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Analysis of landslide reactivation mechanisms in Daunia clay slopes by means of limit equilibrium and FEM methods

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ABSTRACT
The paper presents the analysis of the mechanism of reactivation of a deep landslide process which involves the western slope of Volturino in the Daunia Apennines (Southern Italy), where tectonized and fissured soils of poor mechanical properties outcrop. The reactivation, which is monitored by piezometers and inclinometers, takes place when the water table is approximately at the ground surface, i.e. during winter. Limit equilibrium back-analyses of the current landslide process, with a pore pressure distribution consistent with the field data, were performed to assess the in situ mobilised strengths and the depth of the sliding body. Drained finite element analyses were then carried out to simulate the reactivation mechanism by modelling the presence of a band of softened material within the slope along with the seasonal variation in seepage conditions. The results of the different analyses tend to confirm the higher instability of deep sliding bodies in the slope.

INTRODUCTION
The present paper discusses the mechanism of a landslide reactivation taking place within the western slope of the town of Volturino (FG, Southern Italy). This is a representative case of the widespread mudslide activity in the Daunia Apennines, (Southern Italy; Cotecchia et al. 2010). The landslide process was studied by means of in situ surveys, analysis of aerial photos, field investigations and monitoring, limit equilibrium and numerical analyses. Such study was aimed at recognizing the depth of the sliding surface, the mobilised strengths, the evolution of the landslide process and the predisposing factors of reactivation. Limit equilibrium calculations and finite element modelling were used to check the validity of a phenomenological interpretation of the reactivation mechanism and to investigate the influence of the different landslide factors on the evolution of the slope instability process.

Volturino is representative of many other urban areas in the Daunia Apennines, whose slopes are affected by severe landsliding. The Daunia Apennines are located in the eastern front of the Southern Apennines. As discussed by Cotecchia et al. (2010), in this region landsliding takes place extensively within clayey units that are often part of Flysch formations, within which limestones and/or sandstones are interbedded with clayey marls and clays. Generally, the soils are severely disturbed as a consequence of tectonic or past gravitational processes. Cotecchia et al. (2010) report that in the region the reactivations of old landslide bodies are generally found to take place according to few landslide mechanisms. In addition, these reactivation processes are generally from slow to very slow according to the classification of activity proposed by Cruden & Varnes (1996), i.e. they are
characterized by an average displacement rate between 20 and 200 mm/year. However, in some cases displacement accelerations occur, that make velocities reach values up to 10 m/day.

The high frequency of landslide reactivations in the region is evidenced by the repeated occurrence of extensive damage of urban structures and infrastructures (Cotecchia et al. 2009a). In addition, since most landslide movements occur in winter, landslide activity appears to be seasonal and the slope piezometric regime appears to be a predisposing factor of instability (Terzaghi 1950). Another main predisposing factor is the limited strength and stiffness of the slope soils. In fact, the unstable slopes include either broadly disturbed clayey soils or pre-existing shear bands composed of pre-failed soils. Therefore, a correct interpretation of the landslide reactivation mechanisms in the region requires a thorough assessment of the evolution of the soil mechanical properties from small to very large strains, i.e. both pre and post-failure.

THE CASE STUDY: THE VOLTURINO LANDSLIDE

The geological and geomorphological setting of the ridge on which the urban area of Volturino is founded is shown in Figure 1 and is described in detail by Cotecchia et al. (2010). The old part of the town lies on Faeto Flysch (FAE), which is in contact to the west with Toppo Capuana Flysch (TPC). The slope on the western side of the urban area is location of a 1 km long, 300 m wide landslide body. This landslide has the geomorphological features of a mudslide and is representative of a class of landslide mechanisms widespread in the Daunia chain area (mechanism M2, as defined by Cotecchia et al. 2010). It represents an old landslide, since morphological features indicative of the existence of such landslide could be recognized through the analysis of topographic maps dating 1976. At present, the landslide activity is evidenced by damages to several buildings and infrastructure located along the landslide crest. Back-analyses of structure damages have demonstrated that these result from differential settlements relating to landslide movements in the crown area, which are mostly reactivated in winter (Strategic Research Project-119, 2009 – funded by Apulia Region).

