Internal Progressive Failure in Deep Seated Landslides

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Abstract

Except for simple sliding motions, the stability of a slope not only depends on the resistance of the basal failure surface. It is affected by the internal distortion of the moving mass, which plays an important role on the stability and post-failure behaviour of a landslide. The paper examines the stability conditions and the post-failure behaviour of a compound landslide whose geometry is inspired by one of the representative cross sections of Vajont landslide. The brittleness of the mobilised rock mass was described by a strain softening Mohr-Coulomb model, whose parameters were derived from previous contributions. The analysis was performed by means of a MPM computer code, which is capable of modelling the whole instability procedure in a unified calculation. The gravity action has been applied to initialise the stress state. This step mobilizes part of the strength along a shearing band located just above the kink of the basal surface, leading to the formation a kinematically admissible mechanism. The overall instability is triggered by an increase of water level. The increase of pore water pressures reduces the effective stresses within the slope and it leads to a progressive failure mechanism developing along an internal shearing band which controls the stability of the compound slope. The effect of the basal shearing resistance has been analysed during the post-failure stage. If no shearing strength is considered (as predicted by a thermal pressurization analysis) the model predicts a response similar to actual observations, namely a maximum sliding velocity of 25 m/s and a run-out close to 500m.

Keywords: landslide, progressive failure, brittleness, Vajont, run-out, sliding velocity, material point method, internally sheared compound slide

1. Introduction

The kinematics of a landslide motion is fundamental information to approach the mechanisms of deformation and their implication. The expected geometry of the sliding surface and the terrain topography provide useful initial information. However, the complexity of internal interactions within the rock mass, because of the imposed strain field during sliding, is better addressed through appropriate models. This paper focuses on internally sheared compound landslides (see the updated Varnes classification of landslides, Hungr et al., 2014). Different failure mechanisms (Glastonbury and Fell, 2008a) may be identified in compound slides (Figure 1). In these cases the slope response is determined by the generation of internal shears that

make it possible to convert the landslide into a kinematically admissible. In an internally sheared compound slide, the pre failure stage, defined as the initial motion leading to the development of a continuous rupture surface, requires the continuous shearing not only along the basal failure surface but also along internal planes.

Figure 1. Schematic illustrations of the mechanisms of internally sheared compound slides. (Glastonbury and Fell, 2008).

The onset of failure will be determined by the available strength along the basal and internal shearing surfaces. It is common to find in nature landslides whose basal rupture surface is defined by bedding planes. Others follow low strength strata that have been pre-sheared under tectonic deformation events or previous sliding (some examples are collected and discussed in Glastonbury and Fell, 2008b). This is the case, for instance, of slides associated with synclines or complex rupture surfaces with undulations. These slopes can remain stable even if the safety factor with respect to the basal slip surface is below unity. This fact can be explained because internal shearing planes require the fracture of intact rock in orientations normal to the major direction of anisotropy (see Figure 1). This mechanism implies that the available strength associated with internal shearing is relatively high compared with the resistance offered by basal surfaces. Hence the strength along both the basal surface and the internal shearing bands determine the stability of the slope.

A strain weakening behaviour is expected when shearing rock masses. A progressive failure mechanism is then expected. It will explain the acceleration of the landslide once the

kinematically admissible mechanism is generated and a set of failure surfaces are completely formed. This aspect was also analyzed, in a simple manner, by Alonso et al. (2010) (Chapter 2).

The relevant effect of internal shearing and internal brittleness was invoked by Hutchinson (1987) to interpret the response of Vajont slide (Hendron and Patton, 1985). Mencl (1966) also analysed the kinematics of this landslide and proposed different failure mechanisms, in particular the development of a Prandtl's wedge within the mass. Vajont slide is described as an ancient landslide reactivated due to the combined effect of a reservoir impoundment which submerged part of the slope toe and rainfall infiltration. Two representative cross-sections of Vajont slope are plotted in Figure 2.

Figure 2. Two representative cross-sections of Vajont landslide: (a) Section 2; (b) Section 5. (After Hendron and Patton, 1985). P1 and P2 indicates the position of and length of piezometers. (Horizontal scale = Vertical scale).

The valley slope follows the shape of a syncline structure which folded Jurassic and Cretaceous strata. The basal failure surface was located in clayey layers subjected to intense shearing in past geological times because of previous instability. Above the sliding surface, finely stratified layers of marl and limestone from the Malm period were identified. Given the shape of the basal failure surface the slide movement requires the shearing along internal planes crossing the marl and limestone strata as a result of the sharp transition between the upper and steeper failure surface and the lower and practically horizontal part.

