A deep seated movement in a marly-arenaceous formation: analysis of slope deformation and pore pressure influence

S. Assefa¹ A Graziani and A Lembo-Fazio

Roma Tre University, Engineering Department.
Civil Engineering Section, Via Vito Volterra 62, 00146, Rome, Italy

Abstract: The case history of a deep-seated slope movement in a complex rock formation (Marly-Arenaceous Formation) is analyzed. The movement, monitored for more than 20 years, was recognized after the discovery of intense cracking in the concrete lining of a hydraulic tunnel running across the slope. The time history of displacements shows that the ongoing deformation process is essentially a stationary creep phenomenon, also influenced by transient variations in pore pressure distribution. The slip surface is formed by a tectonized clay gouge layer and the mobilized shear strength is close to residual. The slope has been modelled (UDEC code) as a complex blocky structure defined by several joint sets: bedding joints, inclined and sub-vertical discontinuities. Different geometries of the slip surface, reasonably varied within the range of hypotheses compatible with field evidences, have limited influence on the limit friction angle of the slip surface. Joint patterns have influence on the deformation mode and minor impact on the mobilized friction angle. The model response is less sensitive to the water level at the slope toe as compared to the rise of groundwater table.

1. Introduction

The slope movement analyzed in this study was recognized in 1990 after the excavation of a hydraulic tunnel in the right bank of the Chiascio River (Italy) (figure 1). The movement affects a large portion of a ridge which has the longitudinal direction approximately perpendicular to the river. Just passed the sliding area, the river is crossed by a rockfill dam, which was intended to create a reservoir for water supply, by a diversion tunnel located on the right bank. So far, the diversion tunnel has been operated only once, in 1991-1992, not for regular water conveyance but for temporary deviation of the river flow.

The first evidence of ongoing slope deformation was the intense cracking of the concrete lining in a tunnel section located well inside the ridge, at a distance from the intake of some 250 m, under an average overburden of 50 m. The damaged tunnel stretch was soon interpreted as the place where the tunnel axis intersects a deep-seated slip surface.

The entire area of the dam reservoir, included the sliding slope and the dam foundation, is formed by an outcrop of the Marly-Arenaceous Formation, a Miocene flysch characterized by alternating sequence of marl, sandstone and calcarenite layers. As observed in many flysch formations of Italian Apennines [1], joints are often slickensided as a consequence of tectonic shearing and bending deformation of strata.

¹ Corresponding author (email: sirajm2000@gmail.com)

Tel.: +39 06 57333482, Fax: +39 06 57333469
The presence of major discontinuity planes of low strength represents a key factor for stability analysis in slope and dam engineering and has often required specific investigations [2], [3]. Weak layers of small thickness, such as clay gouge interbeds or thin shear bands, may be easily overlooked during ordinary borehole investigations, particularly in structurally complex formations [4], such as flysch and layered limestone formations [5], [6], [7].

![Study area diagram](image-url)

**Figure 1.** Study area.
Figure 2. Regularly layered marly-arenaceous rock mass in the east side of the ridge (a); disturbed rock mass structure at the toe of slope (b); curved layers exposed during the excavation works for tunnel intake (c); main cracks and fissures seen in the concrete lining of the diversion tunnel (d).

In the present case, it is likely that most of the sliding surface is seated along an over-thrust plane gently dipping towards the Chiascio River valley. This circumstance adds further complexity to the structural setting of the slope and can explain some particular features of the sliding mass, whose overall volume has been estimated of some 18 Mm$^3$.

This paper focuses on the influence of rock mass structure (block shape, joint pattern) and pore pressure distribution on the mechanical response of the slope. First, the main results of geotechnical investigations and displacements measurements performed in more than 20 years are discussed in order to obtain a reference framework for slope modelling. The kinematic characters of the movement are essentially those of a planar-sliding (average velocity of 13-17mm/y) presumably localized along tectonized clay interbeds.

The structural setting of the slope was modelled by a discontinuum medium approach [8]. Therefore, the influence of rock mass structure (slip surface geometry, joint set orientation and continuity) on deformation mode and mobilized strength has been investigated. Finally, the influence of water table profile inside the slope and water level at the toe of the slope has been analysed.

