RELEVANCE OF THREE-DIMENSIONAL STABILITY ANALYSIS FOR TWO LANDSLIDES IN SOUTHERN ITALIAN ALPS

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ABSTRACT

The design of landslide stabilization interventions requires a careful analysis and modelling of the deformation mechanisms occurring in the slope. In some cases, the classical two-dimensional (2D) stability calculation may lead to erroneous estimate of slope safety margin and therefore to incorrect design of stabilization works. Two different lanslides, both located in the Southern Italian Alps, have been back-analysed using 2D and three dimensional (3D) limit equilibrium methods, showing the importante of 3D effects in the estimate of safety factor necessary for a proper design of stabilization actions.

Keywords: Landslides, Stability analysis, Stabilization interventions.

THE LANDSLIDE “VRIDEL” IN WHEATHERED PHYLLITES

Morphological, geological and geotechnical features

As a consequence of heavy spring rainfalls, in the Mocheni Valley in South-Eastern Trentino (Figure 1), a landslide began to move in early May 2002, at a rate of about 5 cm/day, thus interrupting the circulation on a road crossing the slope.

The main features of landslide may be observed in the map of Figure 2, while a view of the collapsed mass is provided in Figures 3. The landslide is funnel-shaped with an average

Fig. 1. Localization of two landslides.

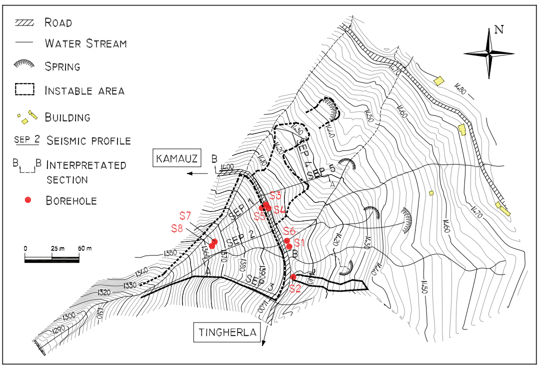

Fig. 2. Topographic map of area with localization of boreholes and seismic profiles.

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Fig. 3. General view of Vridel landslide.

Fig. 4. Uncovered micropiles.

inclination of 30° and involves an area 130 m long and 125 m large, limited at the top by the road and laterally constrained by two rock spurs.

Some other small shallow erosion occurrences, due to the emergence of small wellsprings, interested the slope in the part above the road. Rainfall and spring water are collected in three (small) streams converging at the funnel apex in a unique stream, referred as to Vridel creek, discharging the waters downwards into a steeper and narrow valley excavated in the rock.

In order to prevent additional road movements, as rapidly as possible, a system composed of vertical and 30° inclined micropiles was designed and installed on the downward side of the road: then the road was levelled off with a refill and paved again.
Micropiles was designed on the base of a first geotechnical and geophysical investigation performed along the road, consisting in two seismic profiles, (parallel and transversely crossing the slope, see sections A-A and B-B in Figure 3) and two boreholes (S1 and S2).

Despite the reinforcement system, new significant displacements occurred in the January 2003, causing another drop of the road and the uncovering of micropiles (Figure 4). Therefore, more careful geotechnical and geophysical investigations were planned and carried out to characterize soil profile and properties and to understand the basic sliding mechanism, for a proper design of remedial interventions. The position of boreholes and seismic survey lines are also depicted in Figure 2, while two soil profiles obtained with the seismic refraction technique are depicted in Figure 5.

The outcropping rocks present in the Vridel basin are phyllites, i.e. talco-schists belonging to a metamorphic formation highly tectonized during the Alps formation. The site investigations showed that the two rock spurs dip laterally towards the central longitudinal axis of landslide and, in the instable area, the bedrock is covered by a layer of highly weathered rock, with a RQD equal 25-50% with thickness varying between 5 and 10 m. The upper 3-14 m thick colluvium is composed by rock fragments in a brown-grey silty matrix (residual production of phyllites weathering) together with boulder and cobbles in brown-red matrix (sediments transported by glaciers). The bedrock, characterized by a RQD equal to around 80%, was localized at a relatively constant depth of 17 m in boreholes S1, S4, S5 and S8; only in S3 it was occasionally found at 9 m.

