Advanced numerical modelling of deep-seated landslide effects on the Sparvo Tunnel using a time-dependent elastoviscoplastic soil model

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Advanced numerical modelling of deep-seated landslide effects on the Sparvo Tunnel using a time-dependent elastoviscoplastic soil model

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ABSTRACT: This paper presents an overall assessment of the interaction between a deep-seated landslide and the Sparvo Tunnel, part of the A1 "Variante di Valico" motorway in Italy. The tunnel, excavated using mechanized TBM technology, consists of two parallel tubes with an exceptional external lining diameter of 15.0 m. The structure crosses a slope affected by a large Deep-Seated Gravitational Slope Deformation (DSGSD), characterized by a total mobilized volume of approximately 33 million m^3. Autostrade per l'Italia S.p.A. commissioned Geosolving S.r.l. to assess possible evolution scenarios of the slope–tunnel interaction, with Tecne S.p.A. providing technical coordination and supervision. The study was supported by an extensive investigation and monitoring campaign carried out between 2014 and 2022. A multi-level modelling strategy was implemented, starting from 2D simulations and progressing to a detailed 3D analysis. The numerical models employed a time-dependent, non-linear elastoviscoplastic constitutive law ("SHELVIP"), specifically calibrated for the Palombini Clays formation using both laboratory results and long-term monitoring data. The results confirmed the significant influence of the DSGSD on the tunnel's long-term behaviour and allowed the identification of the most affected tunnel sections where critical stress conditions are close to being critical. The study also demonstrated the effectiveness of advanced time-dependent constitutive models in reproducing the observed deformation trends. The outcomes highlight the need for continuous geotechnical and structural monitoring and for an Early Warning System (EWS), considered essential to ensure tunnel safety and support future risk mitigation strategies.

1 GEOGRAPHICAL SETTING AND INFRASTRUCTURE OVERVIEW

The Sparvo Tunnel, named after the village of Sparvo located above the tunnel alignment, is situated along the A1 motorway, within the "Variante di Valico" section connecting Bologna and Florence (Italy). The tunnel crosses the northern Apennines, a region characterized by complex geological morphology including deep-seated landslides involving the track of the infrastructure.

The Sparvo Tunnel consists of two parallel tubes, northbound and southbound with lengths of 2431 m and 2495 m, respectively, and an exceptional external lining diameter of 15.0 m. Excavation was performed using Earth Pressure Balance (EPB) Tunnel Boring Machines (TBMs). The tunnel lining is made of reinforced precast concrete segments with waterproof gaskets, designed to resist both short and long-term geostatic and hydrodynamic pressures.

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2 GEOLOGICAL CONTEXT AND GEOTECHNICAL MODELLING APPROACH

2.1 *Geological and geotechnical framework of the slope*

The geotechnical characterization of the slope was derived from a critical review of all available data, with particular attention to the previous investigation campaigns performed during the various design phases of the works (2004-2014). These data were subsequently compared with the results of the most recent investigation campaign carried out in 2021, which specifically focused on the Argille a Palombini (APA) formation.

The geological setting includes four main geotechnical units:

- Unit 1: Flysch sequences belonging to the Monghidoro (MOH) and Montevenere (MOV) Formations, not distinctly separated;
- Unit 2: *Argille a Palombini* (APA), representing the main unit interacting with the tunnel;
- Unit 3: Ophiolitic bodies (OFI), clearly distinguishable within the APA sequence;
- Unit 4: *Arenarie di Scabiazza* (SCB) sandstones.

The *Argille a Palombini* formation, the key material involved in the deformation processes, exhibits geomechanical behaviour similar to that of scaly clays. It consists of alternating layers of clay and fissile claystones, predominantly dark grey, interbedded with thin to medium layers of calcilutites, often siliceous and occasionally showing a fine-to-coarse arenitic base. The limestone/claystone ratio is always much lower than unity. From a hydrogeological point of view, data from active piezometers and other monitoring points indicate the presence of a continuous unconfined aquifer. The groundwater table lies, on average, between 2 and 15 m below the ground surface, depending on the measurement location. Seasonal and rainfall-related fluctuations generally range within ±5 m from the mean level.