2. Geological setting and morphology of the slope

The stretch of Chiascio River basin considered in this study is entirely based on a thick sequence of Miocene flysch deposits (figure 3). The main valley as well as the lateral gullies are carved in the same rock formation.
The Marly-Arenaceous Formation is characterized by a layered structure, at least in its less disturbed portions [9], [10]. The different layers can be grouped on the base of the prevailing lithological components:

a) Lower marls unit, with rare thin layers of clay shale,

b) Marly-arenaceous-calcarenitic unit,

c) Calcarenite unit, sparsely present as single layers,

d) Upper marls unit.

The lower marls are characterized by a more regular and continuous bedding. The a) unit was mostly exposed during the excavation for dam foundation, nearby the toe part of the sliding area (figure 3). The excavation works, started in 1984, required the removal of debris and loosened rock up to 15 m depth below the river bed elevation.

![Figure 3. Longitudinal cross-section of the ridge with relevant geological data, location of piezometer boreholes and position of some geodetic targets.](image)

The slope movement herein considered affects a large portion of a long ridge, upstream to the right abutment of the dam. The sliding mass seems mostly composed of alternating layers (thickness, 0.1–0.2 m) of unit b) lithotypes, with the upper marls unit and some debris deposits at the top. The Calcarenite unit consists of two separate strata (each one formed by single layers of 0.1–1 m for a total thickness of 3–4 m) intermingled with the thinner layers of unit b).

While the upper portion of the ridge is gently dipping (10°) towards the Chiascio valley, the frontal slope and lateral flanks are steep (25-40°) and mostly covered by a talus of rock debris and remolded clay. This situation hampers the visual detection of any trace of the slip surface at the toe of the slope (el. 280 m a.s.l.). The upper limit of the ridge (S sector) is characterized by a plateau, located at an average elevation of 395 m a.s.l.

In the east flank of the ridge a regular monocline structure (bedding dips to W, dip angle 20–40°) can be clearly identified. The same structure, with remarkably continuous and consistent orientations of the bedding joints, could be observed also on the foundation surface of the dam, as reported by the construction surveys. Conversely, the west flank of the ridge (bordered by the gully called “Fosso della Torre”) is characterized by a less consistent orientation of bedding, especially in the lower part where bedding orientation is more dispersed, although dip direction from E to NE seems prevailing, with dip angle of 25-50°. Therefore, the overall pattern of the bedding planes suggests the hypothesis of a syncline structure, with axial plane having N-S direction.
The N sector of the ridge could be carefully inspected during the excavation works for the intake structure of the diversion tunnel (figure 2c) and, after the discovery of the ongoing movement, by extensive borehole investigations. In this area the rock mass exhibits a higher degree of fracturing and local disarrangement of the bedding joints.

A fundamental evidence acquired from borehole loggings in the N sector is the presence of quaternary alluvium under the Miocene flysch forming the toe of the slope (figure 3). This circumstance can be reasonably explained as the consequence of an overthrust deformation. By comparing the position of the contact between alluvium and marly-arenaceous formation at different boreholes, it can be argued that the basal plane of the overthrust is almost horizontal and is located at an elevation slightly lower than the river bed. Moreover, the cumulated amount of shearing displacement would be as high as 100-150 m. If such hypothesis holds true, it is likely that also the current process of slope deformation is governed by the same shearing band, i.e., by a layer of strongly tectonized clay gouge.

3. Survey of tunnel damage and slope displacement

As already mentioned, the occurrence of a slow movement in the ridge crossed by the diversion tunnel of the dam, was recognized only after the completion of the tunnel lining. Nowadays, surface evidences of the ongoing deformation can also be detected, especially on the east boundary of the sliding area, where the pavement and curbs of some country roads appear locally displaced and fissured.

The diversion tunnel departs from the right flank of the valley, at an elevation of 287 m a.s.l., approximately 15 m higher than the toe of the slope, and runs through the long ridge with a direction almost perpendicular to the river (figure 1) for some 900 m, before turning to the W, by taking a N238° direction.

The clear evidence of a deep-seated movement came from the heavy damage of the tunnel lining, not yet in operation, which consists of a 4.5 m diameter, 0.5 m thick cast concrete ring. Cracking and opening of the construction joints were localized in a 23 m long stretch of tunnel, between chainage 235 and 258 m from the intake section. The limited width of the damaged stretch is strongly suggestive of a deep movement with localized shearing. The damaged section of the tunnel is under an average overburden of 50 m and is located, in plan, some 40 m inside the lateral boundary of the movement, as successively demonstrated by the geodetic survey of surface displacements.