The groundwater depth was found at 3-5 m in the colluvium and at 12-13 m in wheathered rock, suggesting that a perched groundwater regime exists in the more permeable colluvium, probably recharged by the rainfall and sustained by the impermeable stratum of altered rock.

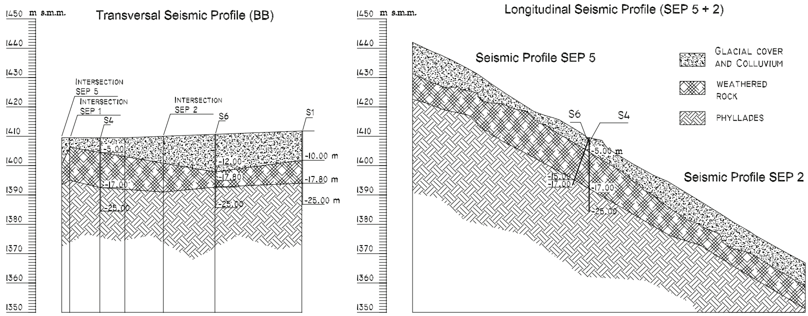

Fig. 5. Transversal and longitudinal seismic profiles.

This upper groundwater flow coexists with a deeper groundwater circulation in the more permeable bedrock.

The colluvium and the altered phyllites are composed of 30-40% gravel, 30-45% sand, 20-35% silt with a small amount of clay (Clay Fraction CF = 1-4%, occasionally equal to 10%). A relative higher fine content (silt and clay fraction) seems to be present at major depths. The fine material is characterized by Liquid Limit LL = 20-25 and Plasticity Index PI = 1-3. The shear tests on the fine matrix provided a shear strength characterized by a friction angle of 30-34° with cohesion equal to 0-13 kPa.

Back-analysis of slope instability

According to the longitudinal seismic profile of Figure 5 and the above groundwater regime, the most probable instability mechanism is a translational sliding movement along surfaces parallel to the slope and located at relatively shallow depth into the colluvium or weathered phyllites. An analysis of such sliding mechanisms, using the classical limit equilibrium approach and accounting for the laboratory measured shear strength of involved soils, does not match the site evidence.

This is clear observing the values of safety factory *FS* reported in the Table 1, which are determined using the 2D limit equilibrium analysis for plane translational slip according to the relationship:

$$FS = \frac{c' + (\sigma_{vo} - u)\cos^2\beta \tan\phi'}{\sigma_{vo}\sin\beta\cos\beta} \qquad (1)$$

Sliding surface depth z (m)	**Safety factory *FS***	
	Water head at 3 m depth	
	ϕ'=30° c'=0 kPa	*ϕ'=30°* c'=15 kPa
7	**0,70**	**0,97**
12	**0,63**	**0,74**

Tab.1 Safety factory determined in 2D limit equilibrium analysis.

where c' and ϕ' are respectively the cohesion and the friction angle, β is the dip angle of the slope (30°) and σ_{vo} and u are the geostatic vertical stress and the pore pressure acting at the sliding surface, determined assuming a soil saturated unit weight equal to 19 and 21 kN/m^3, respectively for the colluvium and the weathered phyllites. The factor of safety *FS* obtained for two possible sliding surfaces, one 7 m deep and the other 12 m deep both localized inside

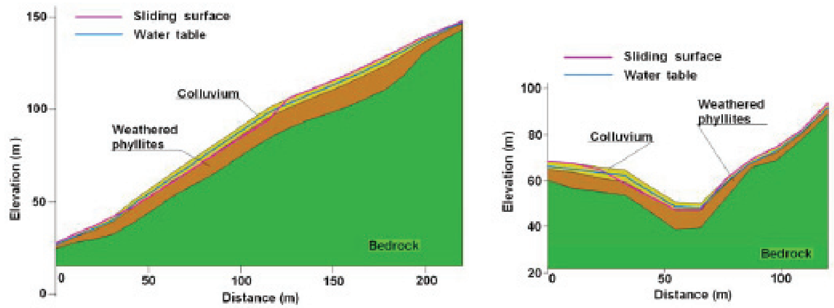

Fig. 6. Analysis with Clara of deepest sliding movement: central long-section and cross section at 42 m distance (from downwards).

the colluvium, is extremely small in all cases and unreasonable for the in-situ observed stability condition.

