MODELLING OF FLOWSLIDE TRIGGERING IN PYROCLASTIC SOILS

L. CASCINI, G. SORBINO, S. CUOMO
Department of Civil Engineering, University of Salerno (Italy)

ABSTRACT: In the paper, the triggering mechanisms of a huge flowslide occurring on May 1998 on Pizzo d’Alvano relief located in the Campania Region (South Italy) are discussed. At this aim, geotechnical analyses have been carried out on a slope section of a sample basin for which stratigraphic conditions, physical and mechanical properties of pyroclastic soils were available thanks to careful in situ and laboratory investigations. Geotechnical analyses have been concerned with the modelling of transient seepage regime both in saturated and unsaturated conditions; the resulting computed pore pressure distributions have then been utilized for the stability analyses. The obtained results have confirmed that bedrock outlet is a key factor in the triggering of the flowslide, as evidenced by preliminary geotechnical analyses and a geological slope evolution model, capable of interpreting past and recent flowslides occurred on the Pizzo d’Alvano massif. Moreover the results evidence the relevant role played by the pumice soil layers and, more generally, by the transmissivity of the pyroclastic cover in order to achieve a simulation of time sequence of slope failure in good agreement with eyewitnesses descriptions.

Keywords: Flowslide, pyroclastic soils, unsaturated soils, numerical modelling

1 INTRODUCTION

Flowslides can be surely considered as one of the most dangerous slope movements, for their capability to produce casualties and remarkable economic damage (Sassa 1998). Such phenomena are widespread in many countries (Jones 1973; Costa, Baker 1981; Ellen, Fleming 1987; Sassa 1988; Takahashi 1991) and they involve different kind of soils, generally in a loose state, which in the post failure stage collapse and rapidly reach the toe of the slope (Costa, Wieczoreck 1987; Hutchinson 1988); the initial mobilised mass often increases during its path downslope either by inducing additional slope failure and/or by eroding the stable in place soils (O.U. 2.38 1998a; Picarelli 1999).

Significant examples of this type of slope movements are those periodically occurring in the Campania Region (South Italy) triggered by critical rainfall events (O.U. 2.38 1998a; Rossi, Chirico 1998; Versace 2001). They involve unsaturated pyroclastic soils - originated by the explosive phases of the Somma-Vesuvius volcano – which mantle the limestone and tuffaceous slopes over an area of about 3000 km².

In this area, there are more than 200 towns frequently suffering flowslides, as pointed out by historical data acquired over a period from the 16th century up to the present (O.U. 2.38 1998a; Cascini et al. 2000). One of the most calamitous events occurred on May 1998, when 159 casualties and serious damages were recorded in the four little towns (Bracigliano, Quindici, Sarno and Siano) located at the toe of the Pizzo d’Alvano relief (Fig. 1).

After this event, the Department of Civil Protection entrusted the emergency scientific management to the University of Salerno where many activities were undertaken. Some activities were devoted to furnish, in a very brief time, answers to practical questions; the aim of the others was oriented to systematically deepen knowledge on geological and triggering factors controlling the flowslides occurrence (O.U. 2.38 1998a; Cascini et al. 2000). In particular, over the whole area of Figure 1, careful geological, geomorphological and hydrogeological investigations were carried out at different scales (1:25.000 and 1:5.000). It was, thus, possible to set up a slope evolution model capable of interpreting - from a geological point of view - past and recent flowslides. Preliminary geotechnical back-analyses for the failure conditions of a particularly huge flowslide, were also carried out (Cascini et al. 2000) with the aid of some preliminary results coming from the geotechnical characterization of pyroclastic soils. For the goodness of the obtained results, detailed geotechnical investigations were planned in order to deepen the mechanical properties of the involved soils, their stratigraphic conditions and the in-situ soil suction regimen.

In this paper, after a brief description of the previous geological and geotechnical analyses performed and of the in situ and
laboratory investigations aimed at the mechanical characterization of pyroclastic soils, a more appropriate geotechnical modelling of the failure mechanisms will be illustrated, and the role of the relevant factors affecting flowslide triggering will be outlined.