The fissures in the lining were repeatedly sealed but formed again, then four 4 couples of crack-meters were installed across the main cracks (figure 2). A clear picture of the relative displacement trend was obtained in 1991-1992, during a short-time operation of the diversion tunnel as temporary discharge tunnel in order to allow the completion works of the dam bottom outlet. In that period of temporary operation, the reservoir water level reached the maximum elevation of 296.5 m. After 3 months of tunnel operation, with average water level at 287 m el, the maximum stroke (50 mm) of the displacement gauges was achieved. The relative displacement vectors measured across the cracks exhibit a negligible plunge and a direction close to the tunnel axis.

3.1. Geodetic survey of displacements

A first network of targets (N1-N11) for the geodetic survey of surface displacements, restricted to the frontal slope of the ridge (N sector), was implemented to integrate the crack-meters installed in the damaged section of the tunnel, before starting the temporary filling of the dam pool. The geodetic network was thereafter extended to the S and W areas, with targets (N12-N20) at greater distances from the N slope, in order to estimate areal distribution and limits of the movement (figure 1).

Since then, all the targets have been continuously monitored by a total station, installed on the opposite side of the valley, at the top of the concrete walls of the dam spillway. Measurements are performed regularly once a month. The prevailing components of displacement are in the horizontal plane; the vertical components are much smaller and usually are not processed.
3.2. **Tunnel extensometer**
In 1994, it was decided to install a steel lining within the damaged section of the tunnel. The annular gap between two steel pipes was not sealed but endowed with a “telescopic joint”, in order to accommodate differential axial displacements between the two tunnel stretches separated by the cracked zone and, therefore, hosted, respectively, inside and outside the sliding rock mass. The telescopic joint was supplemented by an electric dial gauge, thereafter referred to as “tunnel extensometer”.

3.3. **Inclinometer measurements**
The displacement data available after some years of geodetic monitoring were instrumental to trace the approximate boundaries of the slide surface. The lateral limit of the movement along the E flank of ridge were considered with particular attention due to proximity of the right abutment of the dam. Therefore, it was decided to refine the survey by installing a set of inclinometer tubes (figure 1). Taking advantage of the shallow depth of the slip surface in this peripheral zone of the movement, the length of the 8 boreholes (S1…S29 series in figure 1) could be limited to 20-45 m. The boreholes were drilled in 2006-2008, since then, readings have been carried every two months by removable sliding inclinometer probe.

4. **Geotechnical investigations**
The complex structure of the rock mass, as expected from the geological investigations, is clearly confirmed by a comprehensive analysis of borehole loggings (e.g., borehole S27 in figure 4). Borehole investigations have been focused on the N sector, particularly, in the zone of tunnel intake.

Bedding joints have discordant orientation in the upper and lower portion of the ridge, especially on the W flank, as already recognized from field surveys. This situation suggests that the entire body the ridge can be divided in different zones, characterized by a variable degree of fracturing and disarrangement of the rock mass.

![Figure 4. Curves of displacement vs depth of the inclinometer borehole S27 (zero reading on 5/2/2007); RQD diagram and borehole stratigraphy.](image)

Though results obtained from borehole loggings are sparse, chaotic structure typically accounts for 10 to 40% of total borehole logging data. Comprehensive analysis of joint dips from borehole loggings shows that sub-horizontal strata and steep joints are inclined, respectively, between 5-20° and 30-60°.

For instance, the S3 borehole, drilled through the toe of the N slope of the ridge, shows that the base marly-arenaceous-calcarenitic unit has RQD value in the 40-70% range with discontinuities dipping between 18-20° while the upper layer of disturbed marl RQD value is from 15-40% with discontinuity inclined at 12-20°. Lower RQD values between 10-40 % and even null values were obtained in the more disturbed zones, presumably the zones of higher shearing deformation. Borehole S27 (figure 4 ), located
in the NW side of the ridge, shows RQD values of 50-70% for the undisturbed marly-arenaceous-
calcarenitic unit and RQD of 15-30% at the depth of the sheared zone.

4.1. Mechanical properties
The intense shearing deformation occurred along the basal zone of the slope suggests that the mobilized
strength is likely to be close to residual conditions for most of the sliding surface.

