The Cimaganda landslide (SO): hydro-mechanical numerical modelling

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ABSTRACT

The San Giacomo Valley (Sondrio, Italy) as many alpine areas, is quite frequently affected by rock slope landslides at different scales. This work deals with the study of the Cimaganda rockslide which occurred in September 2012 after some days of ordinary rainfall. It involved a rock volume of 20.000 m^3, blocking the main road (SS36) and isolating the municipality of Madesimo and Campodolcino in the upper Valley for few days. The landslide developed at the flank of the historical Cimaganda rockslide, which mobilized some Mm^3 of rock. Following a procedural scheme including field and laboratory analysis followed by stress-strain numerical modelling, this work develops a solid conceptual geomechanical model of the area and the back analysis of the 2012 event, exploring instability-forecasting scenarios.

KEY WORDS: hydro-mechanical modelling, rockslide, rock mechanics.

INTRODUCTION

The study area is located along the San Giacomo Valley, in the Central Italian Alps, between the village of Chiavenna (SO) and the Splügen Pass (Valchiavenna area). The regional geological setting is related to the alpine Pennidic Nappe arrangement, which is characterized by the emplacement of sub-horizontal gneissic bodies, separated by meta-sedimentary cover units (Montrasio & Sciesa, 1988).

In the San Giacomo Valley main structural alignments show the following directions: WNW–ESE, NW–SE, NE–SW and N–S. The first system seems to be related to the regional orientation of the Insubric Line, while the second one has the features of the Forcola Line. The NE–SW system is related to the Engadine Line, although the N–S is represented by a bundle of persistent fractures parallel to the valley and not directly connected to any tectonic line (Ferrari et al. 2014).

The valley, furrowed by the Liro Torrent, is N–S directed and characterized by high sub-vertical rock cliffs resulting from the complex structural and glacial evolution. Deep-seated gravitational slope deformations (DSGSD) affect both valley flanks and superficial slopes instability processes are frequent, principally as rockfalls.

The Cimaganda landslide developed on 27th September 2012 from the east slope of the valley and involved a rock volume of 20.000 m^3, blocking the main road (SS36) and isolating the