2 PREVIOUS ANALYSES AND GEOTECHNICAL INVESTIGATIONS

At the beginning of May 1998, the particular meteorological conditions affecting the territory of the Campania Region resulted in a severe rainfall event throughout the area of Figure 1. In particular, rain-gauges located at the toe of Pizzo d’Alvano massif recorded, for the period April 27 – May 5, 1998, a cumulated rainfall value of 160 mm, of which 120 mm fell in the last two days (Fig. 2).

During and soon after this event, within an interval of about ten hours, flowslides occurred in almost all the basins of the massif, involving pyroclastic soil covers. Surveys conducted following occurrence of the flowslides revealed that triggering zones were located primarily in the upper parts of such basins, characterized by an average slope angle varying from 35 to 41 degrees and a soil thickness ranging from 0.5 m to 5.0 m. In most of these zones, multiple failure phenomena occurred, which progressively involved larger portions of the slope, according to mechanisms and time sequences not easily identifiable after the event occurrence (Cascini et al. 2002). This consideration, together with the relevant extension of the territory affected by flowslide phenomena and the need to provide an answer, in a short time, to the difficult and urgent demands of the Department of Civil Protection, pointed out the necessity to set up an interdisciplinary research program (O.U. 2.38, 1998) to investigate geological and triggering factors of flowslides. The main objectives of this ongoing program were the following: assessment of geological, geomorphological and hydrogeological features of Pizzo d’Alvano massif, at different scales (1:25.000 and 1:5.000); geotechnical analysis of flowslide triggering at a more detailed scale (1:2.000); mechanical characterization of pyroclastic soils; assessment of in situ negative pore pressure regime.

With reference to the geological characteristics, the investigations and studies carried out, together with the information available in the literature, have evidenced that the massif consists of a sequence - reaching a thickness of several hundred meters - of limestones, dolomitic limestones and, subordinately, marly limestones, dating back from Lower to Upper Cretaceous ages.

Pyroclastic soils, coming from Somma-Vesuvius volcanic activities, cover diffusely the limestone slopes, both as primary air-fall deposits and re-worked deposits (vulcanolastic deposits). In the upper portion of the slopes pyroclastic soils have, generally, a small thickness, ranging from some decimetres up to a maximum of 6-7 m; moving downslope, the soil thickness gradually increases, reaching values up to 20 m in the morphological concavities, in the karstic depressions and at the toe of the valleys (Cascini et al. 2000). Stratigraphic conditions are almost everywhere highly variable, although they are generally characterized by alternated layers of pumiceous soils and of ash soils, sometimes with the presence of paleosoil horizons.

As for geomorphological features, Pizzo d’Alvano massif is NW-SE oriented morphological structure. The slopes outline is influenced by structural factors (minor faults and jointing) as well as by morphological frames derived from limestone layers more resistant to erosion. Above the morphological frames, the paleo-drainage network of the limestone slopes are mainly filled with air-fall deposits or with debris colluvial material coming from upstream slopes, so forming the so-called “Zero Order Basins” (ZOB) (Cascini et al. 2000; Guida 2003). Alluvial fans of various ages are located at the toe of the valleys, highlighting the systematic occurrence of depositional events, coming from pyroclastic flows, superimposed on older debris deposits.

By comparing the maps produced according to the acquired geological and geomorphological data with slope portions affected by flowslides of May 1998 (Fig. 1), it has been observed that the majority of triggering zones are located inside the ZOB areas and that the final destination of the mobilized soil masses resulted in the alluvial fans areas described above. On the basis of these results, additional in-situ investigations and studies were devoted to the hydrogeological aspects of the Pizzo d’Alvano massif. The obtained results indicated, for the upper part of the
massif, a subsurface flow system characterized by the presence of perched water tables, whose flow regimen is strictly connected to the arrangement of the main and secondary structural lineaments, acting as relative impermeable layers (Guida 2003). These structural lineaments, shaped as wedges, are closely associated to the seepage and temporary springs (or outlets) located in the upper parts of the slopes (Fig. 3). Temporary springs are mainly located inside the ZOB, and at the top of the filled main catchment basins (Fig. 3b). Seasonal springs can even be observed at the toe of morphological discontinuities, in correspondence with similar, although upper ordered, hydrogeological structures.