The properties of the sheared material (clay gouge) were examined from the exploratory window
opened at the damaged portion of the tunnel, where the slip surface intersects the tunnel axis. The core
of the shear zone consists of a 0.15-0.20m thick band of clay gouge (figure 5) entrapping fragments
formed by shear grinding of the rock materials (mainly marl and sandstone). Index properties of the clay
gouge samples are: Clay = 46%, Silt = 45%, Sand = 9%, γs = 27.2 kN/m3, wL = 72.6%, wP = 40.2 %, Ip
=32.4%, CaCO3 = 20.1%. This material possess high plasticity and its grain size composition is
dominated by a high silt-clay proportion and less coarse materials. The results of shear box testing in
drained condition are: peak, cφ =21.5 kPa and φp = 15° and residual, φr = 7.7° (figure 6).

![Figure 5. Clay gouge interbed exposed at the damaged tunnel section (a); detail of photo (a) with the deformed steel set (b); the observed thickness of the clay layer is approximately 0.2 m.](image)

![Figure 6. Results of direct shear tests on clay gouge sample.](image)
while for the marly-arenaceous formation forming the “bedrock”, i.e., below the shearing zone at the base of the slope, GSI values of 55-60 seem more appropriate. Rock joints can be characterized by JCS= 30-40 MPa and JRC= 3-5. Slickensided joints are frequently found in borehole cores (figure 7), mainly at the contact between marly and arenaceous layers but also within claystone beds.

Typical friction angles from in situ shear tests on slickensided and laminated bedding joints of the Marly-Arenaceous formation are in the range of 12-15° [11], [2]. Laboratory shear strength tests between clay filling and limestone contacts were also conducted by [6]; $\phi_r = 13.5-18^\circ$ was obtained. In addition, from in situ and laboratory direct shear tests, [12] obtained peak and residual strength of 14° for sandstone – sandstone contacts and $\phi_p = 19^\circ$ and $\phi_r = 13-18^\circ$ for sandstone–claystone contacts.

![Figure 7](image)

**Figure 7.** Typical aspect of joint surfaces: joint surface recovered from borehole S27 at depth of 7.4m (a), from borehole S3 at depth of 24.9m (b), presumably close to the depth of the slip surface, and from borehole S23 at depth 13m (c).

A report by [13] shows $\phi_r$ in the range of 10-26° after several ring shear test performed on specimens from an outcrop of the same Marly-Arenaceous in a nearby site, also containing thick clay interbeds. Ring shear tests from [14] show that $\phi_p = 13-27^\circ$ and $\phi_r = 10-15^\circ$ for different types of clay-rich interbeds.

Cross-hole tests up to 30m depth were performed to assess the elastic properties of the dam foundation, formed by the alternate layers of sandstones, marl-clay and calcarenite of unit a). Elastic properties ($E_d, v_d$) were estimated from compression and shear waves velocities (figure 8). A comprehensive study by [15] on several dam sites in Italy shows good correlation between plate loading tests and seismic measurements. Oberti et al. [2] also performed several plate loading tests, perpendicular and parallel to the stratification in marly and sandstone units. Deformation moduli of 18-24 GPa were obtained for tests conducted parallel to the stratification and 16-18 GPa for tests perpendicular to the stratification.
Figure 8. Results of cross-hole test for dam foundation, in boreholes A-B (a), and p-wave velocity from seismic refraction investigation P15 at the slope toe (b).

Investigations conducted on the slope body using down-hole and seismic refraction show that the first few meters, composed of loose detrital layers (i.e., silt and gravel deposits), have P-wave velocity of lower than 0.5 km/s while for the fractured marly-arenaceous formation P-wave velocity increases to 0.5-0.8 km/s. Finally, P-wave velocity of 0.8-3 km/s can be assumed for massive sandstone and calcarenite layers (figure 8).

4.2. Permeability tests and piezometer measurements
Lugeon tests were conducted in the area of the dam foundation. The permeability of the rock mass lies between $1 \times 10^{-7} - 3 \times 10^{-7}$ m/s (figure 9). Similar results were obtained by [2] in the area of Ridracoli dam, with average permeability of $1 \times 10^{-7} - 4 \times 10^{-7}$ m/s at high and shallow depths, respectively.