Figure 1 shows a 3D geological reconstruction of the sliding surface delimiting the Deep Gravitational Slope Deformation (DSGSD) in the investigated area; The extension along the tunnel alignment is approximately 1.5 km longitudinally and 0.6 km transversely, with a sliding surface depth reaching up to 110 m. In Figure 2 the section 2 is also shown together with the reference geological column identifying the main units.

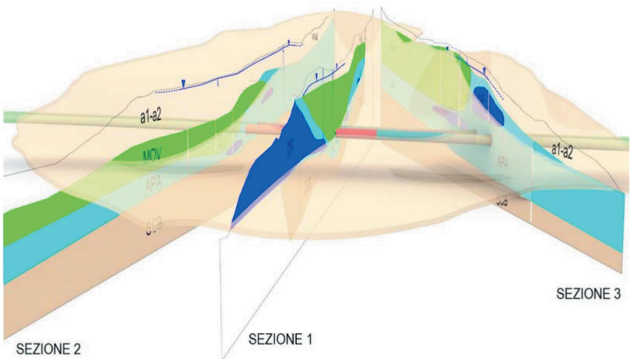


Figure 1. 3D geological reconstruction of DSGSD.

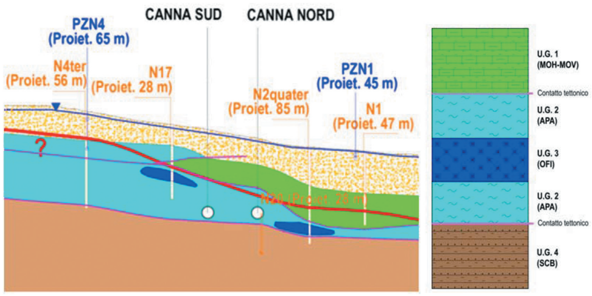


Figure 2. Geological reconstruction of Section 2 and related formation column.

2.2 *The Palombini Clays – From laboratory tests to the geotechnical characterization*

Laboratory investigations performed during the detailed and executive design phases (2004–2007) indicated that strength and deformability parameters revealed a stress-dependent mechanical response. At low effective stress levels, the material typically exhibits dilatant and brittle behaviour with strain-softening features, while at higher effective stresses, most samples showed ductile, contractive behaviour. The swelling potential of the APA is generally low, due to the predominance of non-expansive clay minerals such as illite and chlorite.

Particularly, in the depth range 0-100 m, the elastic modulus varies from 20 to 600 MPa, while in the plastic range it remains below 400 MPa, with a Poisson's ratio of about 0.35. The shear strength parameters show a clear dependence on depth. Within the 0–100 m interval relevant to the slope–tunnel interaction, the peak friction angle varies between 15° and 22°, with residual values ranging from 10° to 15°. The corresponding peak cohesion lies between 10 and 200 kPa, decreasing to residual values between 0 and 150 kPa, consistent with a strain-softening behaviour. The overall interpretation of lab tests leads to the geotechnical characterization in Table 1.

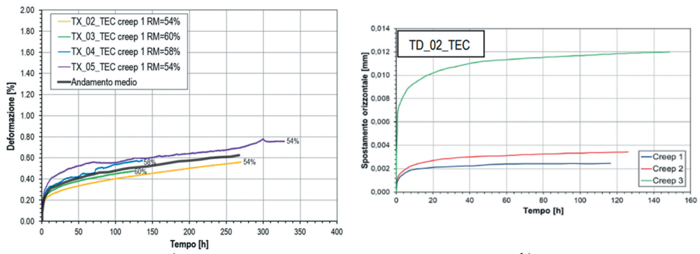
Table 1. Geotechnical characterization of the Palombini Clays.