All the above acquired information allowed the setting up of a slope evolution model (Cascini et al. 2000; Guida 2003) which is capable of interpreting, from a geological point of view, the past and recent flowslides occurred on the Pizzo d’Alvano massif; among the other features, such a model evidences the important role played by the bedrock outlets in the triggering of the flowslides (Cascini et al. 2000).

In order to evaluate the reliability of this model, a preliminary back-analysis (Cascini et al. 2000) was performed for the failure conditions of a huge flowslide which occurred on May 5, 1998 inside a ZOB area of Sarno slopes (Fig. 4a). Geotechnical analyses were carried out on the slope section A-A shown in Figure 4a for which the stratigraphic outline and the mechanical properties of the involved soils were based on the scarce in-situ and laboratory data available at that time. In particular, for the whole slope section, the pyroclastic soils above the limestone bedrock were assumed to be of constant thickness and composed of three horizons (Fig. 4b), respectively equal to 1.2 m (upper ashy layer), 0.3 m (pumice layer) and 0.5 m (lower ashy layer).

Despite the above assumptions, the results obtained from the geotechnical analyses – conducted in a similar manner to those illustrated in the next section – were particularly encouraging as they could simulate, for a given set of mechanical properties, a time sequence of flowslide triggering which was in a good agreement with that observed by witnesses at the time of failure (Cascini et al. 2000). Moreover, the performed analysis confirmed the relevant role played by bedrock temporary springs and highlighted the presence of negative pore pressures in many zones of the pyroclastic covers, at the occurrence of the failure.

On the basis of the above results, it was considered worthwhile to perform further in-situ and laboratory investigations in the sample area shown in Figure 4a as well as in other triggering areas of the Pizzo d’Alvano massif. Such investigations were aimed to acquire detailed data on: stratigraphic conditions inside and outside the ZOB areas; negative pore pressure regimen; mechanical properties of pyroclastic soils, both in saturated and unsaturated conditions.

As regards stratigraphic conditions in the sample area, seismic refraction prospects and more than 60 pits were performed (Fig. 4a). The overall collected data allowed the definition of very accurate stratigraphic outlines of in-situ soils, both along many verticals and slope sections. In particular, it was assessed that the maximum thickness of pyroclastic soils does not exceed 5.0 m and that their stratigraphic conditions are highly variable according to the different colluvial processes experienced by the various zones of the sample area. On the basis of the above results and with the aid of geomorphological analyses performed at large scale for the area of Figure 4a (Guida 2003), a more appropriate reconstruction of the stratigraphic conditions of the slope section A-A, before the flowslide event, has been carried out. The modelled slope section is shown in Figure 5 together with the stratigraphic details sketched for some verticals. As it can be seen, the pyroclastic soil cover is characterized by variable thickness, ranging from 2.0 m to 5.0 m, and presents an irregular pattern below the ground surface, not exceeding, however, 3.1 m.

With reference to negative pore pressure regimen inside the pyroclastic covers, more than 3000 measurements of matric suction have been performed, in many sites over the whole area shown in Figure 1 (Cascini, Sorbino 2002; 2003). In the above sites, measurements started on November 1999 and they were collected at different altitudes, both inside and outside the ZOB areas by using portable tensiometers (Quick-Draw tensiometers) and fill-in-place tensimeters; the depths investigated ranged between 0.2 m and 4.0 m from the ground surface.

