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LANDSLIDE RISK REDUCTION BY COUPLING MONITORING AND NUMERICAL MODELING

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ABSTRACT

The importance of the reference engineering-geology model of a slope is a concept well established in the scientific and technical community facing on large infrastructures. The engineering-geology model is in fact a fundamental informative layer to understand and predict the structure-slope interactions and to design stabilization countermeasures. Such an issue has a relevant role in the case of unstable slopes: at this regard the Vajont case history represents a worldwide reference.

Engineering-geology models can be validated and/or updated by monitoring data. Furthermore, the harmonization of engineering-geology models and monitoring data can be achieved by the implementation of stress-strain numerical models, that represent a validating tool for the engineering-geology models, by collecting the monitoring data and by refining, via calibration analyses, the rheological behaviors, i.e. the stress-strain constitutive laws. In this frame, our experience is referred to an unstable slope involved in a tunnel excavation. A very detailed engineering-geology model was built by means of several in situ and laboratory investigations. The availability of monitoring data with high temporal and spatial resolution referred to slope instability episodes triggered by different external factors (e.g. rainfalls, tunneling, etc) made it possible to better understand the dynamics of the slope-infrastructure system and to refine the numerical model of the slope by using the finite difference code FLAC 7.0. Such a numerical model was implemented by applying a continuum equivalent approach to the involved jointed rock mass, which was considered as a visco-plastic material in order to account for the time dependent behavior. At present promising results have been obtained, especially in terms of assessment of stress-strain variations due to external forces (both environmental and man-induced) and, thus, of forecasting the activation/reactivation of slope instabilities.

Key words: *engineering-geology model, monitoring, numerical modeling*

INTRODUCTION

The importance of designing structures and infrastructures on the basis of a robust engineering-geology model of a slope is a concept well established in the scientific and technical community, especially if facing on large infrastructures (see the "milestones" such as the observational approach proposed by Terzaghi & Peck, 1967 and the papers by Fooke, 1997; Hoek, 1999 and Hutchinson, 2001; Gibson & Chowdhury, 2009). The engineering-geology model is in fact a fundamental informative layer to understand and

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predict the structure-slope interactions and to design stabilization countermeasures. Such an issue acquires greater importance in the case of unstable slopes: in this regard, the dramatic effects caused by the occurrence of the Vajont landslide on 1963 due to the dam construction and its rapid drawdown still represents a strong lesson about the importance of taking into account interactions between slopes and infrastructure both in the design phase and in the executive one.

Nowadays the reconstruction of reference engineering-geology models can take advantage of methods and techniques of investigation (both on site and in laboratory) able to depict a detailed cognitive framework for several purposes within the geo-hazard related fields (e.g.: Bianchi-Fasani *et alii*, 2008; Ganerod *et alii*, 2008). Notwithstanding, based on the geological complexity and/or the width of the area to be investigated as well as on the number of investigations compatible with the dedicated budget, a more or less significant level of uncertainty still persists. In the case of unstable slopes, a fundamental help comes from monitoring activities, which at present benefit from a wide variety of techniques, both traditional and innovative, able to measure at different scales, depths and resolutions the actual displacements and/or the external acting forces (i.e., rainfall, dynamic inputs, etc.). The results – in terms of presence of shear surfaces or zones and of definition of state of activity – can then confirm the preliminary conceptual engineering-geology model or modify it according to the collected data. Once the engineering-geology model is well constrained, it can be transferred into a numerical model, which is the most powerful tool for backward and forward analyses: in a calibration phase, the correspondence between the numerical simulations and the results of monitoring (then the actual displacements/deformations) becomes the best criterion for assessing the goodness of the reference engineering-geology model, which can be eventually modified in a sort of iterative process until the achievement of a final, satisfactory model.