γ [kN/m^3]	E [MPa]	ν [-]	ϕ' PEAK [°]	c' PEAK [kPa]	ϕ' RES [°]	c' RES [kPa]
22.0	500	0.35	21	160	12	0

Based on the results of all creep and swelling triaxial tests of the "Barla type" performed over the years, the mechanical behaviour of the Palombini Clays is characterized by a marked tendency to exhibit time-dependent deformations. These viscoplastic strains develop both under undrained (short-term) and drained (long-term) conditions. In addition, creep shear tests were carried out with the aim of bringing the specimen to a specific level of vertical load, followed by successive creep stages. Each test was concluded with a final shear phase leading the specimen to failure. Figure 3a shows a comparison between the experimental triaxial creep curves and the average reference curve adopted for the calibration of the viscous parameters used in the numerical modeling, while Figure 3b illustrates a representative time–horizontal displacement experimental curve obtained from the shear creep tests.

2.3 *The adopted constitutive law (SHELVIP model) and calibration process*

The adopted constitutive model, named SHELVIP (Stress Hardening Elastic Viscous Plastic Debernardi & Barla (2009) is a time-dependent, non-linear elastoviscoplastic model specifically formulated to reproduce the complex mechanical behavior of stiff clays and clay shales. The classical elastoplastic component of the model follows a Drucker–Prager yield criterion, while the viscous component introduces a time-dependent term governing the viscoplastic strain rate.



a) b)

Figure 3. Experimental results of a) triaxial creep tests and b) shear creep tests.

The comparison between experimental and numerical creep curves allowed a consistent calibration of the viscous parameters, like the viscoplastic dilatancy parameter (ω), the fluidity parameter (γ) and the shape factor (m). However, the initial numerical results did not accurately match the experimental data. As noted by Debernardi (2008), this was due to calibration based on creep strain rates rather than strains. A slight increase in the parameter γ significantly improved the agreement between measured and simulated creep curves, as shown in Figure 4.

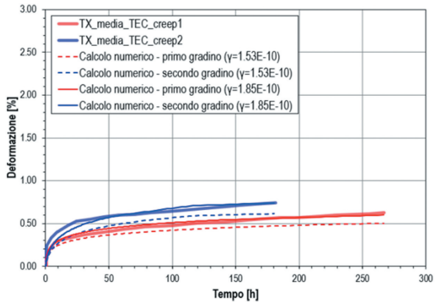


Figure 4. Comparison between the average experimental curves and the computed curves based on the final calibrated viscoplastic parameters.

3 MONITORING SYSTEM, CURRENT AND EVOLVING CONDITIONS OF TUNNEL

The Sparvo Tunnel has been affected by deformation phenomena mainly concentrated in specific sections of both tubes, particularly where the alignment crosses the landslide mass of the DSGSD. These effects have been progressively observed through localized cracking of the inner lining, slight distortions of the tunnel cross-section, and systematic convergence trends recorded by the monitoring system that has been in set in place since the construction phase and progressively enhanced between 2014 and 2022. The system includes both internal sensors (strain gauges, pressure cells, laser convergence measurements) recording the lining response in terms of deformation and stress as well as external instruments (piezometers, inclinometers, and GPS benchmarks) monitoring the evolution of the landslide mass and groundwater levels within the slope.

The dataset collected over the years shows that the most significant deformations are concentrated in the central sector of the tunnel alignment, where the structure intersects the most active portion of the DSGSD. Long-term monitoring confirms a slow but continuous time-dependent deformation, consistent with the viscous behavior of the Palombini Clays and the associated creep effects. In response to these observations, specific structural reinforcement measures were designed and implemented in the most affected tunnel sections. The intervention consisted of an internal steel–concrete shielding system integrated into the segmental lining, combined with the construction of a high-strength concrete invert arch, aimed at increasing the global stiffness and load-bearing capacity of the tunnel lining.