In order to fit real site stability condition, necessary for subsequent design of remedial work, 3D effects were therefore considered performing two different analysis: one with the Clara software (Hungr, 1987) for 3D analysis and one with a more simplified approach. Both approaches are based on the limit equilibrium (LE) method.

To perform 3D analysis with Clara the geometry of potential sliding surfaces has to be previously hypothesized. In this case, the thickness of colluvium material is assumed to decrease moving from the landslide central part towards the lateral spurs and the narrow valley downwards. Two fully-specified 3D sliding surfaces were considered. Similarly to the 2D analyses, the first one reaches the depth of 7 m in the central part of the landslides whereas at the boundaries intersects only the colluvium. The second surface deepens up to 14 m, intersecting partially the altered phyllites. Figure 6 depicts the central long-section and the cross section at the landslide midpoint, used as a base for the input in the numerical analyses.

In these materials, friction angle ϕ' is known whereas cohesion *c*' is a cumbersome parameter to be evaluated. To determine suitable value of cohesion, sensitivity analysis was carried out varying the latter between 0 and 15 kPa (keeping ϕ'= 30°). Water table was selected according to site measurements, as shown in Figure 6. It is interesting to note in Figure 7, that slope movement along with the shallowest surface is prevented with only *c'*=7.5 kPa whereas, to avoid sliding along the deepest surface, a higher cohesion of at least 14 kPa is necessary. In any cases, both values are in accordance with laboratory measurements on the two classes of materials, suggesting that 3D analysis better describes the limit state of landslide.

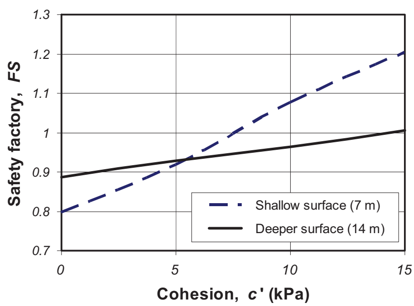

Fig. 7. Sensibility analysis with Clara.

The simplified LE method was properly conceived in order to analyse the basic 3D sliding mechanism (similarly to Clara), but much easier to handle as tool for the stabilization system design. Observing the

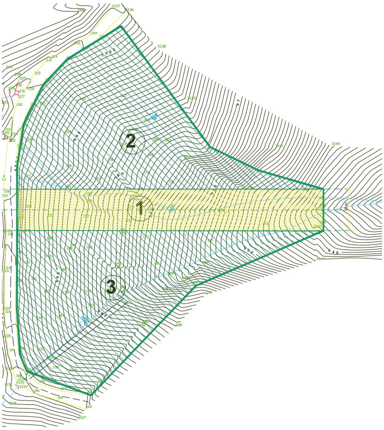

Fig. 8. Subdivision of landslide in three zones.

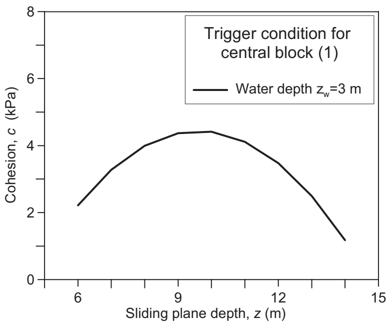

Fig. 9. Sensibility analysis with the simplified LE method.