In other words, therefore, the engineering-geology model is a container within which the objective data, i.e.: actual displacements measured by monitoring, and the results of stress-strain modeling must find their reciprocal consistency. Once the engineering-geology model and, then, the numerical simulation of stress-strain relations is appropriately tested in accordance with the criteria above described, the numerical modeling becomes a tool of fundamental importance for the prediction of deformations expected given the changes of stress state induced – for example – in

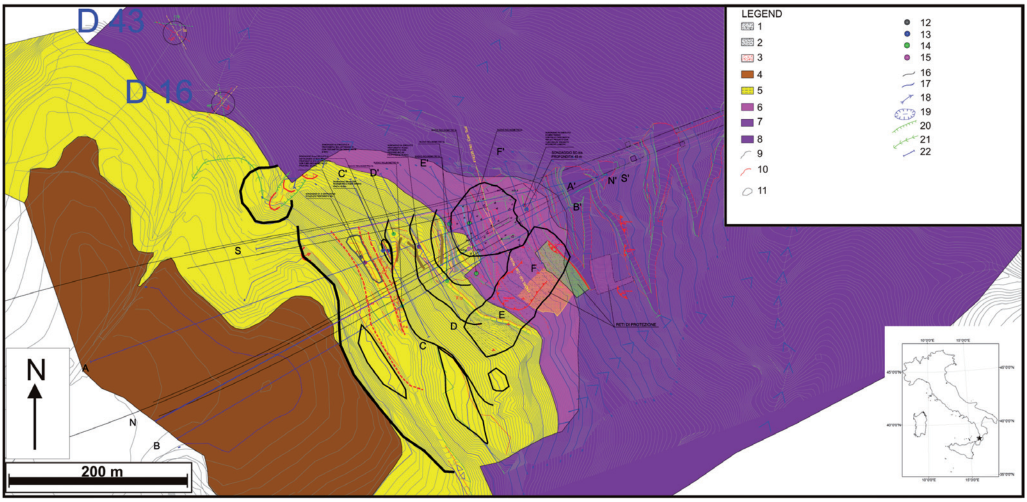


Fig. 1 - Geological map of the landslide area. Key to legend: 1 - March 2007 landslide; 2 - Pre-existing landslide; 3 - Debris; 4 - Marine terrace deposits (Pleistocene); 5 - Yellowish sands (Pleistocene); 6 - Gneiss rock involved in the landslide (RQD < 30); 7 - Gneiss rock involved in the landslide (40 < RQD < 30); 8 - Gneiss rock (RQD > 40); 9 - Landslide scarp; 10 - Landslide scarp activated during monitoring; 11 - Landslide terrace; 12 - Borehole (2005 geognostic campaign); 13 - Borehole (2007 geognostic campaign); 14 - Borehole (2011 geognostic campaign); 15 - Borehole (test site); 16 - Stratigraphic boundary; 17 - Transgressive stratigraphic boundary; 18 - Deepening channel; 19 -Subsidence; 20 - Fluvial erosion scarp; 21 - Gully; 22 - Cross section trace (see Fig. 2)

various stages of execution of work. In this context the monitoring returns to be of paramount importance to check, in the process, the adequacy of the predictions made in terms of deformation/displacement. At the end of this process it could be possible to identify critical thresholds of significant monitored values that can anticipate paroxysmal phases in the slope evolution, thus making the coupling of numerical models and monitoring data a unique forecasting and planning tool that can contribute to undertake efficient risk reduction policies, such as early-warning procedures. The recent management of the landslide in Preonzo - Switzerland (Loew *et alii*, 2012), represents a successful and promising result in this sense.

This paper exemplifies the experience of the CERI - "Sapienza" research team, in the achievement of conceptual modeling based on validation of the engineering-geology model by the integration of monitoring and numerical modeling. The here presented case-study is referred to an unstable slope involved in a tunnel excavation. A very detailed engineering-geology model was built by means of several in situ and laboratory investigations. In addition, the slope has been monitored for five years by means of an integrated monitoring platform made up of "punctual" and areal displacement monitoring devices, integrated with piezometers and rain-gauges. The availability of data referred to slope instability episodes triggered by different external factors (e.g. rainfalls, tunneling, etc.) made it possible to better understand the dynamics of the slope-infrastructure system and to refine the numerical model of the slope.