Vajont slide movements were correlated with the impounding of the reservoir. During three years, from 1960 to 1963, the reservoir level rose from 580 m a.s.l to 700 m a.s.l. During this period of time about 4 m of surface displacement were accumulated (Nonveiller, 1987). The increase of the reservoir level up to 700 m led to a violent and accelerated failure. The Vajont slide behaviour was interpreted by Hutchinson (1987, 1994) who included the effect of internal shearing planes. Consider in Figure 3, the critical stability condition of Vajont slope when the reservoir water level rose to an elevation of about 600 m a.s.l. Under this condition a safety factor equal to 1 can be formally assigned within the context of overall limit equilibrium.

The increase of the reservoir level (from 600 m a.s.l. to 700 m a.s.l.) resulted in a pore pressure build-up and a reduction of the effective shearing strength along the basal plane surface. As a result a progressive increment of the mobilized shearing strength along the internal shearing planes crossing the rock mass is expected in order to maintain equilibrium. Taking into account the brittleness of the Cretaceous limestones and marly limestones, the internal shearing strength will be progressively mobilized. But this process has a limit. Continuous internal shearing planes will eventually develop and a kinematically admissible mechanism will be formed. According to Figure 3, at this moment the safety factor drops suddenly. This time instant marks the sudden acceleration of the sliding mass.

Figure 3. Approximate variation of the safety factor with rising reservoir level showing the internal breaking effect and the sudden acceleration resulting from brittle failure on internal shears (from Hutchinson, 1987).

The post failure behavior in the case of internally sheared compound landslides, in general, and in the case of Vajont, in particular, should be analyzed as a new stage of dynamic deformation. Once the residual strength is reached along the most stressed internal shearing planes and the slide accelerates and moves forward, "new" material from the upper wedge having an available strength higher than the residual one, because it has not been mobilized yet, should be sheared in order to fulfill the kinematic conditions of the motion. This stage will be included in the analysis, as well as the new sliding geometry which is continuously evolving with the motion.

The objective of this paper is to analyze the relevance of internal kinematics and the internal degradation of the sliding mass through an appropriate computational tool capable of describing the landslide motion including the onset of the failure and the runout in the case of a brittle (strain-softening) rock. The Material Point Method (MPM) (Sulsky et al., 1994) has been selected as an appropriate computational technique because it is able to simulate large displacements in a dynamic context. The continuum is discretized by means of a set of lagrangian material points. MPM permits combining geological, geometrical, hydraulic, and geotechnical features in order to understand the stability of the slide. Moreover, the dynamic formulation allows the analysis of the kinematics of the motion and modeling, in a unified calculation, the failure and the post-failure behaviour. Similarly to the work presented by Zabala & Alonso (2011), the brittleness of the material will be simulated by means of an elastoplastic constitutive model which includes strain softening.

The analysis presented in this paper refers to a particular case which is inspired in Vajont landslide. The landslide is triggered by increasing the pore water pressure inside the slope. Before discussing the results obtained, the computational model, details of the analysis and limitations are described. The evolution of the slope deformation during the pre-failure stage and the post-failure response up to the stabilization of the landslide is described.

Vajont landslide was characterized by the high velocity reached, which was estimated to be about 20-30 m/s after 400 m of displacement approximately. These values were estimated according to the height of the generated wave, which reached 235 m above the reservoir level (Hendron & Patton, 1985). Such acceleration can be explained assuming a drop of the basal shear strength to values near zero. The favorite explanation in a number of published contributions on the subject is associated with the development of frictional heat at the sliding surface which induces the increase in water pressure due to the dilation of pore water as temperature increases (Faust, 1982; Hendron and Patton; 1985; Vardoulakis, 2002; Rice, 2006; Veveakis et al., 2007; Goren and Aharonov, 2009; Pinyol and Alonso, 2010a, 2010b; Cecinato et al., 2011; Cecinato and Zervos, 2012). In some contributions heat induced soil plastic collapse of the shearing band is also included in the formulation.

The loss of strength available on the basal failure surface will also be introduced in the analysis presented here. The objective was to calculate the slide run-out and maximum velocity and to compare it with field behaviour. Unlike previous contributions the analysis performed include a massive internal shearing in a brittle rock during the entire motion of the slide, which implies major changes in geometry. The initial and final geometry of Vajont landslide is plotted in

Figure 4. The mass crossed the valley and climbed up the opposite slope. This stage of the slide resulted also in internal rock shearing which was incorporated into the analysis.

Analyses of Vajont were recently reported by Crosta et al. (2015) and Zhao et al. (2015). The first paper presents a FE modelling of the landslide motion (2D and 3D) adopting an arbitrary Eulerian-Lagrangian method. The second paper represents the sliding mass by a Distinct Element model. They pay special attention to the generation of impulse waves in the reservoir.