In Figure 6 the overall suction data have been plotted against time, for the period November 1999 – April 2002, together with the daily rainfall recorded in the same period. As it can be readily observed, suction data show a significant scattering which is, of course, related to the different sites and depths where measurements were performed. However, as evidenced by Cascini and Sorbino (2003), data scattering quite disappears if monthly average suction values, at the same depth, are considered. Following this consideration, Cascini and Sorbino (2003) performed a detailed analysis of soil suction data on a monthly basis, independently of the year in which measurements were taken. The results obtained evidenced that monthly average suction values are characterized by the same behaviour at yearly scale, independently from the measurement site. In particular, monthly suction values attain their lowest values of 10-15 kPa in the January-March period at all investigation sites and at any depth. This circumstance evidences that, all over the Pizzo d’Alvano massif, the monthly suction regimen seems to follow a regular trend during the wet season. Therefore, it cannot be excluded that such conditions did characterise large portions of the pyroclastic covers during the early months (January – February) of 1998.

In order to determine the mechanical properties of the pyroclastic cover, soil samples were collected, at several dates, in the...
sample area of Figure 4a as well as in other triggering areas belong to Pizzo d’Alvano slopes facing Sarno town. For all the samples, physical properties have been determined, while the mechanical properties have been investigated only for the ashy soils, due to the impossibility of collecting undisturbed samples of pumice layers. In particular, as far as the physical properties are concerned, tests for the grain size distribution, Atterberg limits and index properties have been carried out (Bilotta, Foresta 2002; Sorbino, Foresta 2002). Mechanical characterization has dealt with hydraulic properties and shear strength, both in saturated and unsaturated conditions (Sorbino, Foresta 2002). Hydraulic properties in saturated conditions have been investigated by means of permeameter tests, and by means of three different laboratory equipments – i.e. Suction Controlled Oedometer, Volumetric Pressure Plate Extractor and Richards’ Pressure Plate – with reference to the unsaturated conditions. Finally, the shear strength has been evaluated by means of conventional direct shear tests and suction controlled triaxial tests (Bilotta et al. in prep.).

With reference to grain distributions of investigated soils (Fig. 7) it can be observed that the ashy soils are characterized, on average, by a usually prevailing sandy component and non-plastic silt which occasionally reaches percentage greater than 50%; these soils also includes some gravel, due to the presence of small isolated pumices, in percentages always lower than 22%. The pumice layers, instead, are characterized by a coarser grain size distribution and a gravel percentage up to 90%.

The specific gravity of both types of soil is always quite low due to the presence of small internal voids within individual soil particles. Porosity is very high and ranges, on average, between 0.61 and 0.69. The saturation degree is always quite low, independent of depth. The above mentioned physical properties make these pyroclastic soils a very light material, with a total unit weight ranging from 9.02 and 12.04 kN/m³.

The hydraulic properties of the ashy soils show values of saturated hydraulic conductivity ranging from a minimum of $5.0 \times 10^{-6}$ m/s to a maximum of $4.8 \times 10^{-5}$ m/s. For the pumice layers, the data available in literature (Pellegrino 1967) for soils of analogous origin furnish saturated hydraulic conductivity ranging between $1.0 \times 10^{-5}$ m/s and $1.0 \times 10^{-2}$ m/s. As for the hydraulic properties in unsaturated conditions are concerned, the experimental values of volumetric water content and hydraulic conductivity are both plotted against suction in Figure 8; in the same Figure the unsaturated hydraulic characteristics of the pumice soils are also reported, whose determination has been carried out numerically (Green, Corey 1971), on the basis of their grain size distribution curves.

With reference to strength properties, direct shear tests on undisturbed specimens in saturated conditions were performed.
and the Mohr-Coulomb failure envelope revealed values of effective friction angle ranging from 30 to 37 degrees and null values of cohesion intercept. As regards the shear strength in unsaturated conditions, it is well known that it can be modelled by means of the extended Mohr-Coulomb failure envelope proposed by Fredlund et al. (1978), furnished by the following expression:

$$\tau = c' + (\sigma - u) \gamma' + (u - u_w) \gamma_{sat}$$  \hspace{1cm} (1)

where $\tau$ is the shear strength, $c'$ is the effective cohesion, $(\sigma - u)$ is the net normal stress at failure on the plane of failure, $(u - u_w)$ is the matric suction and $\gamma_{sat}$ is the angle of shearing resistance with respect to matric suction.