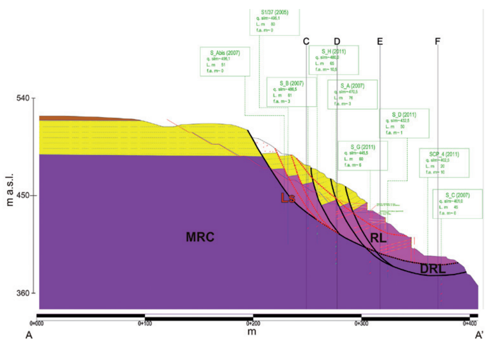


Fig. 2 - Engineering-geology cross-section of the landslide. Key to legend: MRC = Intact rock mass; LR = Rock mass involved in the landslide; DLR = Rock mass involved in the landslide (deep part); Ls = Landslide surface; other symbols in Fig. 1

GEOLOGICAL SETTING OF THE SLOPE

The case study described in this paper concerns a section of a major road imperatively planned on an unstable slope (Fig. 1). In this context during the start-up works for the realization of a tunnel entrance, a shallow translational landslide (with a volume of about 10^4 m^3) occurred, thus completely destroying the already constructed structures. Following this event, the Research Centre for Geological Risks (CERI) of the University of Rome "Sapienza" carried out detailed engineering-geological investigations and surveys (geomorphological, geological and geomechanical field surveys, boreholes, seismic surveys and laboratory tests of samples) on the slope in order to define a reference model to explain the occurrence of the landslide.

The steep slope is made up of jointed and weathered metamorphic rocks overlaid by Pliocene and Pleistocene sandy, marine deposits, while few meters of sandy colluviums cover the slope. Moreover, geological and geomorphological evidence of an old deep roto-translational slide with a total volume of about 1×10^6 m^3, which involved jointed gneiss and the overlying Pliocene and Pleistocene sands, was also recognized (Figs. 1 and 2). The main sliding surface (up to 50 meters deep) of the old landslide and many secondary sliding surfaces were reconstructed by geomorphological surveys and stratigraphic logs (Fig. 2). Minor shallow translational movements of about 1×10^4 m^3 of colluviums and bedrock have been observed in the middle-lower part. Among these, we can include the landslide event which destroyed part of the already-built excavation structures.

The above described geological model was used as a conceptual basis to design the stabilization works. According to this model, the reactivation of both the whole old landslide and part of it are plausible because of slope re-profiling and excavation activities. Therefore, some stabilization work was performed and a continuous monitoring system was used to: i) monitor the evolution of the slope under undisturbed conditions and during construction; ii) predict the occurrence of critical conditions, if any. This system would also optimize planning, design and construction activities and protect the workers during the tunnel construction (Bozzano *et alii,* 2011).

SLOPE MONITORING

Starting from March 2007 excavations have been carried out in order to obtain a more stable profile of the slope. Furthermore, three anchored bulkheads made of 0.1 m diameter and 10-22 m long piles were realised. Bulkheads were coupled with the deeper part of the slope by 35 m long cable inclined of 35° from the horizontal plane. Anchors were cemented in the last 12 m and the three bulkheads were realised from August 2008 to January 2009 moving downslope (Fig. 3 and monitoring data in Fig. 4). The tunnel excavation started in November 2009 and was stopped in February 2010 (Fig. 5). A total amount of 28 m of tunnel were excavated following three main phases before being interrupted due to the high value of displacement recorded on the slope (Fig. 5). The slope was continuously monitored starting from November 2007. The overall monitoring system was planned and realized in order to record both shallow and deep displacement of the natural slope and man-made structures. The monitoring system consists of:

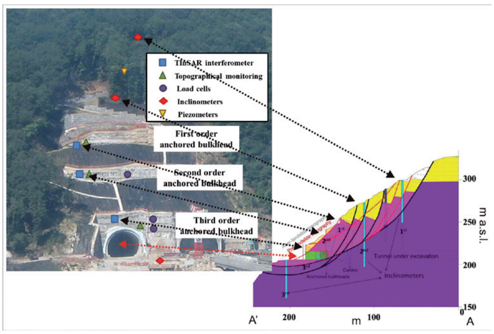


Fig. 3 - Layout of the monitoring system

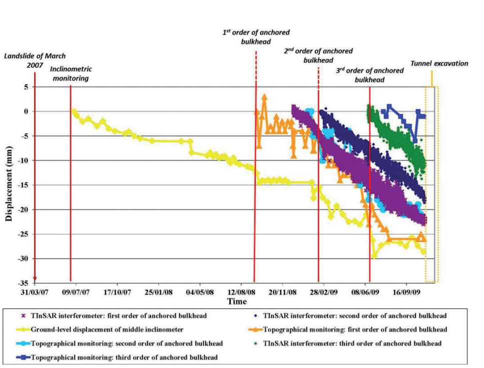


Fig. 4 - Results of monitoring from summer 2007 until autumn 2009. Last part of the time interval is referred to the realization of the three bulkheads

- an integrated remote platform made of a Terrestrial SAR Interferometer (TInSAR) model IBIS-L (by IDS S.p.A.), a weather station and an automatic photo camera (from November 2007) (Bozzano *et alii,* 2008);
- three inclinometers, one full screen piezometer and one 'Casagrande' piezometer (from June 2007);
- topographic monitoring by total station of prisms installed on the bulkheads (from September 2008);
- load cells for monitoring man-made structures after their construction (from September 2008);
- convergence monitoring of the tunnel (from November 2009).

The integrated remote platform was hosted in a specially designed box and installed on the slope opposite to the landslide one, at a distance ranging from 700 to 900 m (Bozzano *et alii,* 2008). This platform has been continuously active from November 2007 with a sampling rate of about 5 minutes. Up to now, more than 16,000 photos, 250,000 measurements of weather data and 250,000 SAR maps have been collected. Inclinometric monitoring of the slope is manually performed since June 2007 with a sampling rate ranging from 15 days to few days depending on the working phases. The 'Casagrande' piezometers installed at the depth of 48 m below the ground level were monitored with the same frequency of the inclinometers but they never recorded presence of water. Topographic and load cells monitoring of man-made structures (anchored bulkheads) is active since September 2008 with a sampling rate ranging from 15

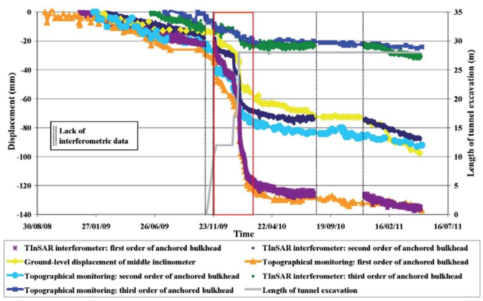


Fig. 5 - Displacement time series (obtained by interferometric and topographic monitoring) of the three anchored bulkheads (November 2008-June 2011) and of the middle inclinometer of figure 3 (September 2009-June 2011). The "y" right axes represents the advancement of tunnel excavation. The slope crisis during the tunnel excavation in the period November 2009 - February 2010 is encompassed by the rectangle with a red outline

days to few days depending on the working phases. The convergence monitoring of tunnel had a sampling frequency ranging from a day to a week, with a step of 7 m for depth of excavation.

Before the beginning of the tunnel excavation the anchored bulkheads showed an almost constant velocity of displacement on the order of 0.05 mm/h. Immediately after the beginning of the excavation the velocity of bulkheads suddenly increased reaching maximum values of 0.75 mm/h, with acceleration and deceleration peaks on the order of 0.02 mm/h^2 (Fig. 4 and Fig. 5). During the three excavation phases discussed above a maximum displacement of about 100 mm was recorded on the first order of bulkhead. In the last two phases, the interferometric monitoring allowed us to clearly recognize a typical creep behaviour.