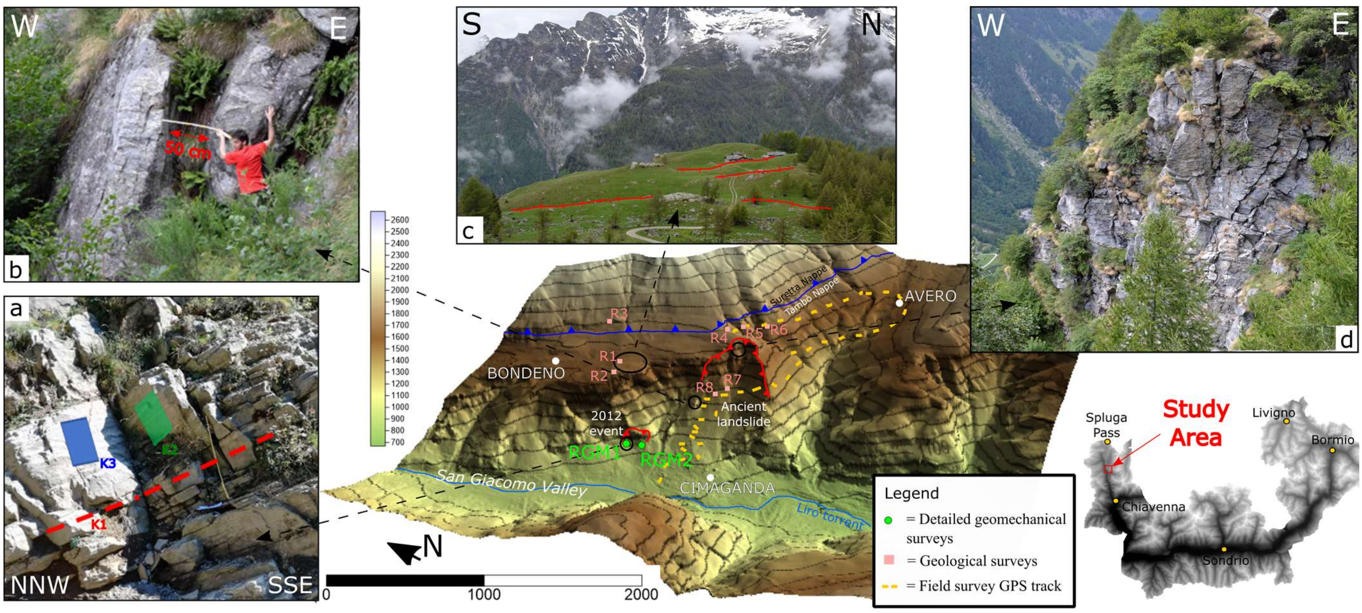

Fig. 1 – Geomorphological features of the studied area and location of geomechanical and geological surveys; a) Three main sets of discontinuities identified with field surveys; b) Maximum joint aperture measured along the slope (K2 set); c) Trenches and counter-slopes at the crown of the ancient landslide; d) Rock masses outcropping at the crown of the ancient landslide with an high degree of fracturing.

municipality of Madesimo and Campodolcino in the upper Valley for few days. It is located at the right flank of an ancient and massive rock avalanche that mobilized about 7.5 Mm^3 of material reaching the bottom of the valley and occluding the Liro Torrent (Mazzoccola, 1996).

The adopted procedural scheme included field and laboratory analysis, the definition of a hydro-geomechanical conceptual model and the development of a stress-strain analysis by 2D FEM. The main purpose of this work is the assessment of preparatory and triggering factors which led to the 2012 event by reproducing in back analysis the slope collapse and to explore instability-forecasting scenarios.

FIELD DATA COLLECTION

Geological, geomorphological and geomechanical surveys performed at the rock wall scar outcrops of the 2012 event and at the ancient landslide crown (Fig. 1), allowed to recognize typical features of deep gravitational deformations and large-scale stress release: trenches and counter-slopes at the crown of the ancient landslide (Fig. 1c) and sub-vertical tensile fracturing along the slope (Fig. 1b, d).

The rock masses are mainly composed of the gneissic bodies of the Tambò nappe Unit. A spatial variation in textural and compositional features was observed, with a less foliated and stronger material along the slope (orthogneiss) than at its top (paragneiss).

Detailed geomechanical field surveys were performed according to ISRM suggested methods (ISRM, 1981). The calculated RMR (Rock Mass Rating; Bieniawski, 1989) and the evaluated GSI (Geological Strength Index; Hoek & Marinos 2000) reveal a generally discrete to good rock mass mechanical quality, with an average GSI value of 60 ± 5. At the top of the slope, which reaches the crown of the ancient landslide, lower GSI values (50 ± 5) were highlighted, due to a more intense fracturing and weathering degree of the rock masses (Fig. 1d).

The structural surveys carried out in the area, allowed to distinguish three main discontinuity sets (Fig 1a), coherent with the regional lineaments orientation: K1 gently dips towards the E-NE, following the regional foliation (which is parallel to the nappe tectonic contact); K2 dips towards NW with a mean dip angle of 70°; K3 dips vertically towards W, parallel to the San Giacomo Valley.

LABORATORY TESTS

In order to investigate the mechanical behavior of rock materials and discontinuities, some geomechanical laboratory tests were performed according to the ASTM standards. Point Load Tests, Uniaxial Compressive Tests and Direct Shear Tests on rock joints were conducted obtaining respectively Uniaxial Compressive Strength of rock material (σ_c), Cohesion (c) and Friction Angle (ϕ) of joints.

Tests were preceded by a careful analysis of materials and joints surface morphology: joint roughness, joint compressive strength analysis and tilt tests were evaluated for each examined surface. Dominant discontinuities shear strength values obtained from direct shear tests showed a strong connection with the surfaces morphological features, with values of friction angle varying from 27° to 48°. The lowest values refer to smooth and altered surfaces sampled at the crown of the ancient landslide (JRC = 3), whereas the highest ones to the more rough and fresh joints coming from the 2012 landslide scarp (JRC = 9).

In order to study both peak and residual behavior of joints shear strength, multiple series of shear loading cycles were applied to each specimen. The maximum shear strength provided by discontinuities sampled at the crown of the ancient landslide is comparable to the residual shear strength values obtained at the fifth shear cycle for joints of the 2012 landslide scarp (Fig. 2). This is in accordance to their different lithological features and to the degree of the weathering and degradation.

Also the intact rock strength behavior is affected by the different degree of weathering and textural features with UCS values ranging from 60 to 175 MPa.