3 MODELLING OF FLOWSLIDE TRIGGERING AND RESULTS

The previously synthesized data allow the carrying out of a more appropriate geotechnical analysis of the failure mechanisms affecting flowslide in the sample area of Figure 4a. In fact, previously not available information (Cascini et al. 2000) is at hand today, coming from detailed in situ and laboratory investigations. The collected data allow a better definition of the stratigraphic conditions of the soils before the flowslide occurrence as well of the mechanical soil properties and in situ soil suction.

With the aid of these data, geotechnical analyses have been carried out on the slope section of Figure 5 assuming for the soil layers are homogenous and isotropic. In order to determine the $\phi^b$ angle Bilotta, Foresta (2002) and Bilotta et al. (in prep.) performed a series of direct shear tests at natural water content as well as triaxial tests under various suction ranging from 30 to 37 degrees and null values of cohesion intercept. As regards the shear strength in unsaturated conditions, it is well known that it can be modelled by means of the extended Mohr-Coulomb failure envelope proposed by Fredlund et al. (1978), furnished by the following expression:

$$\tau = c' + (\sigma - u) \gamma' + (u - u_w) \gamma_{sat}$$  \hspace{1cm} (1)

where $\tau$ is the shear strength, $c'$ is the effective cohesion, $(\sigma - u)$ is the net normal stress at failure on the plane of failure, $(u - u_w)$ is the matric suction and $\gamma_{sat}$ is the angle of shearing resistance with respect to matric suction.

In order to determine the $\phi^b$ angle Bilotta, Foresta (2002) and Bilotta et al. (in prep.) performed a series of direct shear tests at natural water content as well as triaxial tests under various suction ranging from 30 to 37 degrees.

3 MODELLING OF FLOWSLIDE TRIGGERING AND RESULTS

The previously synthesized data allow the carrying out of a more appropriate geotechnical analysis of the failure mechanisms affecting flowslide in the sample area of Figure 4a. In fact, previously not available information (Cascini et al. 2000) is at hand today, coming from detailed in situ and laboratory investigations. The collected data allow a better definition of the stratigraphic conditions of the soils before the flowslide occurrence as well of the mechanical soil properties and in situ soil suction.

With the aid of these data, geotechnical analyses have been carried out on the slope section of Figure 5 assuming for the physical and mechanical properties of the soil layers, those listed in Table 1.

For the above slope section, analyses have primarily concerned the modelling of the seepage conditions inside the pyroclastic cover, in order to predict pore pressure changes during the period extending from January 1, 1998 to May 5, 1998. The computed pore pressure distributions have been utilized as input data for the stability analyses.

Seepage analyses have been performed using the commercial finite element software SEEP/W (Geo Slope Int. Ltd. 1998) able to analyse two-dimensional transient flow regime both in saturated and unsaturated conditions. The governing differential equation utilised in the software has the following expression:

$$\frac{\partial}{\partial x} \left( k_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( k_y \frac{\partial h}{\partial y} \right) = m_w^{\prime} \gamma_{sat} \left( \frac{\partial h}{\partial t} \right)$$  \hspace{1cm} (2)

where $m_w^{\prime}$ is the coefficient of volumetric water changes with respect to a change in negative pore pressure and is equal to the slope of the soil–water characteristic curve, $h$ is the total head, $k_x$ and $k_y$ are the coefficient of permeability of the soil with respect to water, $\gamma_{sat}$ is the unit weight of water, $t$ is the time. The finite element mesh used for the numerical modelling of the seepage conditions resulted in about 4000 quadrilateral elements having lengths lower than 1.0 m.