funnel-shape of landslide (as shown in the map of Figure 8) the instable area below the road may be ideally subdivided in three zones: a central zone (area 1 in Figure 8) formed by a 15 m large and 109 m long strip and two lateral symmetric arc-sectors (areas 2 and 3). Assuming these zones to behave as rigid blocks, the central one can slide downwards only when the shear stress mobilized along the base and lateral constrains overcome the material shear strength. The safety factory may be written as:

$$FS = \frac{F_b + 2F_l + C}{W \sin \beta} \tag{2}$$

where F_b and F_l are the resultant forces due to friction along base and lateral walls of area 1, C is the resultant force due to cohesion along the same surfaces and W is the soil weight. Assuming that the central strip base is a part of the sliding surface located at depth z (from the 30° dipping ground surface), the four above mentioned forces are:

$$F_b = W' \cos \beta \tan \phi \tag{3}$$

$$F_l = L \int_0^z K_o \sigma'_{vo} \, dz \tag{4}$$

$$C = c' \frac{L}{\cos \beta} B \tag{5}$$

$$W = \gamma_{sat} z B L \tag{6}$$

being K_0=1-sinϕ the coefficient of earth pressure at rest and W' the submerged soil total weight of the central block:

$$W' = \left[\gamma_{sat} z - \gamma_w (z - z_w)\right] BL \tag{7}$$

(γ_{sat} = soil saturated unit weight, γ_w = water unit weight, z_w = groundwater table depth and B = strip width and L = strip length).

At constant friction the slope stability is solely a function of the cohesion *c'*, which, in turn, determines the depth *z* of the sliding surface. Therefore, in order to analyse the interrelationship existing between *c'* and *z*, eq. (2) has been solved imposing *FS*=1 and assuming, again, a groundwater regime according to the site measurements. Figure 9 shows clearly that the most critical surface is located at depth of about 10 m (reasonably confirming the results obtained with Clara software). Along this surface the sliding movement occurs for cohesion dropping to 5 kPa or, in other words, a minimum c'=5 kPa is required to assure the stability of the slope.

Remedial works

The stabilization works by the Servizio Sistemazioni Montane of Provincia di Trento started in Summer 2004. They consisted of a combination of channelling of surface water runoff, two planes of sub-horizontal drains and an anchored retaining wall, the latter located at the landslide toe where the slope discharges water in the narrow creek downwards.

Fig. 10. Wooden crossbeams view during the construction.

To channel the free water, the profile of streams crossing the area were set up according to a "*steps and pools*" profile with the construction of short steep open channels with the bottom protected by cemented boulders alternating with wooden crossbeams, that impose to the stream a local energy loss with step of about 1-1.5 m (see Figure 10). This hydraulic system is completed by french drains organized in a chevron layout with the main branch running under the channel bottom covering (Figure 11).

In the upper part of the slope a group of sub-horizontal drains, dipping uphill with an angle of 10°, 35 m long and with 3 m inter-axes distance, were drilled from the road with the aim of draining the seepage water coming from the upper portion of the slope and outcropping in the zone above the road.

Referring to eq. (2), the horizontal force P which has to be applied to the central block of Figure 8 to increase the slope stability factor was determined using the relation:

$$FS = \frac{F_b + 2F_l + C + P}{W \sin \beta} \tag{8}$$

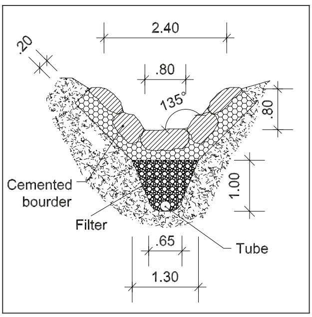

Fig. 11. Typical section of steep channel with the French drain under the bottom.

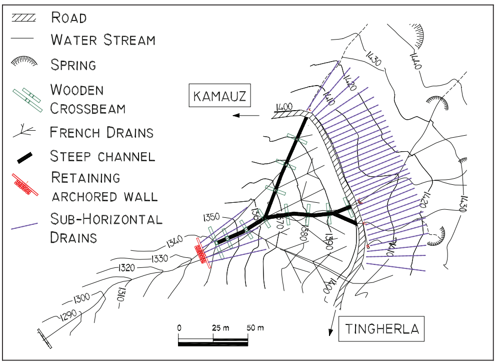

Fig. 12. Layout of remedial work.