Figure 9. Frequency distribution of Lugeon values for a test conducted at the valley.

It is worthwhile to remark that the Marly-Arenaceous formation may exhibit higher hydraulic transmissivity in the direction of strata, with seepage paths limited to discontinuities planes and involving almost exclusively the more fractured sandstone strata, while the clay-rich layers represent low-permeability barriers.
To investigate the pore pressure distribution inside the slope, four deep boreholes were equipped with piezometer cells (figure 3). Each borehole hosts two cells installed, respectively, above and below the presumed elevation of the slip surface. During the monitoring period the water level of the dam pool never exceeded 282 m a.s.l. Therefore, the measured pore pressures have been mainly influenced by rainfall and infiltration process.

5. General hypotheses and kinematic characters of the movement

5.1. Geometry of the sliding mass

Borehole investigations and survey of the N sector of the ridge have shown that a basal slip plane can be reasonably assumed at the top of the buried alluvium layer. Inclinometer profiles and target displacement vectors can give further support to this assumption.

Although the density of measurement points decreases in the S sector, at greater distance from the river bed, magnitude and direction of horizontal displacements measured at different points are generally consistent (figure 11). The overall direction of displacement is approximately parallel to the longitudinal direction of the ridge, with some eastward rotation at the toe of the slope. The basic hypothesis of a planar sliding with some internal shearing of the moving body seems therefore reasonable.

The position of the damaged tunnel section represents a further restraint to trace a likely longitudinal profile of the slip surface (figure 10). Finally, the upper limit (S boundary) of the movement can be located mainly on the base of the morphology of the ridge, i.e., by observing the presence of a plateau with some deformation traces at elevation 400 m a.s.l. The distance from this plateau to the river bed, measured along the longitudinal axis of the ridge, is approximately 1000 m.

The slip surface can be reasonably drawn as a near horizontal plane in the toe part. The slip surface is certainly more inclined in the upper part but the effective geometry cannot be defined in detail. This last issue will be further investigated by comparing different DEM models.

The lateral boundary of the movement can be traced more confidently on the E side, on the base of displacement measurements, while on the W side the separation line between stable and unstable zone fades away (figure 11). The slide boundary is close to the top of the dam but the movement does not affect the abutment zone, as demonstrated by the null displacement of targets N12 and N19.

Figure 10. Slip surface reconstructed at section x-x considering the position of damaged tunnel section and typical dip angles (20-40°) of layers at the East side of the ridge, as evidenced from field investigations (all dimensions are in m).
The current kinematics of the movement can be outlined as a compound mechanism in which a blocky system is sliding on a low inclination basal plane. The depression along the upper profile of the ridge, between the targets N7 and N8 (figure 3), may represent a morphological evidence of past inter-block deformation. Inclinometers are located only at the toe of the slope: the deformation profiles generally show a rigid mass type of movement, with a clearly defined slip plane (figure 4), but in some cases a smeared deformation mode is also observed.
5.2. Time history of displacements
The time history of measured displacements (figure 12) show that the ongoing deformations are characterized by stationary velocity. Apart from the much higher velocities recorded in 1991, during the temporary operation of the diversion tunnel, the average velocity of the targets lies in the 13-17 mm/y range in the period 1992-2001 and tends to decrease in the following years. However, an increase in velocity was observed recently after the last measurements. The tunnel extensometer exhibits a similar trend and consistent magnitude of displacement.

A secondary fluctuation of velocity is also observed. This behavior is likely a consequence of transient seasonal variations in pore pressure distribution. Some targets also show alternated phases of negative and positive velocity, which may be tentatively explained as the combined effect of “stick-slip” phenomena and/or temperature variations. Additional difficulty in analyzing the velocity trend (figure 12) stems from the low frequency of measurements.

6. Pore pressure regime and rainfall influence
The average groundwater table, as recorded by piezometer cells in borehole A, B, C and D, progressively decreases towards the slope toe (figure 3). The cells in the uppermost borehole (D1 and D2), which are located at 80 m and 97 m below the ground surface, respectively, have given null pore pressures for most of the measurement period. The significant difference in piezometric head between the cells in the same vertical reflects a complex groundwater situation, where fractured rock aquifer are separated by the less permeable marly layers.