NUMERICAL MODELING

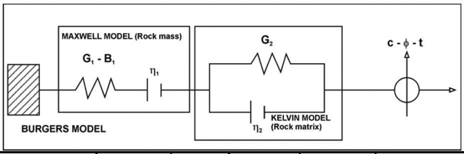
ENGINEERING-GEOLOGY MODEL

The engineering-geology model of the landslide was developed from on-site geomechanical surveys, stratigraphical logs and CID triaxial tests. The geomechanical characterization of the old deep roto-translational slide was achieved by data coming from the stratigraphic logs (Fig.1). The CID triaxial tests were performed on some samples of gneiss at the "Laboratorio della Provincia Autonoma di Trento" geotechnical laboratory to evaluate both strength and deformational parameters of the intact rock (Montagna, 2012). The Young's modulus values measured for the intact rock from triaxial laboratory tests vary in the range of 50–60 GPa and the UCS resulted about 94 MPa (Bozzano *et alii*, 2012). So, by the use of an equivalent continuum approach (Ramamurthy, 1993; Sitharam *et alii*, 2001), the equivalent Young's modulus of the jointed rock has been derived resulting 35.58-36.19 GPa and 34.04-35.04 GPa for the bedrock and for the landslide rock mass, respectively. According to the ISRM standard procedures (ISRM, 2007), the indexes Ib (block size index) and Jv (number of joint per cubic metre) were measured by geomechanical scanlines on the outcropping rock masses. An average Jv value of about 19 joints for cubic meter and a Ib of about 11 cm resulted for the bedrock (MRC in Fig. 2), corresponding to the rock mass class H of Table 1, while a Jv of about 21 joints for cubic meters and Ib ranging from 8.1 to 9.6 cm resulted for the landslide mass (RL and DRL in Fig. 2), corresponding to the rock mass classes L and M of Table 1. Moreover a Q rock mass class was attributed to the landslide surface (Ls) which is characterised by an average Jv of 30 joints for cubic meters and an average Ib of 6.3 cm. By using the two ISRM geomechanical indexes Jv and Ib, a Rock Index (RI) was determined, which is based on a statistical algorithm (Montagna, 2012). This statistical analysis was developed by the Authors thanks to more than 144 geomechanical scanlines which were performed on the rock masses outcropping within an area of 100 km^2 which includes the here considered slope. More in particular, the RI index is based on the transformation of the data series of Jv and Ib in series of data with mean equal to zero and variance equal to one by the relation (Montagna, 2012):

$$x_{SCORE} = (x_i - u)/\sigma$$

where x_{score} is the new series of data, x_i is the series of data (related to Jv or Ib), u and σ are the mean and the standard deviation of the data x_i, respectively. A cluster analysis was then applied in order to define the rock mass classes by using the RI index.

The geomechanical parameter values for the nu-



Lithotechnical Unit	Rock mass class	density	# joints per cubic metre	block size index	friction	cohesion	tension	shear modulus	bulk modulus	Maxwell viscosity	Kelvin viscosity
		(kg/m^3)	Jv	Ib	φ (°)	c (MPa)	σ_t (Pa)	Gj (σ_3=0) (GPa)	Kj (σ_3=0) (GPa)	η_1 (Pa*s)	η_2 (Pa*s)
Ls	Q	2690	30.4	6.3	22	1.0	-4.04E+03	11.74	19.57	8.17E+15	4.47E+12
RL	M	2690	22.2	9.6	32	2.3	-9.19E+03	13.62	22.70	7.51E+19	4.10E+16
DRL	L	2690	20.7	8.1	33	2.5	-9.82E+03	14.02	23.36	3.97E+20	2.17E+17
MRC	H	2690	18.7	11.1	36	3.1	-1.45E+04	14.48	24.13	4.54E+21	2.00E+18
	Intact Rock	2690	0.00		52	10.3		14.99	24.98		