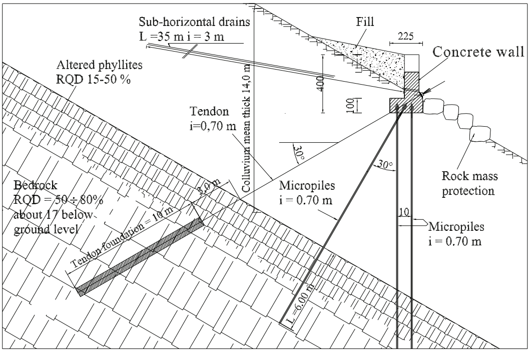

Fig. 13. Section of the anchored wall realized at the base of the slope.

From the above equation, the selected *FS* = 1.10 (used in the design of remedial work) requires a resultant stabilizing force *P* of about 15 MN. The structure used to stabilize the landslide is a 18,5 m large cantilever concrete wall founded on vertical and inclined micro-piles as shown in Figure 13. A system of tendons, anchored deeply in the intact rock, provides the necessary horizontal force *P*.

To protect the creek base from erosion, a permeable fill and rock boulders were set, respectively, at the top and base of the anchored wall. In addition, to drain groundwater from the sliding mass, some sub-horizontal drains were also installed as shown in Figure 13.
From the execution of the above described stabilization work no further sliding movements have been monitored in the whole area.

THE LANDSLIDE "IDRO" IN GRAVELLY SILTS

The Idro landslide (Figure 14) is located in the Provincia of Brescia and is characterized by a very long history. Its origin is traced at the end of Würmian glaciation when, as a consequence of glacier shrinkage, a big rock-fall occurred involving the upper slope in the left side of Val Sabbia: the material collapsed probably occluding the valley, thus forming the beautiful lake of Idro, that extends for about 10 km towards the south-western border between the Trentino and Lombardia.

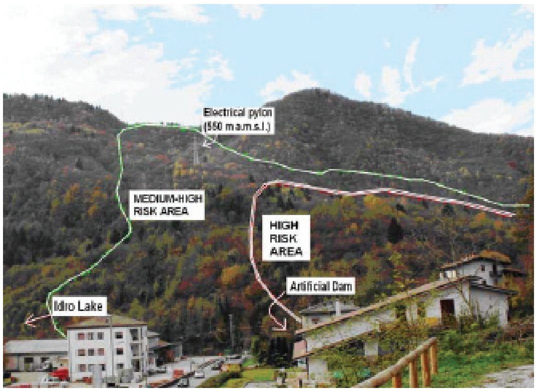

Fig. 14. Front view of Idro landslide with the limits of high risk area.

Before the 2nd world war, an artificial weir was realized on the river bayou at the landslide toe in order to regulate the lake level and use the water for irrigation or energy purposes. The weir was rapidly damaged by the slow movements of the slope, thus revealing the presence of the landslide. The weir was repaired but the slope continued to move at very small rate, hanging out above the river.

A first attempt to analyse the slope stability was carried out by the technical staff of the

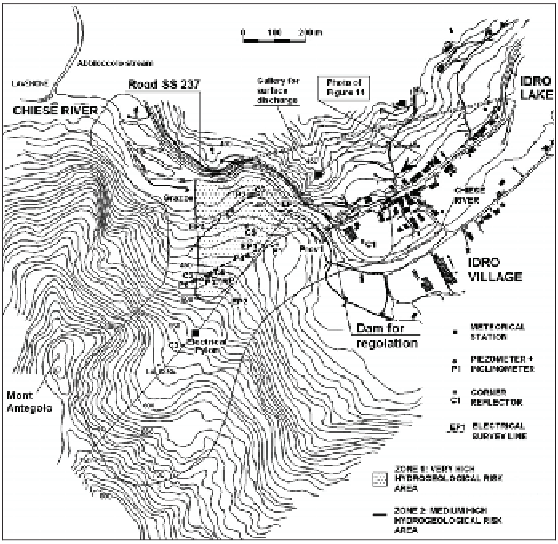

Fig. 15. Map of "Idro" landslide with limits of very high and medium high risk areas and the localization of investigations.