From the results obtained by field surveys and laboratory tests, the application of the Generalized Hoek–Brown failure criterion allowed to estimate the rock mass strength and elasto-plastic properties (Hoek et al., 2002). Moreover, direct shear tests provided the C and ϕ parameters necessary to apply the Mohr-Coulomb law describing joints behavior as well as normal and shear stiffness coefficients.

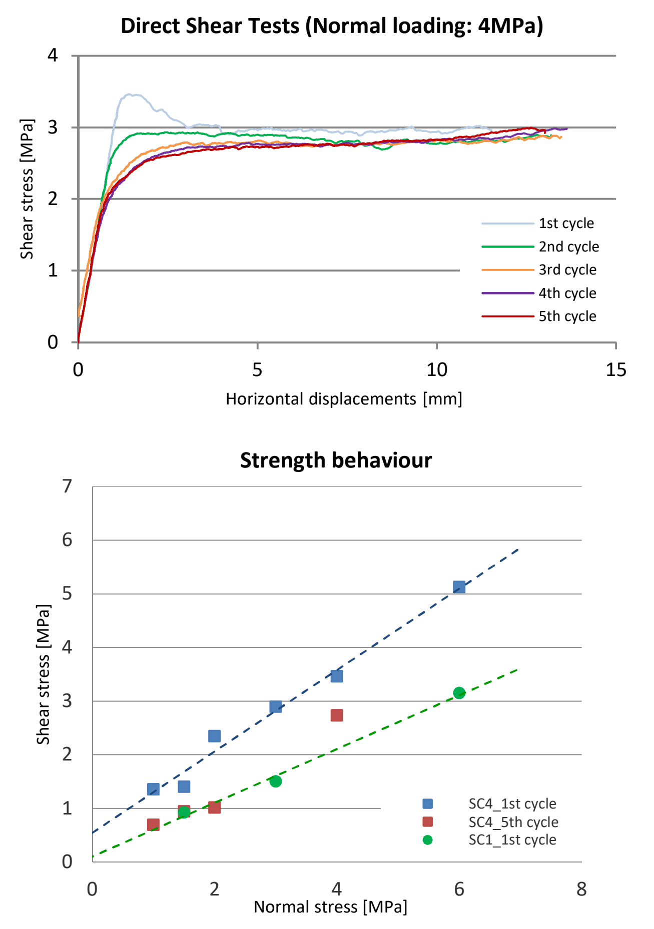

Fig. 2 – a) Stress-strain results of five shear test cycles performed on one of the specimens of the 2012 landslide scarp; b) Peak and residual strength behavior obtained on the K2 discontinuities sampled at the 2012 landslide scarp (SC4) and peak strength behavior obtained on the K3 discontinuities of the crown of the ancient landslide (SC1).

NUMERICAL MODEL

The slope evolution was simulated using the two-dimensional numerical code RS2 (RocScience, 2017), a geomechanical simulation code based on the finite elements method (FEM). The model extends from the Liro Torrent up to the top of the slope at a height of about 1800 meters a.s.l., corresponding with the structural terrace that constitute the crown of the ancient landslide.

Geomechanical conceptual model

The rock mass is assumed to behave as an elasto-plastic medium, according to the Generalized Hoek & Brown strength and deformability criterion. Schistosity was accounted like planes of weakness introducing a jointed material with planes orientation and Mohr-Coulomb strength parameters.

A multiple joint network, made by the two main sets K2 and K3 identified form geomechanical survey, was also introduced. By using Beacher joint network model, which allows a statistical distribution of joints orientation, frequency and intensity, and by assigning Mohr-Coulomb strength parameters, each set of discontinuity was fully represented. Strength parameters were derived both from laboratory analysis and from Barton-Bandis model (Barton & Bandis, 1990), developed using JRC (Joint Roughness Coefficient) and JCS (Joint Compressive Strength) data evaluated during geomechanical surveys and corrected in relation with the scale effect.

The decrement of geomechanical quality with depth is taken into account by modelling three layers at which different elastic and plastic properties were assigned; moreover, areal fracture intensity was reduced from the direct measurements tacking into account a scale effect.

Rock masses principal hydraulic conductivities were calculated by means of the permeability tensor eigenvectors and eigenvalues considering joint orientation, JRC, aperture and fracture frequency of each discontinuity set (Lotti et al., 2012). A value of k_{max}=1.61x10^{-3} m/s and k_{min}=2.83x10^{-7} m/s was calculated from the collected data. Considering a level of 10 MPa, representative of the principal stress conditions at the depth of the developed slip surface, a value of k_{max}=1.26x10^{-5} m/s and k_{min}=2.21x10^{-9} m/s was then obtained.