Readings from piezometer B2, located in the same zone of the damaged tunnel section, show a local falling of the groundwater table. This fact can be explained as the effect of a higher degree of loosening, and therefore higher permeability, of the rock mass. The steep flanks of the ridge in this sector may also facilitates the lateral drainage process.

The time-lag between pore pressure increases in piezometers and rainfall was also analyzed. The maximum fluctuation of groundwater levels around the average value is some 10 m. Peaks in piezometer levels seem best correlated with three months cumulated rainfall exceeding the average values (figure 13). Correlation analyses were also attempted between displacement velocities and rainfall, but results are unreliable due to the low frequency of measurement (figure 13).
Figure 13. Correlation between 3-months cumulated excess rainfall (a), represented by the shaded area, variations in piezometric levels (b) and average displacement velocity of different measurement points (c).

7. DEM modelling of the slope

The aim of this first phase of modeling is to investigate the influence of the structure of the moving mass (i.e., geometry of slip surface and joint pattern) on slope displacements and mobilized friction angle of the slip surface. A 2D discontinuum modeling approach (DEM) was adopted via the UDEC code [8]. In the present modeling block materials are linearly elastic while joints are ideal elasto-plastic and obey the Mohr–Coulomb strength criterion. Block materials have been assumed linearly elastic while joints are ideal elasto-plastic and obey the Mohr–Coulomb strength criterion.

The following analyses are primarily a back analysis of the slope deformation observed in more than 20 years of field monitoring, which has allowed to categorize the movement as a “slow” process. Dynamics aspects often related to catastrophic slope failures (e.g. [16],[17]) are not essential for the present case. Therefore, local damping has been applied, as customary in UDEC code for static or quasi-static processes.

The slope has been modelled as a complex blocky structure defined by several joint sets. Bedding planes, gently inclined and markedly continuous are crossed by steeper and sub-vertical joints.
Frequency and number of joint sets have been increased in the more disturbed portions of the slope, i.e., at the toe and in the transition zone, i.e., where the dip angle of the slip surface changes. The plane section considered in the model is parallel to the longitudinal axis of the ridge (figure 14).

The mechanical properties of the joint sets are illustrated in table 1. All the joint sets have the same stiffness properties ($k_n$ and $k_s = 0.1k_n$) and purely frictional strength. The normal stiffness of the joints has been calibrated so that the equivalent rock mass modulus is in agreement with the elastic modulus suggested by the GSI approach and in the same range of cross-hole data (see Section 4.1).

The present morphology of the slope has been formed by simulating an “erosion” process, starting from an ideal situation with horizontal ground surface and in situ stress condition with horizontal-to-vertical stress ratio $K_0 = 0.5$. The reference groundwater table (stationary conditions) considered in the following analyses represents the average profile obtained from piezometer measurements (vertical boreholes A, B, C and D).

The influence of rock mass structure on the mobilized friction angle of the basal slip surface has been investigated. The shear strength reduction method was employed to calculate the failure condition for a given joint pattern. Equilibrium conditions have been satisfied for each step of reduction in basal friction angle. To this aim, an unbalanced forces ratio [8] of $10^{-6}$ has been adopted as a convergence criterion. Normally, a progressively higher number of calculation steps is required as the model approaches the plastic collapse. Additionally, the displacement histories of some grid points were monitored. The selected control points correspond to the position of specific geodetic targets (N1-N11), located at the toe of the slope as well as in the transition and upper zone of the ridge.

As highlighted by previous studies [18], the problem of finding the strength corresponding to the onset of collapse involves computational difficulties and ambiguous results are likely to be obtained. The expedient of progressively decreasing the step-size of the strength reduction as the system approaches the stability limit proved to be useful but not decisive.

Finally, for the present analyses, it was decided to adopt the following empirical approach. The limit friction angle has been identified by inspecting the displacement vs mobilized friction curves, i.e., it corresponds to the point of maximum curvature, where the displacement exhibits a sharp increase (e.g., figures 15 and 17). The aforementioned criterion may not be fully satisfactory, in general, but seems effective in order to compare the responses of the different models.

Moreover, a factor of safety (F.S.) can be easily calculated as the ratio of the tangent of the actual friction angle to the tangent of the failure friction angle, as suggested in [19].

The shear strength assumed for the bedding planes corresponds to the typical properties determined for the contact planes between marly and arenaceous layers (see Section 4.1). The pattern of cross joints included in the model represents an idealized picture of the blocky structure of the marly-arenaceous formation. Persistent or discontinuous joints can be considered (Model A and Model B in figure 16, respectively), as shown in the following sections.