Figure 4. Cross sections (a) before and (b) after the landslide (from Del Ventisette, 2015; modified after Rossi and Semenza, 1965). (Horizontal scale = Vertical scale).

2. Outline of the MPM method

The Material Point Method (MPM) was originally developed by Sulsky et al. (1994). It is typically classified as a method in between the mesh-free methods and the classical Finite Element Method (FEM). MPM discretises the continuum by means of a set of lagrangian points, so-called material points. Each point represents a subdomain of the media and carries all the information (e.g. mass, volume, position, velocity, strain, stress). Besides, governing motion equations are solved incrementally at the nodes of a background computational mesh which remains fixed throughout the calculation and covers the full domain of the problem. The standard nodal interpolation functions provide the relationship between material points and the grid at every time step. This dual description of the domain avoids mesh tangling and allows the simulation of large displacements without re-meshing. Moreover, the detection of contact between different bodies is automatic and the implementation of specific contact elements is not required. A scheme of the spatial discretization used in MPM is illustrated in Figure 5.

Figure 5. Scheme of the spatial discretization used in MPM formulation.

The MPM formulation for a mechanical problem was presented by Sulsky et al. (1995) in which the dynamic momentum balance equation is discretised. In the geotechnical field, other authors extended this formulation to solve coupled hydro-mechanical problems in saturated porous media (Zhang et al., 2009; Mackenzie-Helnwein et al., 2010; Zabala and Alonso, 2011; Jassim et al., 2013; Abe et al., 2013; Bandara and Soga, 2015). More recently, Yerro et al. (2015) proposed a coupled MPM approach to model unsaturated soils which are three phase (solid, liquid and air) materials.

Although the standard MPM is capable of modelling contact between different entities, it does not allow simulating discontinuous velocity fields. Therefore, when relative movement between different bodies is expected, an additional algorithm should be implemented. The most common contact algorithm was presented by Bardenhagen et al. (2001). It was originally developed to simulate interaction between grains in granular materials and it allows sliding and rolling with friction as well as separation between solid bodies.

An artificial damping force has been considered in this MPM formulation in order to reduce numerical instabilities and the effect of boundaries. It is proportional to the corresponding unbalanced force (proportional factor α) and it is opposite to the material velocity. It can be understood as a frictional force. High values of the proportional factor can be used in quasi-static problems to get the static solution in a fast way. In dynamic problems, where accelerations have an important role, this factor should be very small (0-5%) in order to approximate the correct solution. A value of $\alpha = 0.05$ is adopted in all calculations presented below.

3. Brittle Mohr-Coulomb model

Because MPM is based on continuum mechanics, the constitutive behaviour of the material can be formulated within the theory of elasto-plasticity. In this work, an elastoplastic Mohr-Coulomb constitutive model with strain softening has been implemented to simulate the brittleness of rock masses (Yerro et al., 2014). In order to reduce the singularities of the Mohr-Coulomb yield surface, the modifications proposed by Abbo and Sloan (1995) have been introduced. An explicit sub-stepping algorithm with error control and a correction for the yield surface drift have been applied (Potts and Gens, 1985).

Softening rules describe how the strength parameters vary with plastic straining. In this case, the state parameters are cohesion c' and friction angle φ' which decrease exponentially with the accumulated deviatoric plastic strain invariant ε_d^p as:

$$c' = c'_r + (c'_p - c'_r) e^{-\eta \varepsilon_d^p} \quad (1)$$

$$\varphi' = \varphi'_r + (\varphi'_p - \varphi'_r) e^{-\eta \varepsilon_d^p} \quad (2)$$

The deviatoric plastic strain invariant ε_d^p is defined as follows

$$\varepsilon_d^p = \sqrt{\frac{2}{3} \mathbf{e}_{ij}^p \mathbf{e}_{ij}^p} \quad (3)$$

where $\mathbf{e}_{ij}^p$ is the deviatoric part of the plastic strain tensor.

The model requires the specification of peak (c'_p, φ'_p) and residual (c'_r, φ'_r) strength parameters. The rate of strength decrease is essentially controlled by the plastic shear strain but an additional “shape factor” parameter η is also included in equations (1) and (2). The higher the shape factor η , the smaller the loss in strength.

The inclusion of strain-softening features in continuum numerical methods leads to strain-localization problems. In order to reduce the mesh dependence associated with this phenomenon, Rots et al (1985) describe a smeared crack approach as a suitable regularization technique. It postulates that the total work dissipated by a shear band is equivalent to the fracture energy dissipated in a theoretical discrete crack. Then, assuming that the thickness of a shear band is approximately the size of mesh elements, the parameter η can be calibrated by performing a set of numerical simple shear tests. An acceptable relationship of shear stress-

displacement provides a value of η for a certain element size. This process has also been recently described in Soga et al. (2015).