The solution process of the non-linear differential equation (2) requires the definition of the soil water characteristic curve, the permeability function, the boundary and initial conditions. In the case of the analysed slope section, hydraulic properties have been selected among those experimentally determined (Fig. 8) assuming the soil layers are homogenous and isotropic. In order to define the boundary conditions on the ground surface, a preliminary analysis has been performed on the rainfall events recorded over a period of two years (2000 – 2002) at rain gauges located at the toe of the slope (installed before May 1998) and at those installed at the top of Pizzo d’Alvano massif after May 1998. The above analysis has evidenced that the amount of accumulated rainfall recorded at the toe and at the top of Pizzo d’Alvano massif does not differ significantly when computed over a period equal or longer than a month. On the contrary, relevant differences arise, if rainfall intensities recorded during each rainstorm event are compared. In particular, in the analysed period, the ratio between hourly rainfall intensities recorded at the top rain gauges and those recorded at the toe rain gauges is higher than unity in the 56% of the recorded rainfall events.

Following the above results, a flux boundary condition equal to daily rainfall intensities recorded at the toe rain gauges has been assumed on the ground surface for the period January 1, 1998 - May 3, 1998; for the last two days (May 4-5) the flux boundary condition has, instead, assigned hourly rainfall intensities, with values equal to two times those recorded at the toe rain gauges. As to the boundary conditions at the contact between the pyroclastic cover and the limestone bedrock, two different cases were considered. In the first case, the bedrock was treated as impervious; in the second, this impervious condition was replaced by a flux condition only at the bedrock outlet (Fig. 5).

Following the previously stated considerations on suction measurements, a uniform distribution of soil suction has been adopted as an initial condition, assuming values equal to 5, 10, 15 and 20 kPa all over the pyroclastic cover.

As regards stability analysis, the Janbu’s simplified limit equilibrium method was used to compute the distribution of minimum safety factor. This has been determined taking into account more than 170 potential slip surfaces, of different exten-

Table 1. Geotechnical characterization of soil layers

<table>
<thead>
<tr>
<th>Soil properties</th>
<th>Upper ashy soil layer</th>
<th>Pumice soil layer</th>
<th>Lower ashy soil layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight</td>
<td>$\gamma_d$</td>
<td>17.2 kN/m$^3$</td>
<td>14.4 kN/m$^3$</td>
</tr>
<tr>
<td>Saturated unit weight</td>
<td>$\gamma_{sat}$</td>
<td>13.1 kN/m$^3$</td>
<td>13.1 kN/m$^3$</td>
</tr>
<tr>
<td>Porosity</td>
<td>n</td>
<td>0.58</td>
<td>0.69</td>
</tr>
<tr>
<td>Saturated hydraulic conductivity</td>
<td>$k_{sat}$</td>
<td>$10^{-3}$ m/s</td>
<td>$10^{-3}$ m/s</td>
</tr>
<tr>
<td>Effective cohesion</td>
<td>$c'$</td>
<td>4.7 kPa</td>
<td>0 kPa</td>
</tr>
<tr>
<td>Friction angle</td>
<td>$\phi^b$</td>
<td>32°</td>
<td>37°</td>
</tr>
<tr>
<td>Rate of increase in shear strength due to suction</td>
<td>$\phi^b$</td>
<td>20°-30°</td>
<td>20°-30°</td>
</tr>
</tbody>
</table>

Figure 8. Unsaturated hydraulic properties of some ashy and pumice soils in the ZOB areas of Sarno (from Sorbino, Foresta 2002, modified).

Volumetric water content $w$

<table>
<thead>
<tr>
<th>Suction (kPa)</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$</td>
<td>0.1</td>
<td>0.2</td>
<td>0.3</td>
<td>0.4</td>
<td>0.5</td>
<td>0.6</td>
<td>0.7</td>
<td>0.8</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Continued...
sion, located at variable depths along the analysed slope section; at this slope the commercial software SLOPE/W (Geo Slope Int. Ltd. 1998) was used. In correspondence with the attainment of instability conditions, the collapsed portion of the pyroclastic soil was removed and the safety factor was re-computed taking into account the new slope geometry; in this way, a possible time sequence of the failure stages occurring along the slope section could be simulated.