The definition of the landslide triggering factors requested the analysis of the rainfall regimes. The 2012 rockslide was preceded by a rainfall event with a cumulated precipitation of 267 mm in four days ("San Giacomo Filippo" ARPA station). Studying the rainfall data, an infiltration function over time was evaluated assuming that the total amount of available water infiltrates in the slope.

Boundary conditions – meshing – phases of modeling

Boundary conditions at the base and on both sides of the model, further enough to avoid boundary effects in the area of interest, implied no displacements along both X and Y axes. The mesh used is uniform with 6 nodes triangles elements (4301 elements and 8299 nodes).

Starting with a fully dry model, these successive steps were performed:

(1) The system was led to elastic equilibrium using gravity load. At the end of this step resulting displacements were completely reset (Model 0).

(2) Elasto-plastic mechanical strength parameters were attributed to the rock mass, and to the joint network, considering the best conditions, i.e. the highest values detected in the studying area (Model 1).

(3) The effect of groundwater presence was evaluated considering a static piezometric level supposed as the local ordinary hydrogeological regime (Fig. 3b). Due to the absence of direct measurements of groundwater levels in the slope, some hydrogeological assumptions were necessary (Model 2).

(4) The application of a second perturbation to the system, representing the mechanical degradation of the rock masses, was established reducing the GSI index (and consequently the Hoek & Brown strength parameters) from 60 (average value calculated for the SC4 site) to 55 (average value calculated for the crowning outcrops) (Model 3).

(5) The mechanical properties of the rock matrix and of the joints sets have been downgraded (maintaining a GSI of 55), introducing the parameters detected at the crown of the ancient landslide (Model 4).

(6) The effects of the rainfall event, which preceded the landslide, was simulated using a semi-coupled hydro-mechanical analysis (variations in pore pressure affect effective stresses and thus deformation distributions, but deformations do not affect pore pressure). Rainfall and hydrogeological conditions were reproduced by a transient analysis, specifying a vertical infiltration along the slope and a constant head at its toe to consider the presence of the Liro Torrent (Model 5).

Following the concept of a progressive failure mechanism, steps 1 to 5 stands for the preparatory mechanisms which lead the system to a certain mechanical degradation (Fig. 3a). This causes a general strength reduction and an increase in induced deformations. In the last step, the introduction of rainfall infiltration represents the collapse triggering factor.

RESULTS

At the first step (Model 1), introducing the best mechanical properties, it was possible to simulate the general conditions of the slope, where the predisposition to instability is highlighted by the presence of yielded elements located at the top of the slope and in correspondence with the pre-failure cliff. The distribution and entity of the simulated displacements are suitable with the direct measurements carried out during geological surveys: the vertical component of displacements (in the order of 10^{-2} m) clearly identifies areas in subsidence at the top portion of the slope; meanwhile the horizontal one (in the order of 10^{-1} m) concentrates at its central portion.

The introduction of lower geomechanical properties and the groundwater base flow induce a significant increase in the yielded elements and a slightly increase of the displacements modulus. Focusing on the central part of the slope where the landslide occurred, a shear strain surface begins to develop, with a tension crack opening at the top (Fig. 3d).

Without a hydrogeological triggering factor, a critical state (i.e. collapse) is not yet reached (step 1 to 5), although an instability predisposition is evident. Finally, the introduction of rainfall infiltration, accompanied by a restricted and temporary rising of piezometric level, develops a localized excess of pore