4. Description of the MPM model

Geometry and numerical parameters

The model presented in this work (Figure 6) is inspired in a simplified geometry of the Vajont slope but close to the real one (section 5 in Figure 2b). It is based on a representative cross-section of the valley located 600 m upstream of the dam position (Hendron and Patton, 1987).

The geometry of the problem consists of a rock mass volume lying above a pre-existing basal sliding surface (Figure 6). Below the sliding surface the material remained unaffected hence it is not included in the model.

In 1960, after the dam was built and the reservoir partially impounded, some field observations revealed the existence of a peripheral crack (Belloni and Stefani, 1987). This continuous crack was an indication that a huge rock mass was partially detached from the rest of the slope.

The model presented in this work is a thin slice, 20 m wide, made of 4609 tetrahedral elements. All nodes of the computational mesh are contained within the two slice faces. In this way, a plane strain modelling is carried out restraining to zero the perpendicular movement to the slice faces.

Figure 7 illustrates the initial distribution of the material points representing the rock mass above the sliding surface. The computational mesh defines the whole domain of the problem. Four material points are initially distributed within the fully filled elements. These are located at the corresponding integration points of a 4-point Gaussian quadrature. The mean edge element size where the failure is expected is 20 m.

Although the relationship between reservoir level and slide velocity was not entirely clear, it seemed evident that the reservoir filling was the determining triggering mechanism of the Vajont landslide. In fact, the groundwater conditions are poorly known. Only the measurements of a few piezometers distributed along the slide (Figure 2) were available (Hendron and Patton, 1987, based on data from Müller, 1964). Probably, the water pressures that really controlled the stability of the slope were those prevailing at the sliding surface, but none of the piezometers was deep enough to reach it.

In the MPM model presented here, the water level is maintained horizontal throughout the calculation. This hypothesis is, to some extent, supported by the piezometric measurements, which recorded levels very similar to the water table of the reservoir. This fact indicates that the toe of the sliding mass was permeable, which suggests that this area was intensely fractured.

A drained analysis has been carried out, by means of the MPM mechanical formulation combined with an effective stress analysis. At each time step, the water pressure is imposed directly on the material points. For a certain level of the reservoir h_w , points located above this level remain with no pressure; meanwhile the water pressure in points located below the water table (including those along the contact surface) corresponds to the hydrostatic profile for the given level at a corresponding time. In addition, free reservoir water is not modelled in this analysis; hence the hydrostatic pressure against the slope is not taken into account. This assumption implies that the model is closer to a limiting state than the actual field conditions. However, this effect is small in this case.

Figure 6. Cross-section of the MPM model. Initial geometry. Length in meters.

Figure 7. Computational mesh and initial distribution of the material points.

Kinematic constraints of the model

Alonso et al. (2010) analysed the kinematics of Vajont slide. They assumed that all points slid parallel to the basal failure surface with the same absolute velocity. In advanced stages of the post-failure, the motion of the slide implies that mass from the upper wedge becomes part of the lower one (Figure 8). During this process, the lower sliding surface accumulates shearing strains. On the other hand, the rock mass arriving to the position of the internal shearing band becomes sheared to a certain extent and moves downhill leaving intact the rock upslope (as indicated in Figure 8). According to this scenario, the internal shearing band receives continuously new undisturbed rock as the slide moves downhill. Shearing along the internal shearing band will result in a process of progressive failure which, in an extreme case, will take the rock to residual conditions.

The model presented below will clarify if these assumptions are consistent with the brittle constitutive model of the rock material and the overall geometrical dynamic evolution of the motion.

Figure 8. Kinematics of sliding. Two-block mechanism and internal rock degradation (a case of compound landslide in a brittle material).

Materials and initial state

Vajont rockslide was, in fact, the reactivation of a paleolandslide. Semenza (2001) attempted to reconstruct the past history of the slide and he came up with a set of cross-sections illustrating a possible sequence of events. He interpreted that erosion processes at the toe of the slope caused by the Vajont river could explain the initiation of the failure surface and the past motion of the slope. Figure 9 illustrates a simplified reconstruction of the paleolandslide. During the deformation process the slope was subjected to cumulative relative displacements and intense fracturing, because of the kinematic constraints imposed by the kink of the sliding surface. Because the rock strength depends on the deformation history, it could be expected that, at the time of the dam construction, some areas could already be damaged.

Figure 9. Simplified reconstruction of the paleolandslide: (a) original profile; (b) profile after paleolandslide; (c) initial profile for the present analysis after erosion of the toe.