Tab. 1 - Rheological parameters used for the stress-strain numerical modeling, calibrated by the monitoring results. The upper box exemplifies the adopted visco-plastic model

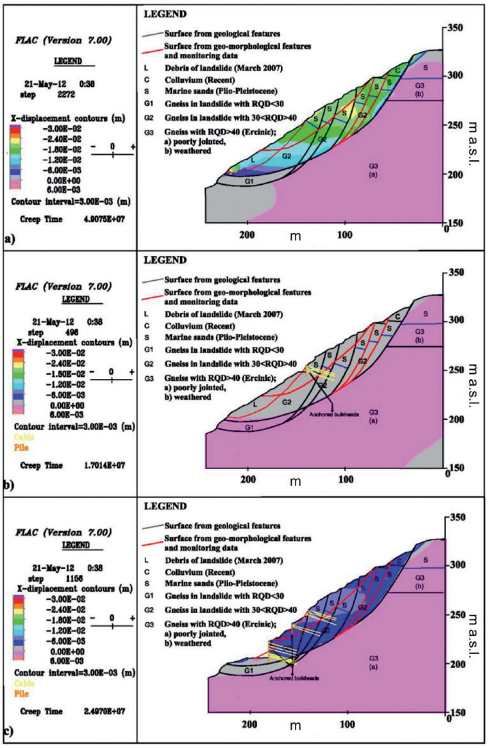


Fig. 6 - Sequential numerical modeling performed for calibrating the rock mass rheological behavior

merical simulation are summarized in Table 1 and they were derived for each rock mass class after a stress-strain calibration process via numerical modeling, by assuming a creep rheology according to the visco-plastic Burgers model.

NUMERICAL MODELING

A 2D finite difference model was implemented along three different sections obtained across the slope, by the finite difference code FLAC 7.0 (ITASCA, 2011). Geological features as well as structural elements (such as anchored bulkheads, retaining walls) were also modelled in order to follow the stress-strain changes induced within the slope by the on-going human activities. The rheological behaviour of the involved rock mass was reproduced according to an equivalent continuum approach (SITHARAM *et alii*, 2001), i.e. taking into account by equivalent parameter values the intact rock mass properties as well as the jointing conditions (Hoek and Brown failure criterion combined to Mohr-Columb failure criterion).

Use was made of two different creep models to

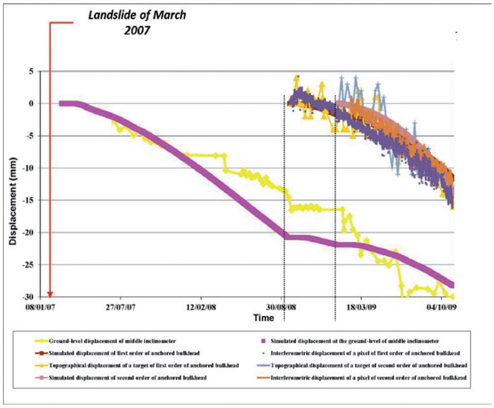


Fig. 7 - Simulated vs. measured displacements at the inclinometer (violet and yellow lines) and at the anchored bulkheads (brown lines)

simulate the rheology of the MRS and of the DRL, RL and Ls lithotechnical units (Fig. 2). Respectively:

1) a Burgers visco-plastic model was assumed for the MRS;
2) a Burgers visco-plastic model coupled with a plasticity threshold was assumed for the RL, DRL and Ls.

Tab. 1 summarises the parameter values attributed to the adopted rheological models. According to BOZZANO *et alii* (2012), the viscosity values for both the visco-elastic and the visco-plastic elements of the Burger model were selected in the range 1019-1023 Pa•s and calibrated to match morphological evidences and the displacements recorded as a consequence of the landslide.