By adopting the above described procedures, the first step of the analysis has been concerned with the simulation of seepage conditions in the period January 1, 1998 – May 3, 1998. Figure 9 shows, for some verticals along the slope section, the computed pore pressure distributions on May 3, 1998 assuming different initial conditions and a value of 7.5×10^5 m/s for the saturated conductivity of pumice layers; in the same Figure is also reported the position of the slip surface (broken line) observed after the flowslide occurrence.

As it can be noted (Fig. 9), soil layers located above the slip surface are characterized by partially saturated conditions, with negative pore pressure distributions quite uniform, not affected by the different initial conditions adopted. On the slip surface and at depths approaching the limestone bedrock, a generalised increase of pore pressure can be observed with attained values strictly dependent on the initial condition, especially in the middle part (verticals A and B) of the slope section (Fig. 9), where pumice layers are more frequent. Here, the initial condition is capable of inducing or not the formation of a perched water table inside the pyroclastic cover. It is interesting to note that the pore pressures distributions shown in Figure 9 do not differ significantly if they are computed assuming, for the ashy layers, hydraulic properties varying in the range of those experimentally determined.

A possible explanation of this behaviour, as well as the influence of the initial conditions on the pore pressure distribution, can be almost certainly ascribed to the role of pumice layers and, in particular, to their hydraulic properties. As Figure 8b shows, although saturated conductivity of pumice soils is highest among the soils forming the slope section, its conductivity decreases more rapidly than the other layers with respect to suction; in particular, the pumice conductivity show a variation of about two orders of magnitude with respect to suction when this last varies between the minimum (5 kPa) and the maximum (20 kPa) value assigned as initial condition. So, for the case of an initial suction of 5 kPa, pumice layers are able to convey relevant amounts of water towards larger depths; according to water movement, pore pressures increase at large depth with respect to initial value. For the case of an initial suction of 20 kPa, pumice layers behave as low permeability layers; meaning that smaller amounts of water flow deeper into the slope and thus negative pore pressure at large depths are not highly affected.

The relevant role played by pumice soils on the seepage conditions and on the consequent pore pressure distributions in the slope section, has been also confirmed by a sensitivity analysis carried out with respect to their saturated conductivity for which, as previously stated, data were obtained from literature. Figure 10 shows pore pressures computed at May 3, 1998 along the slip surface observed after flowslide occurrence, assuming an initial suction condition of 10 kPa and saturated conductivity values ranging between 5.0×10^7 m/s and 5.0×10^8 m/s. As it can be seen, computed pore pressures differ significantly with respect to saturated conductivity with variations up to 15 kPa in the central and lower part of the slope.

The pore pressure variations just before the flowslide occurrence called for a close examination of the influence of both pumice saturated conductivity and initial condition on the stability condition of the slope at that time. In Figure 11 the minimum safety factor computed for the various zones of the slope (Fig. 5) are reported, assuming a zero value for q and an initial condition of 10 kPa for pore pressure analysis. As it can be seen, whatever is the saturated conductivity of pumice layers, the entire slope is characterized by stable condition as safety factor as-
stability has been evaluated by comparing safety factors for the various zone of the slope assuming for \( \phi^b \) values obtained from laboratory tests. As Figure 12 shows, assuming 10kPa as initial condition at January 1, 1998 and a saturated conductivity of 7.5\( \times 10^{-5} \) m/s for pumice soil layers, the increase of safety factor taking into account the unsaturated conditions ranges from 4% to 20% over the whole slope while there aren’t significant differences assuming 20\(^\circ\), 25\(^\circ\) or 30\(^\circ\) for \( \phi^b \).