The nature of the Vajont sliding surface was discussed by Hendron and Patton (1985). They concluded that it was a layer of a few centimetres thick of high plasticity clay. Taking into account the scale of the whole problem (the final displacement of the slope was several hundred meters) the thickness of the shear band is neglected in this work. A contact algorithm proposed by Bardenhagen et al. (2001) and implemented by Al-Kafaji (2013) is used to model the basal sliding surface. This is a frictional contact in which the slip occurs when the tangential force exceeds the maximum allowable threshold determined by Coulomb friction.

Considering the past history of the landslide it is clear that the residual friction angle of the clay layers was the most relevant parameter controlling the strength of the basal sliding surface. Most of the early Vajont stability analyses concentrated on determining the basal sliding strength necessary to maintain the equilibrium of the slope (Menzel, 1966; Skempton, 1966; Kenney, 1967). Some of them are based on classic limit equilibrium methods (LEM) in

which the basal stability angle (ϕ'_b) was estimated in the range of 18-28°. These values are not consistent with laboratory tests. Some authors (Hendron and Patton, 1985; Tika and Hutchinson, 1999) examined the shearing strength by direct and ring shear tests on clay samples from the basal sliding surface and estimated that the static residual friction angle was around $\phi'_b = 10^\circ$. Hendron and Patton (1985) suggested that the roughness of the failure surface could amount to two additional degrees and proposed $\phi'_b = 12^\circ$ as a suitable choice. In this analysis $\phi'_b = 12^\circ$ has been used in the contact algorithm.

Above the basal sliding surface, marls layers were identified. Lying above marls, there were limestone strata (Semenza, 2001). In this model, a unique rock material has been used to simulate the whole rock volume. Following Hoek (2007), Alonso et al. (2010) approximated the strength envelope of the material above the sliding surface. The effective Mohr-Coulomb strength parameters were determined for a range of normal stresses of 2 MPa ($c' = 787$ kPa and $\phi' = 38.5^\circ$). They also analysed the static equilibrium of the slide by using a simple two-block model and suggested a range of effective strength values. These strength parameters are rough average approximations.

Rock material is typically characterised by brittle behaviour. In this work the Mohr-Coulomb peak envelope is defined by the strength parameters $c'_p = 1900$ kPa and $\phi'_p = 42^\circ$; the residual strength of the rock is defined by $c'_r = 300$ kPa and $\phi'_r = 36^\circ$. The shape factor adopted to control the rate of strength decrease is $\eta = 150$. Other material parameters are summarized in Table 1. Even if there are uncertainties on an appropriate set of average “in situ” parameters (Superchi, 2012), the selected set offers a good approximation to discuss the effect of internal degradation of the material during the landslide motion.

Table 1. Material parameters of rock mass.

Material parameter	Value
Porosity, n	0.2
Young Modulus, E [GPa]	5
Poisson ratio, ν	0.33
Solid density, ρ [kg/m ³]	2700
Peak effective cohesion, c'_p [kPa]	1900
Residual effective cohesion, c'_r [kPa]	300

Peak effective friction angle, ϕ'_p [°]	42
Residual effective friction angle, ϕ'_r [°]	36
Shape factor, η	150

357

358 Reproducing the initial stress state in the field is a difficult task in the absence of data.
359 However, trying to exactly reproduce the processes that have affected the rock mass during its
360 past geological history is out of the scope of this work. In this analysis, a gravity loading has
361 been applied to the intact material in order to calculate an initial stress state. It seemed
362 reasonable to consider the rock weight as the most relevant factor to determine the initial
363 stress state in the slope.

364 In the model presented below, a gradual gravity loading is applied to the intact material. As a
365 result, some points above the kink are sheared, accumulating plastic **strain** and a two-block
366 kinematically admissible mechanism is initiated. The strength is reduced locally along this
367 “initial” internal shear band and its mean value becomes intermediate between peak and
368 residual states (Figure 10). Finally, a stable geometry is obtained. During this process, the level
369 of the reservoir is maintained at the position of the horizontal lower basal sliding surface ($h_w =$
370 0 m). As a consequence, the rock located within the current internal shear band damages
371 according to the brittleness of the material.

372 Because the paleolandslide was not simulated, the toe of the slope remains under peak
373 conditions. The stability of a compound slide is essentially controlled by the strength of both
374 the basal surface and the internal shearing band. Therefore, the strength of the rock mass that
375 has already been sheared at the kink does not play a significant role in the stability of the
376 slope.

377 Figure 10 shows the accumulated deviatoric plastic strain after gravity loading. It is localised
378 along an almost vertical band that initiates at the existing kink of the basal surface and
379 progresses a significant distance upwards.

Figure 10. Scheme of deviatoric plastic strain field under gravity loading (initial state).