For calibrating the viscosity values of the Burger model referred to the MRC, the viscosity values of the Kelvin-Voight visco-elastic element were always assumed to be one order of magnitude higher than the ones used for the visco-plastic Maxwell element. Based on the performed numerical calibration, the here considered viscosity values result the lowest for justifying the landslide activation in the present slope shape with respect to the river valley evolution. On the other hand, for calibrating the viscosity values for RL, DRL and Ls a best fit was performed between the monitored displacements, referred to the different excavation and re-shaping steps within the landslide mass and the numerical modeled ones.

Fig. 6 shows the outputs in terms of displacements resulting from the sequential numerical modeling performed for calibrating the rock mass rheological behavior of the landslide mass. A slope re-shape was realized starting from March 2007 and three orders of anchored bulkheads were installed to stabilize the slope after the cut of corresponding parts of the slope. Horizontal displacements up to some centimeters involved the landslide mass after the occurrence of the March 2007 landslide (Fig. 6a); further centimeter-scale displacements added to the previously occurred ones as a consequence of the re-shaping and of the cut of the trenches (Fig. 6b and 6c).

The graphs reported in Fig. 7 illustrate the final modeling results, i.e. the horizontal displacements resulting by the numerical model after the rheological calibration (i.e. according to the rock mass parameters summarized in Tab. 1); the numerically computed displacements are compared with the ones measured along the inclinometer borehole and at the anchored bulkheads during the re-shape and consolidation works. The comparison demonstrates the very good fit among the modeled and the measured displacement values, thus providing a stress-strain scenario which strongly reproduces the actual behavior of the slope, observed during the consolidation works.

CONCLUSIONS

This study reports the experience of the CERI - “Sapienza” research team on the interaction of monitoring and numerical modelling in the achievement of a conceptual model based on a engineering-geology one with reference to a specific case-study. In this frame, the coupling of numerical models and monitoring data could represent an effective forecasting and planning tool that can contribute to undertake efficient risk reduction policies.

In the here proposed case-study, a stress-strain numerical modelling was performed by reproducing the monitored displacements due to the re-shaping of the considered slope and the realization of reinforcements (bulkheads). During the here considered time interval the slope experienced a total amount of displacement of more than 100 mm. By this back-simulation the best rheological model describing the rock mass mechanical behavior was determined and its parameters were calibrated. The stress-strain effects related to the slope re-shaping and reinforcement were referred to the engineering geology model and it can now provide scenarios of possible effects due to destabilizing actions external to the slope as well as to the management of the road tunnel.

Finally, we tuned up a numerical tool able to manage strain effects on slopes constituted by jointed rock by applying the continuum equivalent approach to the time dependent behavior. Up to now, such an approach has been experienced and validated by other authors (Ramamurthy, 1993; Sitharam *et alii*, 2001; Sitharam & Madhavi Latha, 2002; Sridevi & Sitharam, 2000) for an elasto-plastic behaviour as an useful application for tunneling; in this paper we demonstrated the reliability of such an approach also for the visco-plastic behavior applied to sliding slopes emphasized by coupling it with time and space high resolution monitoring data.

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REFERENCES

Bianchi-Fasani G., Cavinato G.P., Petitta M., Scarascia-Mugnozza G. & Voltaggio M. (2008) - *The geological model of Celano town area for seismic microzonation activities.* Soil Dynamics and Earthquake Engineering, **28**: 978-985.

Bozzano F., Mazzanti P. & Prestininzi A. (2008) - *A radar platform for continuous monitoring of a landslide interacting with an under-construction infrastructure.* Italian Journal of Engineering Geology and Environment, **2**: 35-50.

Bozzano F., Cipriani I., Mazzanti P. & A. Prestininzi (2011) - *Displacement patterns of a landslide affected by human activities: insights from ground-based InSAR monitoring.* Nat Hazards, **59** (3): 1377-1396, DOI 10.1007/s11069-011-9840-6.