On the basis of these results further analyses have been carried out in order to evaluate the influence, on the instability conditions, of the bedrock outlet, as evidenced by the slope evolution model discussed in section 2. For this scope, seepage analyses, for the period January 1, 1998 – May 5, 1998, have been re-performed removing the condition of an impervious boundary at the base of pyroclastic cover where bedrock outlet is located. Here, a flux boundary condition has been assigned and considered operating during various time intervals within the period April 28, 1998 – May 5, 1998; according to Cascini et al. (2000), the outlet flow rate has been assumed, equal to 1.0\( \times 10^{-3} \) m\(^3\)/min.

Figure 14 shows the most relevant results obtained for the evaluation of the slope stability conditions, assuming \( \phi^b \) equal to zero in order to compare the results of the present analysis and those of the preliminary one (Cascini et al. 2000). First of all, it can be observed that the presence of bedrock outlet is a key factor in order to assess the attainment of failure condition for the whole slope. However the time sequence of the slope failure is strongly affected by the combination of both initial condition assumed at January 1, 1998 and the period in which bedrock outlet operates.

In particular, the best simulations of the recorded times of failure can be obtained by assuming, for the month of January, an initial suction value ranging within the interval 10 – 15 kPa, considering a value of saturated conductivity equal to 7.5\( \times 10^{-5} \) m/s and the bedrock outlet to have become operative between May, 2 - 3 (Fig. 14).

It is worthwhile to observe that the above results are in good agreement with those achieved by Cascini et al. (2000) with reference to a simplified geotechnical section of the slope and assuming for pumice soil layer a saturated conductivity value equal to 10\^-7\) m/s. Such agreement can be explained by computing the saturated transmissivity of the section considered by the above Authors and that utilized in the present study (Fig. 5).

Authors and that utilized in the present study (Fig. 5).
As well known, saturated trasmissivity for a stratified slope is given by the expression:

\[ T = \sum_i k_i b_i \]  

where \( k_i \) and \( b_i \) are, respectively the saturated conductivity and the thickness of the layer \( i \). According to (3), in Figure 15 the trasmissivities of the two sections are compared. As it can be noted, despite the different stratiographic outline and thickness of the two geotechnical sections, their trasmissivities are very close. This, in turn, evidences the relevant role played by pumice layers, which strongly influence trasmissivities, on the attainment of instability condition for pyroclastic cover.

4 CONCLUDING REMARKS

In recent years, flowslides occurring in pyroclastic soils within the territory of the Campania Region have increased in frequency and size, drawing considerable attention. Because of the almost total absence of significant knowledge on the subject, following the calamitous events of May 1998, it was considered necessary to set up a large-scale research program. Initially, geological studies were preferred in order to assess the main physical features of the territory where the flowslides occurred. At a subsequent stage, geotechnical analyses were performed to validate the results of previous studies from an engineering point of view.

These geotechnical analyses, based on preliminary data from in situ and laboratory investigations, confirmed most of the hypotheses suggested by the previous geological studies. In consideration of this, a more detailed in situ and laboratory investigation program was planned. It was thus possible to improve the geotechnical analyses aimed to assess the role played by some factors controlling failure occurrence.

The obtained results stress that the bedrock outlet, previously highlighted by the geological slope evolution model, represents a key factor in the triggering of the analysed flowslide; in fact, only by taking into account the presence of bedrock outlet, the modelled sliding surface and resulting thicknesses of mobilised pyroclastic soil match the in situ evidence observed immediately after the event. Moreover, a significant agreement between the simulated time sequence and the one deduced from the descriptions of eyewitnesses can be obtained by assuming, in the seepage analyses, an initial suction value of 10 kPa and a saturated conductivity of pumice layers equal to a mean value (7.5 \( \times 10^{-5} \) m/s) of the range documented in the literature. Obviously, the full confirmation of such results needs further analyses to be performed in order to investigate the role of other factors, such as \( \phi \) angle, on the time sequence of the slope failure.

5 REFERENCES


Guida, D. 2003. The role of Zero-Order Basins in flowslides-debris flows occurrence and recurrence in Campania (Italy). This Conference.