5. Numerical results

Dynamic behaviour and internal degradation of rock mass

According to Hendron and Patton (1985) and Hutchinson (1987) (Figure 3), Vajont slide reached critical stability conditions when the reservoir water level was about 20 m above the lowest level of the slide toe. However, surface measurements indicated an accelerated movement, for the first time, when the water level was around 60 m. Immediately afterwards, the reservoir was partially emptied in order to stabilise the slope. Later on, the reservoir level was increased and decreased again. Finally, the fast landslide occurred when the reservoir elevation was about 120 m.

In this work, only an initial increase of the water level (60 m) has been simulated and no further decrease of reservoir elevation has been considered. For this reason, the reservoir level was maintained at 60 m. In the model presented here, the water level is increased in intervals of 10 m up to failure (Figure 11). Afterwards, it is maintained constant thereafter until stabilisation of the motion.

The movement of point P0 (Figure 6) is illustrated in Figure 12. Note that the displacement is in meters and the scale is logarithmic. As the water pressures increase within the slope, P0 is stable. When the water level reaches a value of 60 m, the displacement of P0 increases rapidly leading to a final movement of 8.5 m. In the same figure, the movement of point P1, located at the upper sliding wedge, is also represented. It is clear that both points (P0 and P1) displace the same amount, which supports the kinematic hypothesis suggested by Alonso et al. (2010).

Water level rise causes a reduction of the mean effective stresses within the slope, leading more points to reach the yield function envelope. The strain softening behaviour reduces the available strength and the progressive internal degradation of the rock mass continues. This mechanism was identified as a progressive failure phenomenon by early contributions

(Skempton, 1964; Bishop, 1971; Cooper, 1996). The displacement of the slope increases slowly during this process whereas a shearing plane, dividing the sliding rock mass in two well defined wedges, is being formed. When $h_w = 60$ m, the admissible kinematic mechanism is completely developed and the slope accelerates immediately afterwards.

In Figure 13 the evolution of plastic strain is illustrated at five different times. As water pressure increases, the initial internal shear band is formed (Figures 13a, 13b). It can be distinguished in Figure 13c. In Figure 13d the reservoir level reaches 60 m and just afterwards the instability occurs. Figure 13e corresponds to the final stabilised geometry of the slope, in which a noticeably wider shear band can be observed. A moderate change in the direction of the shearing band is also noticed. The initial shearing band (Figure 10 and 13a) is subvertical. The shear band at the end of the progressive failure (Figure 13e) follows closely the bisector plane of the two basal sliding planes.

In order to analyse the progressive failure mechanism developed within the internal shear band just before the instability, the mobilised friction angle $\hat{\phi}'$, defined by Skempton (1964) is used to determine the degradation of a material point. It is defined as follows:

$$\sin \hat{\phi}' = \frac{q}{p' + c' \cot \phi'} \quad (4)$$

where p' and q are the deviatoric and the effective volumetric stress components.

Figure 14 shows the mobilised friction angle at eight material points distributed along the internal failure surface. The curves indicate the evolution at different times of the calculation. It is important to highlight that the available strength at the initial state (t_0) depends on the material point because some points have previously plastified during the gravity loading. As h_w increases, the material degrades progressively upwards. Point 8 is the last one to reach the peak strength envelope just when $h_w = 60$ m (t_4). Finally, when the motion begins the strength of the whole surface is reduced to residual conditions (t_5). Note that the mean available strength at the beginning of this process is smaller than the peak value due to previous accumulated shearing.

The relationship between run-out and velocity of the moving mass is presented in Figure 15. The slide reaches a maximum velocity of 0.5 m/s and stops after a displacement of 8.9 m. Following the classification system proposed by the International Union of Geological Sciences Working Group on Landslides (1995), this can be considered as “very rapid” landslide (velocity

limits: 0.05-5 m/s). The calculated run-out-velocity relationship is very similar to the relationship calculated by Alonso et al. (2010) in which the dynamic motion equation for two interacting wedges was solved.

Figure 11. Water table evolution.

Figure 12. Run out of the slide. Variation of absolute displacements of points P0 and P1, in terms of reservoir water level.

(b)

(c)

(d)

(e)

Figure 13. Contours of deviatoric plastic strain at different positions of the water level. (a) $h_w = 50$ m; (b) $h_w = 54$ m; (c) $h_w = 57$ m; (d) $h_w = 60$ m; (e) final stable geometry.

Figure 14. Progressive failure. Evolution of mobilised friction angle along the internal shear band at different times (in terms of water level): (t_0) initial state; (t_1) $h_w = 20$ m; (t_2) $h_w = 40$ m; (t_3) $h_w = 50$ m; (t_4) $h_w = 57$ m; (t_5) $h_w = 60$ m.

Figure 15. Calculated run-out and slide velocity of point P0.