According to the geo-morphological analyses, the mudslide appears to involve mainly TPC, that here includes mainly clays and marly clays. The mudslide toe appears to be located close to the Giardino stream (Figure 2). The outcropping geometry of the landslide body (Figure 1) suggests an influence on the landslide mechanism of the east-side contact of TPC with FAE (which underlies TPC before outcropping to the east). However, along the whole longitudinal section shown in Figures 1 and 2, the slope involves solely TPC down to large depths. Therefore, bi-dimensional analyses of the slope conditions have been carried out along the longitudinal section shown in Figure 2. At present, the landslide process is most active at the top, where retrogression occurs.

Preliminary inclinometer measurements taken in borehole I2 from October 2008 (zero reading) to April 2009, confirm the occurrence of shear localization between 45 and 55 m of depth (Figures 2 and 3), since incremental displacements of about 2-3 mm were logged both between 43 and 48 m and between 51 and 53 m depth. The occurrence of shearing at these depths has been confirmed by the recognition (through soil coring analyses) of the presence of remoulded clays at depths ranging between 43.5 and 45 m in borehole I2. The profiles of plasticity index
(PI), clay fraction (CF) and consistency index (IC; plotted as: [IC-1]×100), measured along borehole I2 are shown in Figure 3.

![Figure 1: Geological and geomorphological setting of Volturino area: 1) Sub-Apennine clays, 2) Toppo Capuana clays and marls, 3) Red Flysch, 4) Faeto Flysch, 5) geological contact (a: stratigraphic, b: tectonic), 6) landslide (a-crown, b-body), 7) field investigation boreholes, 8) axis of section shown in Figure 2 (after Cotecchia et al., 2009b, modified).](image1)

![Figure 2: Longitudinal section of the western slope of Volturino: 1) landslide body (a) and retrogression area (b), 2) Sub-Apennine clays, 3) Toppo Capuana clays, 4) Faeto Flysch, 5) geological contact (a: stratigraphic, b: tectonic), 6) field investigations: inclinometer (a), open pipe piezometer (b), Casagrande piezometer (c), borehole (d).](image2)
The PI profile is indicative of an increase in soil plasticity with depth, that is expected to induce a decrease with depth of the intrinsic strength parameters (e.g. critical state and residual friction angle) of the slope soils. Therefore, the soil composition properties are recognized to represent an internal factor promoting the generation of deep failure in the slope. The data in Figure 3 also show a decrease with depth of the consistency index of TPC clay samples. These samples were sheared in the triaxial apparatus. The shear test results are indicative of a decrease with depth of the clay brittleness and dilation angle and of a corresponding reduction of the clay strength parameters \( \sigma'_c \) and \( \phi'_p \). In particular, on the undisturbed sample taken at 45 m depth, peak shear strength parameters \( c'_p = 0 \) kPa and \( \phi'_p = 18.5^\circ \) were measured, which are lower than the mean peak strength parameters measured on all the other undisturbed TPC clay samples (\( c'_p = 15 \) kPa e \( \phi'_p = 18.5-20^\circ \)) tested in the area. Therefore, the index test data, along with the shear test data, appear to corroborate the recognition of a weakening of the slope clays with depth and of the presence of a highly disturbed clay layer located in the range of depths where shearing was logged by inclinometer monitoring.

![Figure 3: Borehole I2: vertical profiles of inclinometer incremental displacements and of the soil index properties.](image)

Piezometric measurements carried out in October gave evidence to a seepage flow in the slope with water table level at about 4.5 m depth below ground level. The water table was found to rise up to about 2 m below ground level in the following winter.

**LIMIT EQUILIBRIUM AND FINITE ELEMENT MODELS**

Limit Equilibrium (LE) back-analyses of the landslide body (Figure 2) were performed with the Morgenstern and Price method (1965). The top and the toe of the sliding body were assumed to be located as shown in Figure 2 and the depth of the sliding surface was varied in the analyses, from 20 m up to 55 m depth.

Seepage in the slope under study was simulated using finite element calculations, which resulted in piezometric predictions in accordance with the field
data. Such piezometric distribution was implemented in the LE analyses (SLOPE/W - GeoStudio 2004). In particular, two finite element (FE) seepage analyses were performed using the code SEE/P/W (GeoStudio 2004): the first consistent with the piezometric data measured at the beginning of autumn and representative of the summer seepage regime; the other consistent with a water table close to ground level and representative of the winter seepage regime in the slope.