Provincia of Brescia (1995) on the base of the results of an in-site investigation consisting of three boreholes instrumented with two piezometers and two inclinometers. The analysis was performed assuming standard plane strain condition and hypothesizing that the movements might take place along with the direction of the ancient landslide. As far as the stability condition appeared to be approaching the critical state, in 1998 the area has been classified by the Regional Geological Authority as "high risk area" (Figures 14 and 15). Thus, in order to reduce the risk of flooding due to the possible occlusion of the valley caused by possible slope failure, the water level in the lake has been and is still today maintained relatively low.

The negative economical and psychological effect of this situation on the population of the village of Idro (facing both the lake and the landslide) induced the Regional Authority to plan a more comprehensive geophysical and geotechnical investigation for studying the sliding phenomenon and selecting the most appropriate stabilization interventions. Landslide monitoring has been and it is still performed through classical inclinometers, installed up to 90 m, and also by using the satellite radar interpherometry. Piezometers were also installed in the boreholes. Figure 15 reports the topographic map of the area with the localization of investigations carried out so far.

Even if soil cores were collected down to 90 m from ground level, that means at depths below the river bed, no bedrock was reached in any of the investigated verticals. The debris forming the sliding mass is composed by a heterogeneous mixture of gravel and sand in a matrix of silt and clay. During drilling, several highly weathered and crumbly sandstone boulders, belonging to the original rock-fall, were come across.

The soil composition was analysed in detail in borehole S4 (Figure 16a), leading to the observation that the percentages of soil components are relatively constant with depth with a major concentration of fines in some thin layers. The results of inclinometric measurements recorded so far (Figure 16b) indicate that the shear bands occur in correspondence of these thin layers.

Some shear tests performed on the fraction passing to sieve n.40 (particle size less than 0.42 mm) provided an almost constant friction angle of 25° independently from the fine content in the specimens. This is due to the fact that the fine fraction is never so high to create soil

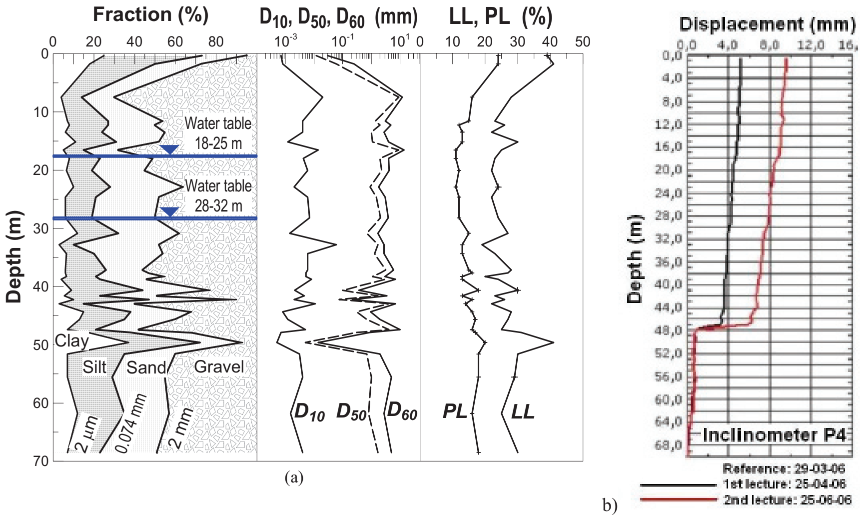

Fig. 16. Profile of soil composition and Atterberg limits (a); displacement recorded by inclinometer P4 (b).

structures commonly referred as to “block in matrix” (Simonini and Cola, 2004).

In the upper part of the landslide, piezometer readings indicate the local presence of two groundwater tables on the same vertical (see piezometers P4-P, P6-P and P7-P in Figure 17): the upper one, about 10-30 m deep, is relatively stable; the deeper (about 33-55 m deep, shows excursions of 3-7 m during rainy periods. This latter requires a very long time to be recharged (about 2-3 weeks after heavy rainfall).

Figure 18 shows the resistivity distribution obtained from the geophysical exploration along the line EP3 (ARPA Lombardia, 2007). A submerged zone characterized by a relatively high resistivity can be noted in the centre of the profile, and it may be justified by the presence of a less weathered rock mass or a core of unsaturated and impermeable soil. At ground level this core corresponds to the presence of a ridge which is lined up with the electrical power line crossing the area and behaves as a watershed dividing the slope in two hydraulic basins.