Reduction of basal surface friction angle

Vajont slide moved forward approximately 450 m in a few seconds, reaching an estimated velocity of 20-30 m/s (Hendron and Patton, 1985), which is an “extremely rapid” motion. A

relevant question in Vajont landslide is why the sliding mass accelerated so much. The maximum speed in the model discussed in the previous section is about two orders of magnitude lower, despite the effort to reproduce the real case.

Authors tried to explain the increase in speed in different ways. One of them is the effect of strain rate on clay friction. Tika and Hutchinson (1999) tested at different strain rates remoulded specimens from clay layers belonging to the sliding surface. They found that friction angle decreases with strain rate and they reported minimum residual values of $\phi'_b = 5^\circ$. Many authors have reported in recent years the results of high velocity shearing (limited to around 1.3 m/s in most cases) in a variety of soil types (Di Toro et al, 2006; Mizoguchi et al, 2007; Liao et al 2011; Han and Hirose, 2012; Yang et al, 2014) tested both saturated and unsaturated as well as in samples from Vajont weak layers (Ferri et al, 2010). The change in friction angle with shearing rates is however a controversial issue discussed in some detail in Alonso et al. (2015).

The most accepted explanation for the extremely high sliding velocity is associated with the development of frictional heat at the basal sliding surface (Uriel Romero & Molina, 1977; Voigt and Faust, 1982; Nonveiller, 1987; Vardoulakis, 2002; Pinyol & Alonso, 2010). The initiation of the movement and the frictional work input on the sliding shearing band results in an increase in temperature. Then, water pressure within the clayey band increases and the available effective stress along the basal sliding surface is reduced to near-zero values and shearing strength vanishes (which is equivalent to the condition $\phi'_b \approx 0^\circ$).

In order to analyze the effect of the basal strength on the sliding speed two additional calculations have been carried out. Starting at the stabilized geometry (described in the previous section), and maintaining the water level at $h_w = 60$ m, the effective friction angle imposed along the basal contact surface was reduced from $\phi'_b = 12^\circ$ to $\phi'_b = 5^\circ$ in the first calculation, and to $\phi'_b = 0^\circ$ in the second one.

Figure 16 shows the results for the two analyses. The evolution of the accumulated deviatoric plastic strain is presented. The two slides accelerate immediately after the reduction of the basal contact strength. Mass from the active wedge enters into the internal shearing zone. Strength in the band reduces due to the brittle behaviour of the rock and, this degraded mass becomes part of the passive wedge as the motion proceeds. When the mobilised mass reaches the opposite side of the valley another shearing zone is developed at the position of the kink of

the basal surface. Now, the toe of the slope, which until now has remained at peak strength, is sheared as it climbs the slope. The smaller the basal strength, the longer the upward displacement of the rock mass before getting a new stable geometry.

In order to illustrate the degradation of the rock material during the post-failure stage, Figure 18 shows the mobilised friction angle in thirteen material points when $\varphi'_b = 0^\circ$. These points are located 130 meters above the basal surface and they are distributed along the landslide as shown in Figure 17. Note that points 5 and 6 are initially plastified because they are located within the shear band developed during the increase of the water level in the reservoir. The upper wedge is sheared as it approaches the position of the kink (Figure 18a). When the slope reaches the opposite side, the material points from the lower wedge plastify as well (Figure 18b). Finally, points 7 and 8, located in the central zone, remain in elastic conditions.

Figure 19 shows the relationship between run-out and the velocity of the moving mass in the two calculations. For $\varphi'_b = 5^\circ$, the slide reaches a maximum velocity of 14 m/s and it moves forward 344 m. If the basal friction vanishes ($\varphi'_b = 0^\circ$), the mobilised mass moves faster, reaches a maximum velocity of 25 m/s and the calculated final run-out is 491 m. This data is consistent with the accepted behaviour of Vajont landslide.

Figure 16. Accumulated deviatoric plastic strain at different times. Effect of effective friction angle at the basal sliding surface.

Figure 17. Location of thirteen material points (a) just before the vanishing ($\phi'_b = 0^\circ$) of the basal friction angle and (b) at the final stable geometry.

(a)

(b)

Figure 18. Rock degradation for $\phi'_b = 0^\circ$. Evolution of mobilised friction angle at thirteen material points distributed along the slope, parallel to the basal failure surface (indicated in Figure 17). (a) Initial degradation of the upper wedge; (b) toe degradation.

Figure 19. Calculated run-outs and sliding velocity. Effect of friction angle at the basal sliding surface.