Thereafter, the stress-strain conditions in the Volturino western slope were investigated by means of FE analyses performed with the code PLAXIS 2D (2003). In particular, the study has investigated both the initial development of a shear band within the slope and the following slope movement reactivations (in presence of a pre-existing shear band) under varying pore pressure regimes.

The mesh adopted in the FE simulations is composed of 15-node triangular elements (Figure 4). The mesh is coarser at the bottom and finer near the surface, where two predefined shear bands have been included in the TPC layer. In fact, two alternative depths of the shear band were accounted for in the analyses: 37 m and 50 m, in order to explore the slope response with slip surfaces varying in the whole range of depths resulting from both the inclinometer monitoring (Figure 3) and the LE analyses (as discussed later). A linear elastic-perfectly plastic model, with a Mohr-Coulomb strength criterion and a non associated flow rule ($\psi = 0^\circ$, where $\psi$ is the dilation angle), was adopted in all the analyses for all the materials, in accordance with the moderately brittle soil behaviour observed in the laboratory. The values of the mechanical and hydraulic parameters used in the FE simulations are reported in Table 1.

Strength parameter values equal to the rest of the TPC deposit were assigned to the shear band materials in the analyses aimed at the interpretation of the first failure mechanism in the slope. No evidence of the slope geometry before first failure could be gathered from analyses of old topographic maps, since this failure occurred far before the oldest available map (dating 1976). Therefore, in the FE analyses investigating first failure the current slope geometry was used and the corresponding results include an error due to this assumption. In the analyses aimed at simulating the slope reactivation movements, shear strength parameters representative of pre-failed soil conditions (post-rupture strength) were assigned to the shear band materials. Two alternative assumptions about the weakened soil parameters were used in the analyses (Table 1), in order to explore the variations in slope response for strength parameter values varying in the range of the mobilized strength parameter values resulting from the LE analyses, as discussed later. The analyses were carried out under plane-strain drained conditions and the pore pressure distributions were
defined by performing steady-state seepage calculations representing either summer or winter regimes. The initial stress state in the slope was achieved by simulating a simplified geological history of the slope. In particular, the gravity loading was applied initially to a horizontal deposit formed of an upper TPC clay layer, a FAE calcareous layer below and a deep Sub-Apennine grey clay (ASub) layer, assuming a value of the at-rest earth pressure coefficient \( K_0 \) equal to 1 for all the materials. Thereafter, the current geometry of the slope was implemented by simulating an excavation process, carried out by removing progressively ten layers of TPC. The water table was assumed to lie few meters below the slope profile at each excavation stage. At the end of the excavation phase, a steady-state seepage analysis was carried out to simulate the summer seepage regime in the slope. In this case, an impermeable boundary was assigned to the bottom of the mesh, whereas constant values of the groundwater head were prescribed along the left and right boundaries of the mesh, equal to 714 m and 494 m, respectively. A pore pressure equal to zero was imposed to the nodes corresponding to the Giardino stream in the lower part of the slope (Figure 4) and a negative hydraulic head, equal to -6 m, was prescribed to the rest of the nodes at ground level along the slope profile. The resulting water table is approximately at 5 to 6 m below ground level, except for the slope crest, where its depth reaches 15 m, and at the slope toe, where the water table is at ground level. This water table is consistent with the field data and the results of the summer seepage analysis performed with SEEP/W. The winter seepage regime was generated by substituting the hydraulic boundary conditions imposed along the slope profile for the summer regime with a free draining condition. This resulted in a water table coincident with ground surface along the whole slope profile.