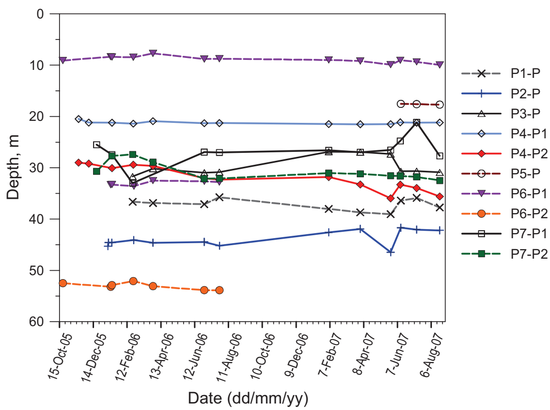

Fig. 17. Groundwater depth in piezometers.

The movements (Figure 19 sketches the displacements recorded in the last year) are characterized by two directions forming around 30° angle with the longitudinal symmetry axis

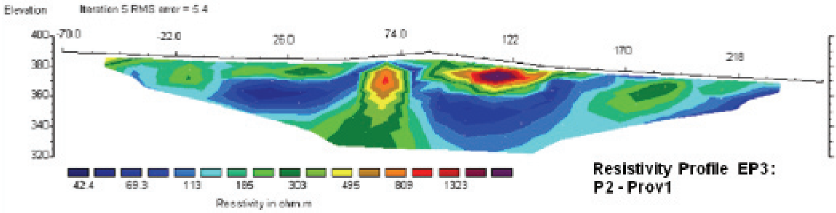

Fig. 18. Resistivity profile along line EP3 (line P2-Prov1).

thus dividing the mass into two turning out bodies. The rate of displacement is not constant over time, the mass generally accelerating in spring, two months after the wet period.

Unfortunately the corner reflectors were installed only at the end of spring 2007, so up to now data from SAR interpherometry are not available.

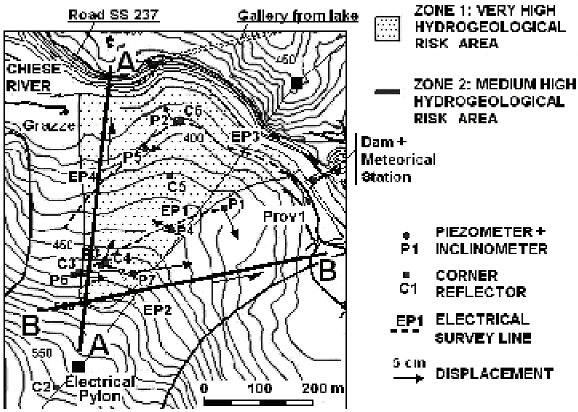

Fig. 19. Detailed map with displacement vectors recorded in 2007.

Back-analysis of slope instability

To back-analyse the movements and individuate the most probable sliding surfaces, new stability analyses have been performed using the Clara software. According to the movements recorded up to now, 3D analyses were carried out examining two possible cinematic movements, respectively along the directions AA and BB of Figure 19. The minimum safety factor for elliptical sliding surfaces was searched with the automatic procedure based on the Janbu LE method.

The slope was considered as composed by a uniform soil with saturated unit weight γ_{sat} = 18 kN/m^3, friction angle ϕ'= 25° and cohesion c'= 1 kPa.
Since the Clara software required the definition of all the 3D surfaces (i.e. water table, ground surface and interface layering, ect.) in a regular grid format, a careful input data preparation was necessary. The assigned 3D water pressure distribution was determined with a 3D function, that interpolates, by Kriging technique, the maximum water depths measured with the piezometers: the resulting surface is shown in Figure 20. The ground surface was obtained digitalizing the contour lines from an aerophoto-grammetrical map in 1:10000 scale, and interpolating the elevation at the nodes of the regular grid used by Clara.

Figure 21a-b shows the results of the LE method used to analyse the stability condition in AA direction. To note that the critical sliding surface is an ellipse with a ratio between the longitudinal and transversal axes of 0.70. Similar results were obtained for section BB.