6. Discussion

The model presented above is a simplification of the Vajont compound slide in which a sharp transition between the inclined upper planar sliding surface and the horizontal lower one has been considered. According to Semenza (2001), the actual transition between the two fairly planar surfaces was probably more rounded (see Figure 4a). This modified geometry could, “a priori” have a significant impact on the results, especially as far as the failure mechanism is

concerned. In order to study this effect an additional calculation incorporating a more realistic geometry has been carried out. A circular arc having a 200 m radius ($R=200\text{m}$) substitutes the previous sharp kink and characterises now the fold of the new sliding surface. It reproduces in a satisfactory manner Figure 2b. Rock properties, gravity loading, and the trigger of instability did not change with respect to the previous model.

The new basal failure surface is shown in Figure 20. Also shown in the figure are the distribution of plastic shearing strains when the slope was made unstable by reducing the basal friction angle to $\phi'_b = 0^\circ$. The plot corresponds to a time $t = \dots\text{s}$ after the initiation of the motion. Unlike the previous single internal shearing band starting at the kink (Figure 13), two shearing planes progress from the fold towards the surface of the slope defining a sector-shaped area (Figure 20), leading to a wider and irregular shearing zone.

The relationship between run-out and velocity for the new analysis is given in Figure 21. The maximum velocity and run-out obtained are slightly higher than the results obtained previously with the simplified sharp geometry (Figure 19). For $\phi'_b = 0^\circ$ the calculated maximum velocity is now close 27 m/s and the maximum run-out is 503 m. These figures could be compared with the equivalent values for the sharp geometry (25 m/s and 491 m respectively). If the basal friction is reduced to $\phi'_b = 5^\circ$ in the rounded case these values are 17 m/s and 384 m, against 14 m/s and 344 m for the sharp geometry. Results are similar. Even if a rounded more realistic basal surface is introduced, the estimated velocity and run-out of Vajont landslide requires the cancellation of the basal friction.

It is concluded that the rounded geometry introduces a change on the internal failure mode which is probably a more realistic result. However, the essential kinematics of the motion are very similar to the calculated single internal shearing plane developing for the simplified geometry (Figures 7 and 8). Differences between maximum velocity and run-out are considered minor.

Figure 20. Failure mechanism for a rounded basal sliding surface.

Figure 21. Calculated run-outs and sliding velocity considering a rounded basal sliding surface. Effect of friction angle at the basal sliding surface.

7. Conclusions

The paper provides some insight into the role of internal shearing to explain the motion of compound landslides. The analysed example, directly based on Vajont geometry, is typical of an initial syncline folding of strata followed by river erosion. In cases of reactivation of previous instability, internal shearing plays a key role to stabilize the impending motion. The limit is the exhaustion of the strength in a shearing area. If the material involved is brittle, the internal failure results in a progressive failure mechanism.

In the example presented, the internal failure process is complex because the initial equilibrium state resulted in some post-peak strain softening at a few locations. Further loading (in our case controlled by a rise in water level) resulted in a progressive failure which essentially evolved from the “kink” (sharp or rounded) defined by the geometry of the basal

583 slip surface towards the surface of the slope. Once the last point resisting under peak
584 conditions in the internal shear band advances into strain softening, the failure mechanism
585 becomes kinematically admissible. Afterwards, the slide acceleration begins.

586 An interesting outcome of the analysis presented is that the resulting kinematic mechanism for
587 the sharp kink of the basal surface is essentially defined by a single localization plane which,
588 after some displacement, can be approximately defined as the bisector of the two planes
589 defining the basal sliding surface. This outcome was not obvious because many kinematic
590 mechanisms may explain the motion of the geometry analysed. However, details of the
591 precise continuity and irregularities of the sliding plane will most likely control the geometry of
592 the kinematic mechanism and the position, orientation and number of internal shearing
593 surfaces. This comment was also illustrated in the paper when a more realistic rounded basal
594 sliding surface was analyzed. In this case, two distinct internal shearing surfaces develop in the
595 rock mass.

596 The calculated thickness of the internal shearing planes was significant (20-50m) but this result
597 may be a consequence of the size of elements of the computational mesh. Note also that the
598 rock was discretised as a homogeneous elastoplastic brittle material with no internal structure:
599 bedding planes and fractures. They could play a significant role in the development of
600 kinematic mechanisms. The computational method (MPM), inherently dynamic, provides
601 interesting information on the post-failure motion of the slide. Velocity and run-out are
602 calculated, as well as the evolving geometry during the motion.

603 There was a special interest in checking if the known velocity and run-out of Vajont landslide
604 could be simulated by a combination of internal rock brittleness and a non-zero friction angle
605 at the sliding surface. In fact, a number of previous analyses which rely on thermal
606 pressurization along the basal sliding surface indicated that a zero shear strength was required
607 to reproduce the actual observation. The present contribution confirms that this is the case,
608 even if a significant brittleness is assigned to the internal shearing and even if the kinematic
609 sliding mechanism is simulated with a reasonable accuracy.

610

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