**Table 1. Parameters adopted in the FE simulations of the Volturino slope.**

<table>
<thead>
<tr>
<th>Layer</th>
<th>( \gamma ) [kN/m(^3)]</th>
<th>( E' ) [MPa]</th>
<th>( \nu' ) [-]</th>
<th>( c' ) [kPa]</th>
<th>( \phi' ) ['']</th>
<th>( \psi ) ['']</th>
<th>( k ) [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sub-Appenine Clays (ASub)</td>
<td>18.5</td>
<td>100</td>
<td>0.25</td>
<td>40</td>
<td>23</td>
<td>0</td>
<td>1x10^-10</td>
</tr>
<tr>
<td>Faeto Flysch (FAE)</td>
<td>20.0</td>
<td>100</td>
<td>0.25</td>
<td>40</td>
<td>25</td>
<td>0</td>
<td>1x10^-08</td>
</tr>
<tr>
<td>Toppo Capuana Clays (TPC)</td>
<td>18.8</td>
<td>70</td>
<td>0.25</td>
<td>13</td>
<td>20</td>
<td>0</td>
<td>5x10^-09</td>
</tr>
<tr>
<td>Shear band – 1(^{st}) assumption</td>
<td>18.8</td>
<td>70</td>
<td>0.25</td>
<td>8</td>
<td>18.7</td>
<td>0</td>
<td>5x10^-09</td>
</tr>
<tr>
<td>Shear band – 2(^{nd}) assumption</td>
<td>18.8</td>
<td>70</td>
<td>0.25</td>
<td>0</td>
<td>20.0</td>
<td>0</td>
<td>5x10^-09</td>
</tr>
</tbody>
</table>

**RESULTS OF THE ANALYSES AND DISCUSSION**

The LE results (Figure 5) suggest that, when assuming the winter seepage regime, the critical depth of sliding is about 37 m if \( c' \) is supposed to be zero, with a mobilized friction angle \( \phi'_m = 19\degree \); it increases when \( c' > 0 \), reaching, for instance, about 45 m of depth for mobilized \( c'_m = 5 \) kPa and \( \phi'_m = 18\degree \). Therefore, the results indicate that the failure mechanism in the slope is deep (\( > 30 \) m) and that, during winter reactivations, the shear strength mobilized in the shear band crossing the TPC clays is lower than the peak strength measured for such clays when undisturbed (\( c'_p = 15 \) kPa e \( \phi'_p = 18.5+20\degree \)). When the summer seepage regime is considered, the safety factor, \( F \), of the landslide body increases of about 7\%10\%; for example, assuming \( c'_m = 5 \) kPa, \( \phi'_m = 18\degree \) and a maximum sliding depth of 35\%45 m, \( F \) increases by 7\%. Such variations in \( F \) represent the decay in slope stability resulting from the feeding of the seepage system in the slope during autumn and winter. Therefore, they corroborate the hypothesis that the landslide reactivation results from the seasonal variations of the pore water pressures.
The FE results show that the slope before failure (assuming $c'$ and $\phi'$ of the undisturbed TPC also in the shear band) was stable when the summer seepage regime is considered. At this stage three shear zones appeared to develop, the first with toe in the centre of the slope, the other in the area immediately above the stream and the third in the lower portion of the slope, all propagating backwards at depth and reaching the contact between TPC and FAE layers (Figure 6). The factor of safety of the slope at this stage was calculated to be $F = 1.15$ by means of the strength reduction technique (PLAXIS 2D 2003). The deformed mesh associated to $F = 1$ condition is shown in Figure 7 and indicates that, if weakening of the soils had occurred in the whole slope at this stage, given all the other conditions, this would have brought the slope to fail according to a deep-seated mechanism, mostly active up-slope, that would have reached the contact surface between the TPC and the FAE layer (maximum depth of 65 m). Therefore, the simple model used confirms that in the slope under study first failure was deep. However, the results in Figures 6 and 7 should be considered indicative of the first landslide activation only in first approximation, since the identification of the original first failure would have required implementing in the analyses the original geometry and seepage conditions, which are unknown as recalled before.
Afterwards, the analyses were continued in order to explore the process of landslide reactivation. The presence of a band of pre-failed material was simulated by reducing the strength parameters in one of the shear bands included in the mesh. In particular, the shear band at 37 m depth was activated in a first analysis and then the one at 50 m depth was activated in a second analysis. The weakened soil parameters were first assumed to be \( c' = 8 \text{ kPa} \) and \( \phi' = 18.7^\circ \) (1st assumption in Table 1). With these parameter values, the slope with the 37 m deep shear band remained stable in summer, although further concentration of plastic shear strains occurred in the central portion of the band, that is connected to the locus of shear localization resulting from the previous analysis and outcropping at mid-height on the slope. In this portion the maximum values of the incremental shear strain (\( \Delta \gamma = 0.18\% \)) were reached. In addition, further shear localization occurred down-slope (Figure 8), with toe just above the stream. After this modelling stage, the *winter seepage regime* was activated. The resulting effective stress changes appeared to induce a progression of the two deformation mechanisms described above, followed by the slope failure.