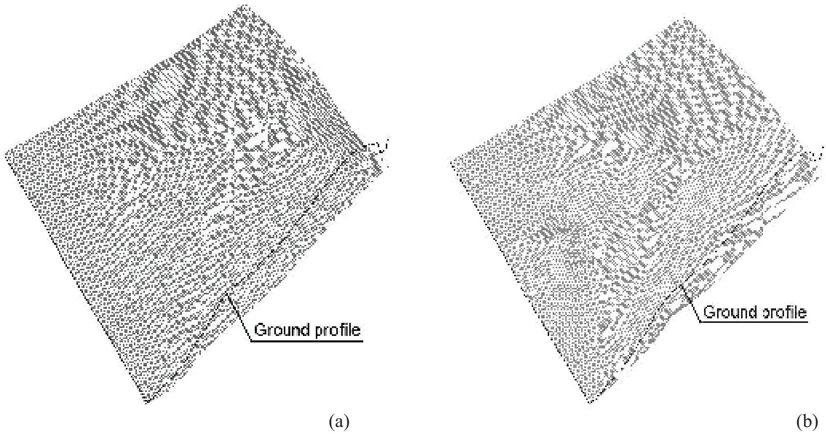

Fig. 20. 3D view of free water surface for analysis with Clara along AA direction: (a) undisturbed surface, (b) with a well installed.

Remedial works

The planned stabilization works consist of several large vertical drainage wells, with depth = 40 m and diameter = 8 m, located in the upper portion of the slope. In order to enlarge the soil volume drained by the large vertical wells, several 50 m long microdrains are drilled sub-horizontally from the shaft of each well into the ground.

The best location of wells was selected by using an optimization procedure relating the increase of safety factor, determined for the two critical sliding surfaces obtained in the previous back-analysis, to the effectiveness of drainage system. To this end, the effect of each deep vertical well with the surrounding sub-horizontal drains is schematized as a larger well, characterized by an influence radius given by the Sichardt equation:

$$R = C_w \sqrt{k} \left(H - h_o \right) \tag{9}$$

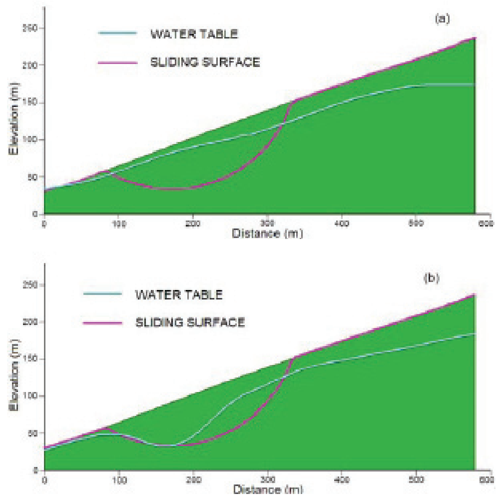

Fig. 21. Critical elliptical surface from 3D analysis performed along AA direction: (a) section, (b) map.

being C_w an experimental constant, k the permeability of soil and H-h_o the reduction in water level imposed with the well. Assuming $k = 10^{-6}$ m/s, the coefficient C_w =3000 (s/m)$^{0.5}$ and H-h_o=15 m, the influence radius turns out equal to 45 m. Figure 20 shows the modified surface corresponding to a vertical well located along the AA section, close to piezometer P4. According to this modified water pressure distribution the safety factor turns out 1.23.

To evaluate the effectiveness of the drainage system on the groundwater pressure and the reliability of calculation, the execution of a first deep well has been planned. The monitoring of soil response in terms of groundwater regime and slope displacements should provide information to design the final whole intervention.

CONCLUSIONS

On the basis of the analysis of two different interesting case histories, the paper discussed the importance of 3D stability analysis for the evaluation of landslide conditions to design the remedial stabilization intervention. The judgment on the relevance of 3D analysis with respect to the case under consideration is of course strongly dependent on the geometry of the landslide, soil and rock layering and on the groundwater regime. Since 3D approach requires much more information with respect to the classical 2D analysis, a careful geological and geotechnical investigation together with accurate surveying and site monitoring are of course necessary.

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