A particular remark concerns the shear strength of the cross joints (joint sets 3 and 4). A minimum friction angle of 30° and 18° is required for the cross joints of Model A and B, respectively, in order to mobilize a translational movement of the rock mass along the basal plane. For lower friction angles, a
different collapse mechanism is obtained, i.e., the slip failure of a steep wedge of rock mass in the frontal zone of the slope (between points N3 and H in figure 14). This local failure mode was to be avoided since it is not in agreement with the observed characters of the real movement.

The aforementioned difference in the friction angle required for cross joints can represent a measure of the interlocking contribution to shear strength due to the non-persistent joints (figure 16). Conversely, it has been found that the friction angle of cross joint sets has limited influence on the mobilized friction angle of the basal slip surface.

Table 1. Joint properties adopted in UDEC simulations.

| Description       | Joint spacing | Dip angle | Friction angle | Normal stiffness | Shear stiffness |
|-------------------|---------------|-----------|----------------|------------------|----------------|
|                   | S (m)         | $\psi$ (°) | $\phi$ (°)     | $k_n$ (GPa/m)    | $k_s$ (GPa/m)  |
| Bedding planes (1)| 8-16          | 10        | 18             | 2.3              | 0.23           |
| Joint set (2)     | 12-24         | 40        | 30             | 2.3              | 0.23           |
| Joint set (3)     | 10-20         | 30        | 30             | 2.3              | 0.23           |
| Joint set (4)     | 20-40         | 80        | 30             | 2.3              | 0.23           |
| Slip surface      | -             | 0-10.5$^b$| Varies$^c$     | 2.3              | 0.23           |

$^a$Spacing is different for the upper and lower (transition and toe) portions of the slope.

$^b$Range of dip angles of the slip surface (c) assumed for the model in figure 14.

$^c$Friction angle of slip surface is varied according to the strength reduction method.

7.1. Influence of blocky structure

7.1.1. Block size. Since the joint spacing of the model must be increased with respect to the real situation, to get a reasonable number of blocks for modelling [18], [20], it is worthwhile to analyze the influence of block scaling. The effect of block size has been preliminary investigated by comparing the mechanical response of two simplified models in which only two joint sets, (1) and (4), are implemented with spacing S and 0.5*S, respectively (figure 15).

The results of figure 15 show that block scaling affects the overall stiffness of the rock mass [21] but does not change the mobilized friction at collapse. The block size was further decreased (1/3 of block size shown in figure 15a), again, the model results show an increase in displacement magnitude but the effect on the mobilized friction angle of the slip surface is still negligible. Similar results were obtained by [18], where the failure friction angle was found to be progressively less sensitive to block size as the number of blocks becomes sufficiently higher.

![Figure 15. Models with joint spacing S (a) and 0.5*S (b); calculated displacement of point N3 vs mobilized friction angle of the slip surface (c).](image-url)
For the models shown in figure 15, as the number of blocks increases the model exhibits not only a basal sliding but also significant shearing along the sub-vertical joints and toppling deformation in the frontal part of the slope. These features are not in agreement with the observed deformation style.

7.1.2. Joint pattern. A set of analyses were performed with the model of figure 14 (Model A), in which all the joint sets are continuous. On the base of field survey and borehole data, a staggered joint pattern has been also implemented (Model B in figure 16), in order to analyze the influence of non-persistent joints and interlocking.

The model with staggered joints exhibits less internal shearing of the rock mass and a more uniform distribution of horizontal displacements. The deformation mode of Model (A) is more complex: the lower portion of the slope, particularly N1 and N3 points, moves more than the upper portion (figure 17), where shearing deformations along the joints dipping towards S (joint set 2) are also activated. Similar influences of joint pattern on the deformation mechanism of rock slopes have been reported by [18], [22], and [23].

The mobilized friction angle at collapse is 7.6° and 7.2° (figure 17), respectively, for Model (A) and Model (B). The limit strength is therefore close to the residual friction obtained from shear box tests. For the staggered joint model (Model B) the limit condition is anticipated by a marked increase of displacement since the friction angle is reduced to 8°.

