the volume concentration of the particles; $p$=1-$C^*$ is then the cluster porosity, $g$ is the acceleration of gravity, $a_L$ the thickness of sediment involved, $h_0$ the depth of water on the surface. With the substitution of ($V_s/V$) for ($C^*$) (where $V=V_s+V_p$ is the total soil volume, $V$ the mineral volume and $V_p$ the volumes occupied by pores)equation (11) is exactly the same stability equation used in SHALSTAB. Let us assume that above the cluster there is water flowing with uniform motion and with flow depth $h_0$ and that the cluster itself is at rest. After the simplifications, this leads to the following inequality:

$$\frac{h_0 / a_L + C^* \Delta + 1}{C^* \Delta} \tan\alpha \le \tan\phi \tag{12}$$

where $\Delta=(\gamma_s-\gamma)/\gamma$ As we can infer from the figure, when the instability conditions begin, it is exactly the material surface layer which starts moving: in general, the thickness $a_L$ is proportional to the particle diameter: $a_L=n\ d$. With $n=1$, this condition was proven to be the same required by the Shield's theory [*Armanini and Gregoretti*, 2000].
In short, the cluster is stable as long as the inclination angle α is such that:

$$\tan\alpha \le \frac{C^* \Delta}{h_0 / (nd) + C^* \Delta + 1} \tag{13}$$

Characteristic of stony, non-cohesive material- for the above equation we could have the following parameters:

$$C^*=0.7 \qquad \Delta=1.65 \qquad \tan\Phi=0.8 \tag{14}$$

According to Takahashi [1978], the values of the ratio ($h_o/n\ d$) able to generate a real debris flow range from 0 to 1.33. For values lesser than 0 there is a partially dry cluster which gives rise to a landslide when, for high enough slopes, it becomes unstable. For values greater than 1.33 there is a movement more similar to the ordinary solid transport rather than to a debris flow. In short, the slopes for which there is debris-flow formation, according to Takahashi [1978], range between the two following extreme angles:

$$\tan\phi \frac{C^* \Delta}{C^* \Delta + 1} \ge \tan\alpha \ge \tan\phi \frac{C^* \Delta}{1 + C^* \Delta + 1.33} \tag{15}$$

and, after the use of the typical values of parameter in (14) we obtain:

$$23^o 51' \ge \alpha \ge 15^o 9' \tag{16}$$

For slopes lesser than the limit on the right of the relation (16), there is ordinary solid transport (hyperconcentrated); for slopes greater than the limit on the left of the relation (16) there is motion even in non-saturation conditions (and then with a relatively lesser presence of water), that is why the material hardly accumulates on the hillslopes with greater slopes. Once the movement is triggered, the flow can go on even with slopes lesser than the limit on the right of the relation (16).

## 4. Coupling hydrology and stability to determine the debris flow triggering

This stability analysis can be coupled with the peak flow analysis if a depth-discharge relation is assumed. If the geometry of runoff flow is known, this relation can be approximately obtained by the kinematic wave relationship [e.g. Singh and Prasana, 1999] or the usual Gaukler-Strikler (Manning) formula. In general, we can reasonably use a depth-discharge relation in form of a power law:

$$Q = q\ h_0^{s} \tag{17}$$

where $q$ is the average discharge for unit depth. Leopold and Maddock [1953] suggests that a reasonable value of $s$ is about 2.5. In this hypothesis, stability is granted when:

$$Q \le q\left(\left(\left(\frac{\tan\phi}{\tan\alpha}-1\right)C^{*}\Delta-1\right)^{s} a_L^{s}\right) \equiv R \tag{18}$$

$Q$ is known on the basis of the analysis of the previous sections, while $R$ is function of quantities either measured (from DEMs or surveyors) or assumed on the basis of literature.
Verified the instability condition for a certain point (assigned the return period of the rainfall and for the peak flow), the first approximation of the failure timing is given by solving:

$$\left(\left(\frac{\tan\phi}{\tan\alpha}-1\right)C^{*}\Delta a_L - a_L\right)^{s}\frac{q}{p} = \begin{cases} A_T S_T & t \le t_p \\ A_T\left[S(t)-S\left(t-t_p^{*}\right)\right] & t > t_p \end{cases} \tag{19}$$

where (4) is used and the critical rainfall duration, $t^{*}_{p}$ has been already determined. The solution of (19) gives the time of failure, $t_f$, and it is $t_p \le t_f \le t^{*}$. The accuracy of $t_f$ depends on the validity of the calibration of the parameters in the theory and in general needs to be accepted "cum grano salis" (after some wise reflection). Forthcoming studies will try to assess these uncertainties and include them into the theory. Much more acceptable is the estimation of the critical rainfall duration $t_p^{*}$ (and its associated rainfall intensity) which in many practical cases it is all what is needed.
For $h_0$=0 there is a completely saturate cluster with no water flowing on the surface. In this case the stability condition becomes:

$$\tan\alpha \le \tan\phi\frac{C^{*}\Delta}{1+C^{*}\Delta} \tag{20}$$

Condition with $h_0$<0 are already treated by SHALSTAB and are independent on surface flow.
Once the cluster starts moving, it transforms quickly into a debris flow. In this situation the dynamic friction is able to capture the layers of the material which lie below the initially unstable layer and the cluster can greatly grow.

## 5. Conclusion

We have developed a simple methods for treating debris flow triggered by surface runoff. This method is consistent in all of these parts and allows also for the determination of the initiation of ordinary sediment transport (according to Shield's theory). Many of the parameters used by the theory can be easily obtained from a DEMs and the model was actually implemented under the GRASS GIS by the authors. The simplicity of the method actually allowed to include in the PEAKFLOW-SF numerical model the estimation of (19) for all the channel links inside a basin, thus overcoming the usual limitation of rainfall-runoff methods which give the discharge just at the outlet of the basins. The method is however still missing field validation (both in terms of the hydrologic and geotechnic parameters) which are the goal of the ongoing research.


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