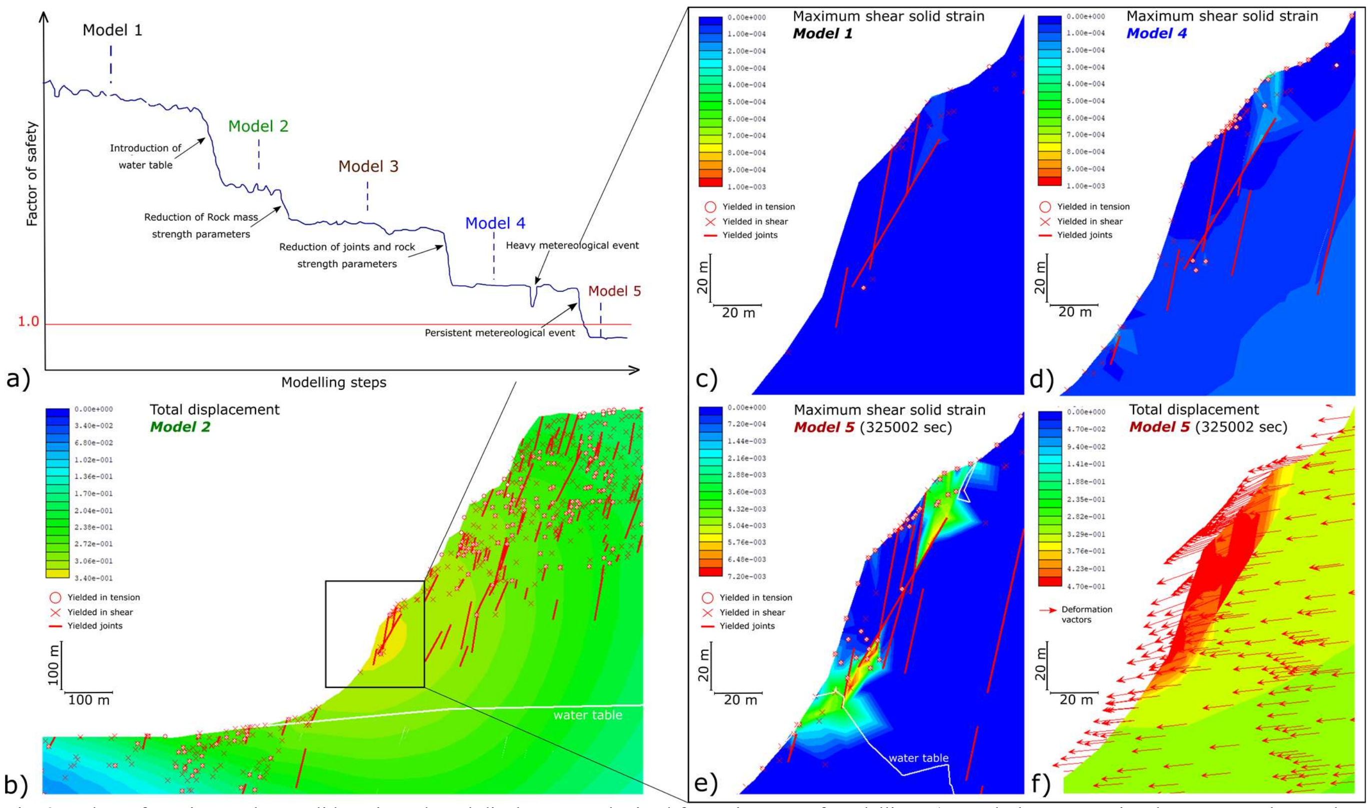

Fig. 3 – Plots of maximum shear solid strain and total displacement obtained for main steps of modelling. a) Graph that summarize the conceptual procedure used during modelling; b) Plot of total displacement and yielded elements obtained introducing a static piezometric level which represent the ordinary condition; c-e) Plots of maximum shear solid strain at three different steps; f) Total displacement obtained at the last step of modelling where a critical state was reached.

pressure sufficient to lead the slope to a critical state (Fig 3e). The maximum shear solid strain and total displacement distribution (in the order of $5x10^{-1}$ m) clearly show the presence of a critical composite shear sliding surface roughly coincident with the observed one (Fig. 3f).

CONCLUSIONS

An accurate geomechanical characterization of the 2012 landslide slope, which represents the evolution of the historical Cimaganda rock-avalanche, was carried out. This led to implement a numerical model through which was possible to simulate the general evolution of the slope and, considering a semi-coupled hydro-mechanical analysis, to reproduce the recent instability event. The simulated instability is associated to the worst detected geomechanical rock mass properties and to not ordinary hydrogeological conditions. After the 2012 landslide, the slope appears to have reached a general state of equilibrium and new landslide events should develop only as a consequence of progressive degradation or very intense rainfalls. This does not exclude single rock mass falls which are favored by the joint orientation and persistency.

Nevertheless, the FEM analysis and the use of a semi-coupled hydro-mechanical modeling, easier than a distinct element model and fully-coupled approach, does not reproduce the natural groundwater flow along the fracture network and consequently imply an overestimation of the landslide triggering factors. The use of a distinct element model approach would be thus explored in order to overcome the hydrogeological simplifications of FEM approach.

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