Figure 16. Deformed shape (x200) of Model A (a) and Model B (b) for a mobilized friction angle $\phi = 7.6^\circ$ on the basal slip surface.
7.1.3. Depth and shape of the slip surface. As already discussed in Section 5, depth and shape of the slip surface can be unambiguously defined only in the toe portion of the slope. However for the upper part of the slope some reasonable hypotheses can been formulated, taking into account the location of the damaged tunnel section and the slope morphology. Figure 18 shows the set of four different hypotheses (a, b, c and d) which have been considered and implemented in Model A (model with continuous joints). Dip angle and depth of the upper portion of the slip surface vary reasonably between 7–13° and 54–100 m, respectively. As figure 18 shows, the slip surface (c) gives a minimum friction angle very close to the residual value of shear box tests. The slip surfaces a and b, for which the limit friction angle is in the 9-9.5° range, can be considered realistic too.

Figure 17. Calculated displacement vs mobilized friction angle of basal slip surface for Model A (a) and Model B. (b).

Figure 18. Considered slip surface profiles (a) and their limit condition (b).
7.2. Influence of groundwater table rise and reservoir filling

In a first set of analyses, the water level of the reservoir was kept constant at 282 m a.s.l., while the groundwater table inside the slope rises from the average conditions (AGW) to the maximum level recorded throughout the monitoring period (AGW + 6 m, AGW + 10 m and finally AGW + 15 m). The limit friction angle of the slip surface increases from 7.6° to 8.8° (figure 19). Figure 20 shows the displacement contours when the limit friction angle of slip surface is 7.6° for the case with maximum groundwater level (AGW +15 m).

Then the influence of reservoir filling has been considered. In these analyses a horizontal water table is introduced inside the slope till the intersection point with the average groundwater table profile. The effect of reservoir level increase, up to the maximum project level of 330 m a.s.l., has been analyzed by keeping constant the friction angle of the slip surface (7.6°). Figure 19 shows the “rate” of increase in displacement (displacement increment divided by the increase in water level) obtained for different water pool elevation.

The reservoir filling alters pore pressure only in the lower portion of the slope. The main effect consists in buoyancy forces induced in the rock mass.

As it is shown from the displacement of target point N3 in (figure 19) the rise of reservoir level from 305 to 310 m a.s.l. gives the maximum rate of displacement. The upper portion of the slope, N8 point, experiences a slightly higher displacement as compared to the lower zone.

The slope deforms differently in the upper part, between points F and G, where effective stress is less affected by filling. Vertical displacements are directed upwards between G and H and downwards in the upper zone of the ridge. The surcharge load from the reservoir seems to provide also some restraint to the toe of the slope, even when it is mostly submerged.

Figure 19. Average and maximum elevation of groundwater table, maximum level of reservoir considered in DEM modelling (a); horizontal displacement increases calculated for progressive reservoir filling (b); horizontal displacements vs mobilized friction angle curves calculated for different groundwater profiles (c).
8. Conclusions

The analysis of monitoring data has shown that the slope movement exhibits the typical features of a stationary creep with seasonal reactivation due to change in pore pressure and rainfall regime. Three months cumulative excess rainfall gives better correlation with pore pressure increase and peaks in displacement velocity. The limit strength of the slip surface obtained from DEM modelling is close to the residual friction of the clay gouge.

The influence of the slip surface geometry on the mobilized friction angle has been examined by considering different reasonable hypotheses. It was found the limit friction angle can vary as much as 2°. The influence of block size on model response was investigated particularly for the case of persistent joints. A change in block scale affects deformation and displacement magnitude but has negligible influence on the mobilized friction angle. Also, the joint pattern has limited influence on mobilized strength, but the deformation mode is different for persistent and staggered joint systems. The model with staggered joints exhibits less shearing inside the rock mass and more uniform deformation. Both models can be considered meaningful and instrumental for a better understanding of slope behavior.

The influence of groundwater level rise was investigated with respect to the average groundwater table derived from piezometer measurements within the slope. The minimum friction angle required for stability increases from 7.6 to 8.8° when the groundwater table rises from the average conditions to the maximum expected level. Further analyses were focused on the response of the slope to reservoir filling. The increase in reservoir level determines small displacements and negligible changes of the mobilized friction angle. Vertical displacements are directed upwards in the lower part of the slope, directly affected by buoyancy forces, and downwards in the upper part.

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