Figure 7. Deformed mesh at the end of the strength reduction analysis for summer season.

Figure 8. Contours of incremental shear strain due to the activation of a 37 m deep softened band \((c' \neq 0)\) for summer season.

Figure 9. Contours of incremental shear strains due to the activation of a 50 m deep softened band \((c' \neq 0)\) for summer season.
The alternative assumption of a cohesion intercept equal to zero and a friction angle of 20° in the 37 m deep shear band led, for summer conditions, to the development of the same straining mechanisms as those described above, although with higher shear strains in the upper part of the band (maximum incremental value $\Delta \gamma = 0.72\%$). Also in this case failure occurred in winter. The alternative activation of the 50 m deep shear band, with $c' = 8$ kPa and $\phi' = 18.7°$ and a *summer seepage regime* in the slope, induced a more pronounced concentration of shear strains in the shear band with respect to what resulted for the 37 m deep band. As shown in Figure 9, two shear zones with maximum values of $\Delta \gamma = 0.72\%$ developed within the softened band, which were connected to the two toes activated since the analysis with no band. Nevertheless, the slope remained stable in summer. Differently from the case of the 37 m deep band, the simulation of the *winter regime* enhanced accumulation of shear strains only in the upper part of the band, as shown in Figure 10. Moreover, a very shallow failure mechanism was also activated in a portion of the slope where the gradient increases and where local shallow bending of inclinometer I2 was observed in March 2009 (Figure 3). With regard to the overall slope stability, after implementing the *winter regime* in the analysis, the slope became unstable in the crest area, where maximum vertical settlements of about 11 mm were calculated (Figure 11). In this case the contours of the vertical incremental displacements gave evidence of a retrogression also up-slope the crest of the shear band. This area corresponds to a portion of the urban centre where buildings are currently severely damaged. When adopting zero intercept cohesion and $\phi' = 20°$ within the 50 m deep shear band in *winter-like* conditions, a more diffuse development of plastic straining and larger shear strains in the band were found to occur. Although the maximum vertical settlement at the crest was about 12 mm, in this case the retrogressive process was less evident.

![Figure 10. Contours of incremental shear strains due to the activation of a 50 m deep softened band ($c' \neq 0$) for winter season.](image)

![Figure 11. Contours of incremental vertical displacements due to the activation of a 50 m deep softened band ($c' \neq 0$) for winter season.](image)
CONCLUDING REMARKS

The present paper discussed the case study of the landslide process currently affecting the western slope of Volturino, which is representative of the reactivation of mudslides of class M2 (Cotecchia et al. 2010) in the Daunia Apennines. The landslide has been studied by means of geological surveys, field investigations and monitoring, LE and FE analyses. In particular, limit equilibrium calculations and finite element modelling were carried out to interpret a mechanism of landslide reactivation consistent with the in situ observations.

The limit equilibrium analyses showed that the strength currently mobilized along the sliding surface is just slightly lower than the peak strength measured for the Toppo Capuana clays forming the slope. Therefore, the LE analyses confirmed that the slope is only marginally stable in summer and becomes unstable in winter. The sliding surface was deep, according to both the field monitoring and the LE analysis results. Also, the finite element results indicated that the slope tends to fail under winter seepage conditions according to a deep-seated mechanism. Localization of shearing within a softened band had the tendency to develop according to an up-slope progression; higher shear strains were observed in the upper part of the slope, with toe at mid-height of the slope. Moreover, evidence of local retrogression was shown by vertical displacements predicted in the area up-slope the landslide crest when assuming a 50 m deep band in the slope, in accordance with the damage pattern observed in situ in buildings located thereby.

Further developments of the numerical modelling will be carried out using more advanced soil constitutive models in order to explore the initial activation of the shear band and the following reactivation processes, but avoiding the inclusion of the band of weakened soils in the slope.

REFERENCES