Bozzano F., Martino S., Montagna A. & Prestininzi A. (2012) - *Back analysis of a rock landslide to infer rheological parameters.* Eng. Geol., **131-132**: 45–56, doi:10.1016/j.enggeo.2012.02.003.

Bozzano F., Cipriani I., Martino S., Mazzanti P. & Prestininzi A. (2013, in press) - *Forecasting methods for landslides interacting with infrastructures.* In: Proc. of 2nd World Landslide Forum, Roma, 3-9 September 2011.

Fookes (1997) - *Geology for engineers: the geological model. Prediction and performance.* Quart. Journ. of Eng. Geol., **30**: 293-424.

Ganerød G.V., Grøneng G., Rønning J.S., Dalsegg E., Elvebakk H., Tønnesen J.F., Kveldsvik V., Eiken T., Blikra L.H. & Braathen A. (2008) - *Geological model of the Åknes rockslide, western Norway.* Engineering Geology, **102**: 1-18.

GIBSON A.D. & CHOWDHURY R. (2009) - *Planning and geohazards*. Geological Society, London, Engineering Geology Special Publications, **22**: 113-123. doi: 10.1144/EGSP22.8

HOEK E. (1999) - *Putting numbers in geology: an engineer's view point.* Quart. Journ. of Eng. Geol., **32**: 1-19.

HOEK E., CARRANZA-TORRES C. & CORKUM B. (2002) - *Hoek-Brown failure criterion.* 2002 Edition. Proc. NARMS-TAC Conference, Toronto, **1**: 267-273.

HUTCHINSON J.N. (2001) - *Reading the ground: morphology and geology in site appraisal*. Quart. Journ. of Eng. Geol., **34**: 7-50.

ISRM (2007) - *The complete ISRM suggested methods for rock characterization, testing and monitoring: 1974-2006.* In: ULUSAY R. & HUDSON J.A. (EDS.). *Suggested Methods Prepared by the Commission on Testing Methods*. International Society for Rock Mechanics, Compilation Arranged by the ISRM Turkish National Group, Ankara, Turkey, 628 pp.

ITASCA (2011) - *FLAC 7.0: user manual.* Licence number 213-039-0127-18973 (Sapienza University of Rome, Earth Science Department).

LOEW S., GISCHIG V., MOORE J.R., KELLER-SIGNER A. (2012) - *Monitoring of potentally catastrophic rockslides.* In: CLAGUE J.J.& STEAD D. (EDS.) - Proc. of 11th Int. and 2nd North Am. Symp. on Landslides and Engineered Slopes, Banff, Canada, 3-8 June 2012, 1: 101-116. Cambridge University Press, Cambridge, U.K.

MONTAGNA A. (2012) - *Approcci equivalenti continui nella costruzione di modelli geologico-tecnici di versanti naturali in roccia.* Unpublished Ph.D thesis – University of Rome "Sapienza", http://padis.uniroma1.it/handle/10805/1355.

RAMAMURTHY T. (1993) - *Strength and modulus responses of anisotropic rocks.* Comprehensive Rock Engineering, **1** (13): 313-329.

SITHARAM T.G. & MADHAVI LATHA G. (2002) - S*imulation of excavation in jointed rock masses using a practical equivalent continuum approach*. International Journal of Rock Mechanics and Mining Sciences, **39**: 517-525.

SITHARAM T.G., SRIDEVI J. & SHIMIZU N. (2001) - *Practical equivalent continuum characterization of jointed rock masses*. Int. J. Rock Mech. Min. Sci., **38**: 437-448.

SRIDEVI J. & SITHARAM T.G. (2000) - *Analysis of strength and moduli of jointed rocks*. Geotechnical and Geological Engineering, **18**: 3-21.

TERZAGHI K. & PECK R.B. (1967) - *Soil mechanics in engineering practise*. 2nd ed., Wiley, New York.