4 2D NUMERICAL MODELS AND PARAMETRIC ANALYSES

4.1 *2D numerical models' setup and simulation of slope-tunnel interaction*

Two-dimensional numerical analyses were carried out to investigate the interaction between the landslide and the Sparvo Tunnel. The modelling was divided into two complementary phases:

1. Back-analysis of the current slope configuration (2022): This step aimed to reproduce the entire sequence of events that led the soil–tunnel system to its present stress–strain state as the natural slope, the excavation of the tunnels and the installation of structural reinforcements. The back-analysis allowed verification of the adopted constitutive laws and calibration of the model parameters against the observed displacement and stress patterns.
2. Prediction analysis from the 2022 state: Starting from the calibrated stress–strain state, predictive simulations were carried out to evaluate the medium-term behaviour of the soil–tunnel system over the next 20 years. This analysis focused on assessing the performance of the structural reinforcements and updating previous forecasts about tunnels.

Three 2D numerical models were implemented, corresponding to the cross-sections illustrated in Figure 1. These models were selected to represent the spatial variability of the slope–tunnel system, with particular emphasis on the section exhibiting the most critical interaction between the landslide and the tunnel. A shear band was introduced within the Palombini Clays, adopting a representative thickness of 15 m, derived from the interpretation of long-term inclinometer measurements and selected as the value that best represents the average behaviour of the DSGSD sliding mechanism (Geosolving, 2024). In the model of the section 1 (Figure 5), which is considered the most representative, this shear band intersects the upper half of the southbound tube and nearly the entire northbound tube. The numerical analyses were performed using FLAC 8.0 (Itasca, 2017). All geotechnical units outside the shear band were represented with an elastoplastic constitutive law, adopting a Mohr–Coulomb failure criterion, while the shear band was modeled with the SHELVIP law. The tunnel lining and structural reinforcements were simulated as linear elastic materials, reflecting the behaviour of the concrete and steel elements.

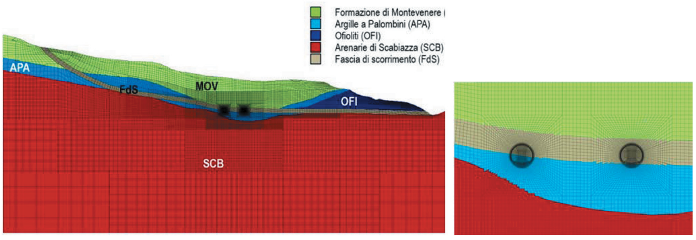


Figure 5. 2D numerical model – Section 1.

For computational efficiency, time-dependent behaviour was applied exclusively to elements within the shear band. Although this simplification neglects minor viscoplastic effects outside the shear band, it allowed the analysis to remain within acceptable computational limits.

The pre-excavation stress state was reproduced by simulating progressive slope erosion to reach the current topography, assuming a geostatic condition with $k_0 = 1.5$. The groundwater table was set as a single shallow aquifer with average depth of 10 m below ground. The time-dependent creep of the slope under gravity was then simulated until a pseudo-stationary condition consistent with satellite interferometric measurements was achieved.

The numerical analyses were articulated in multiple calculation phases reproducing the key construction and post-construction stages of the tunnel system. These included the excavation of the two tunnel tubes, the post-excavation stage, the installation of the structural reinforcements and finally, the long-term evolution up to present day conditions.

4.2 *Model results*

The reliability of the selected elastoviscoplastic parameters was assessed by comparing the computed horizontal displacements along Section 1 with the monitored data collected on the slope. As shown in the Figure 6, the model demonstrates excellent agreement with the observed surface deformation trends, accurately reproducing both the magnitude and the temporal evolution. The comparison refers to the horizontal displacements of 3 surface points selected in the model and the corresponding satellite measurements acquired over the DSGSD sector.

An additional comparison, illustrated in Figure 7, concerns the stress state in the lining of the South tunnel after excavation and the profile of horizontal displacements measured by the installed inclinometers. In the first graph, a good agreement can be observed between the average of the doorstopper measurements performed in 2014 and the mean value along the lining thickness obtained from numerical results. In the cross-sectional view, the curves derived from the analyses at specific time steps (end of North tunnel excavation, 2022, and 2042) are compared with those measured during the corresponding excavation stages. The numerically obtained trend reproduces the observed deformation profile quite well.

Finally, the models provide insight into the expected evolution of slope deformations over the medium-term period (next 20 years). This forecasted behaviour was adopted as a reference for the structural assessment of the current tunnel reinforcement system.

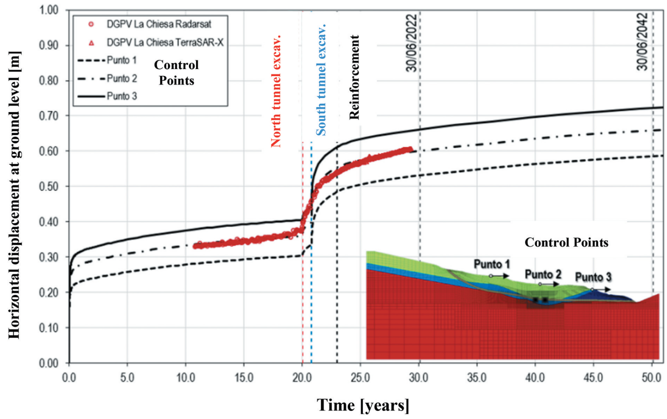
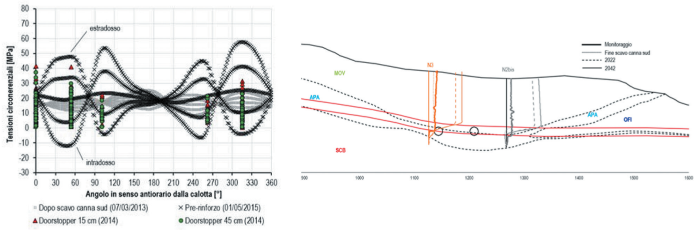


Figure 6. Time history of horizontal displacement at three surface points.



a) b)

Figure 7. a) Comparison of lining stresses and b) profiles of horizontal displacements.

4.3 *Sensitivity and seismic analyses*

The subsequent parametric study focused on the variability and uncertainty of key parameters influencing the long-term behaviour of the tunnel. In particular, the analyses assessed:

1. The influence of hydrogeological conditions and groundwater level variations;
2. The impact of changes in the thickness of the shear zone mobilizing the landslide mass;
3. The effect of mechanical parameters adopted to simulate the behaviour of the shear band.

The results showed that the groundwater level plays a key role in controlling both the slope displacement rate and the cumulative deformation over the medium-term period. When the groundwater level approaches the ground surface, the slope movement accelerates, leading to increased stress within the tunnel lining and reinforcements. However, under the most realistic conditions, corresponding to a +2 m variation from the average monitored groundwater depth (≈10 m below ground surface), the stress changes in the tunnel remain limited. A further rise of the water table close to the surface, instead, could induce overstressing in the reinforced invert of the northern tube, highlighting the need for continuous monitoring of groundwater fluctuations.

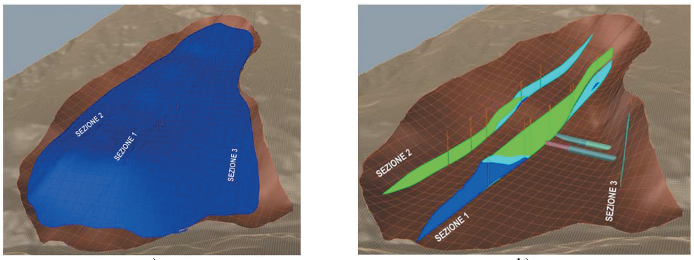
The analyses also revealed that reducing the shear band thickness to approximately 11 m leads to higher stress concentrations along the tunnel lining and a less accurate agreement with in-situ stress measurements. Moreover, this configuration fails to reproduce the long-term deformation behaviour of the landslide, which shows an unrealistic reduction in displacement rates. Finally, reducing the shear strength parameters (φ′ and c′) proportionally increases slope displacement rates and cumulative deformations, potentially leading to bending failure in the reinforced invert.

Seismic analyses were performed according to the Italian standard (NTC, 2018) using three site-specific design accelerograms (PGA = 0.22 g, SLV limit state), selected through magnitude-distance disaggregation and spectral compatibility, and subsequently processed for numerical implementation. The results indicate that the effect of the seismic action on the tunnel is modest compared to pre-existing stresses. In the southbound lining, stress variations remain around ±10 MPa, while in the reinforced invert of the northbound tube reaches ±7 MPa. These stress increments remain below the material strength limits as well as the residual stress increments after the events are negligible, confirming that tunnels remain essentially unaffected by seismic action.

5 3D MODELLING AND COMPARATIVE RESULTS

5.1 *Development and configuration of the 3D model*

A detailed study was carried out to define the spatial geometry of the main geological boundaries of the slope and related piezometric surface in order to develop the 3D numerical model. The shear band mobilizing the DSGSD landslide mass was delineated with improved accuracy. The updated 3D reconstruction was also based on all available inclinometric measurements and piezometer readings from site and updated to 2024. Figure 8 presents the resulting 3D representation of both the piezometric surface and the shear band involved in the mechanism.



a) b)

Figure 8. a) 3D piezometric surface and b) 3D shear band of the landslide mass.

The 3D analyses were performed using the FLAC3D 7.0 (Itasca, 2019), following the same assumptions adopted in the 2D modelling phase, including stress state, material properties and calculation stages. The only refinement concerned the validation of the SHELVIP constitutive law in the 3D domain, which required a dedicated calibration. Specifically, a numerical triaxial creep test was simulated to reproduce the average creep curves obtained experimentally.

5.2 *3D numerical results and comparison with 2D analyses*

As in the 2D models, the reliability of the 3D numerical model and the selected elastoviscoplastic parameters was first assessed by comparing the computed displacements with the monitored ones along the slope. The comparison was complemented by the most recent 2024 measurements of stress within the tunnel lining, obtained from doorstopper readings, allowing for a coupled evaluation of ground and structural responses. Figure 9 shows the total displacements in different stages of the analysis.

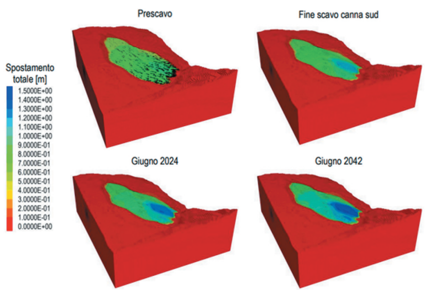


Figure 9. Results of the 3D model in terms of total displacement in different calculation stages.

The 3D analyses confirmed and further refined the main trends observed in the 2D models, highlighting that particular attention should be paid to the transition zone, where the tunnel most significantly interacts with the active shear band of the landslide. Critical stress conditions, being close to threshold values, are mainly concentrated in the invert and in the unreinforced segments adjacent to the strengthened rings. In these areas, longitudinal deformations induced by the landslide–tunnel interaction could lead to possible ring misalignments or excessive stress in the segmental joints.

6 CONCLUSIONS

The multi-scale analyses evaluated the long-term interaction between the Sparvo tunnels and the DSGSD slope. 2D and 3D models, employing the SHELVIP elastoviscoplastic law and validated against monitoring data up to 2024, confirmed overall system stability and the effectiveness of current reinforcements on tunnels. Parametric and seismic analyses indicated minor impacts on tunnel safety. The 3D model highlighted localized stress concentrations, particularly near the invert and in the transition zones. Continued monitoring and completion of the Early Warning System (EWS) are essential to ensure the tunnel's long-term reliability.

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