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Back analysis of the original design predictions by using tunnel face mapping, in-situ measurements and numerical modelling - A1 Lugoj-Deva motorway tunnels

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Abstract

In order to predict the stability of the future excavated tunnels in the project of LOT2 Lugoj-Deva motorway tunnels project, this paper integrates geotechnical information with the results of numerical simulations. The numerical modelling of the tunnel excavation with the primary lining is based on a finite element numerical simulation approach. The objective of the present study goal was to verify the current design and recommend new tunnel support systems that would make excavation safer, more economical, and more efficient. The objectives encompassed the incorporation of additional input parameters update and the re-examination of previously accessible data. The constitutive model of the hardening soil and the input parameter list utilized by the designer during the initial design were both accepted. In addition to the hardening soil model, a classical sophisticated constitutive law, elastic-perfectly plastic model was incorporated. A series of modifications were made to the input material parameters for this reason. Ultimately, it became evident during the excavation of the tunnels that they had traversed a significantly weak rock mass, which was in direct opposition to the predicted overconsolidated soil mass. The following section will provide a synopsis of the advanced numerical modelling of the finally proposed tunnel support systems in both two-dimensional (2D) and three-dimensional (3D) formats. Furthermore, a range of innovative tunnel support types were implemented throughout the excavation process, based on the "design as you go" philosophy and in-situ experiences. The present study provides a comprehensive overview of the tunnel mapping, the geotechnical monitoring system, and the preliminary findings.

Keywords

Hardening soil, Rock mass, NATM, Excavation, Tunnelling.

1 Introduction

The Romanian Road Infrastructure Company (CNAIR), has published a report, stating that a consortium led by the Romanian company Spedition UMB has signed a contract for the construction of two segments of the Lugoj-Deva motorway section in October 2022.

According to a press statement from CNAIR, the project is expected be completed within 45 months (from the commencement of the site work), accompanied by a 10-year warranty. The consortium also includes local companies Tehnostrade, SA&PE Construct, and Bosnia's Euro-Asfalt.

The construction of two road tunnels, with a combined length of 2.13 km, is planned between Margina and Holdea villages. This initiative is aimed at preserving a forest area.The project's financing is predominantly delivered from EU funds which are administered under Romania's national recovery and resilience plan. The Lugoj-Deva section is part of the A1 motorway project, which aims to establish a direct connection between the country's western border andSouthern Transylvania, as well as Bucharest.

Spedition, established in 1997, functions as a division of the UMB Group. Tehnostrade was established in 2004 and is a member of the UMB Group. In 2013, SA&PE Construct was founded in Bacau.

The Euro-Asfalt company was founded in 1996 in Sarajevo, Bosnia & Herzegovina. Euro-Asfalt was responsible for the tunnelling design. The final design has been prepared by the Euro-Asfalt.

The geological and geotechnical ground investigations were completed by GeoStud Srl. a company based in Romania. These investigations were initiated by Irgo Consulting d. o. o. (from Slovenia).

The tunnelling process commenced in February of 2025. At present, the tunnels have been completed to the extent of 65%.

2 Geological background

As the preceding research, namely the Studiu geotechnic, and the subsequent report have thoroughly addressed the geological and geotechnical concerns, it is deemed unnecessary to reiterate these aspects in this particular study.The territory traversedby Lot 2 of the future Lugoj-Deva motorway and specifically the site of the tunnels, exhibits a complex geological composition. It lies entirely on crystalline Carpathian Mountains, a region characterized by its fractured geological structure.

The region is also characterized by the presence of volcanic eruptions and thermal and mineral waters. In the plain, Mesozoic formations are deposited on the crystalline bedrock, over which follow Tortonian-Pliocene sediments. On the surface, the plain is composed of a succession of sands, clays and gravels, all of which are of Quaternary age. The lower plain is characterized by the predominance of recent alluvium. The hills are composed of marls, sands and gravels of Pannonian age. The LipovaPodiș is characterized by the presence of Cretaceous and Neogene eruptive deposits. The Poiana Rusca Mountains are composed of green shale rocks.

From a geomorphological perspective, the site is a characterized by a hilly topography, with elevations ranging from 200 to 400 meters. Morphologically, this area is part of the broader topographical landscape, which includes hilly terrain and valleys (Fig. 1). From a geological perspective, the region in is part of the Pannonian Depression, bordered by the Getic domain with a predominantly mountainous character in its southern and central part, determined by the crystalline and eruptive formations of Retezat, Tarcu and Poiana Rusca mountains.

The elevated areas are bordered in the west, northwest and east parts by intermountain basins with hilly relief, slightly accentuated, corresponding to tertiary sedimentary formations. The majority of the geological formations of the Getic canvas are part of the Southern Carpathians. The northern part of the region encompasses the southern extents of the Apuseni Mountains (Drocea and Metaliferi Mountains). The oldest formations, which date back to the Proterozoic and Paleozoic periods; are widely distributed and are primarily composed of crystalline schists and granitoid rocks. These formations are particularly prominent in the mountainous regions of Retezat, Tarcu, Poiana Rusca and Semenic (Toderas 2022).

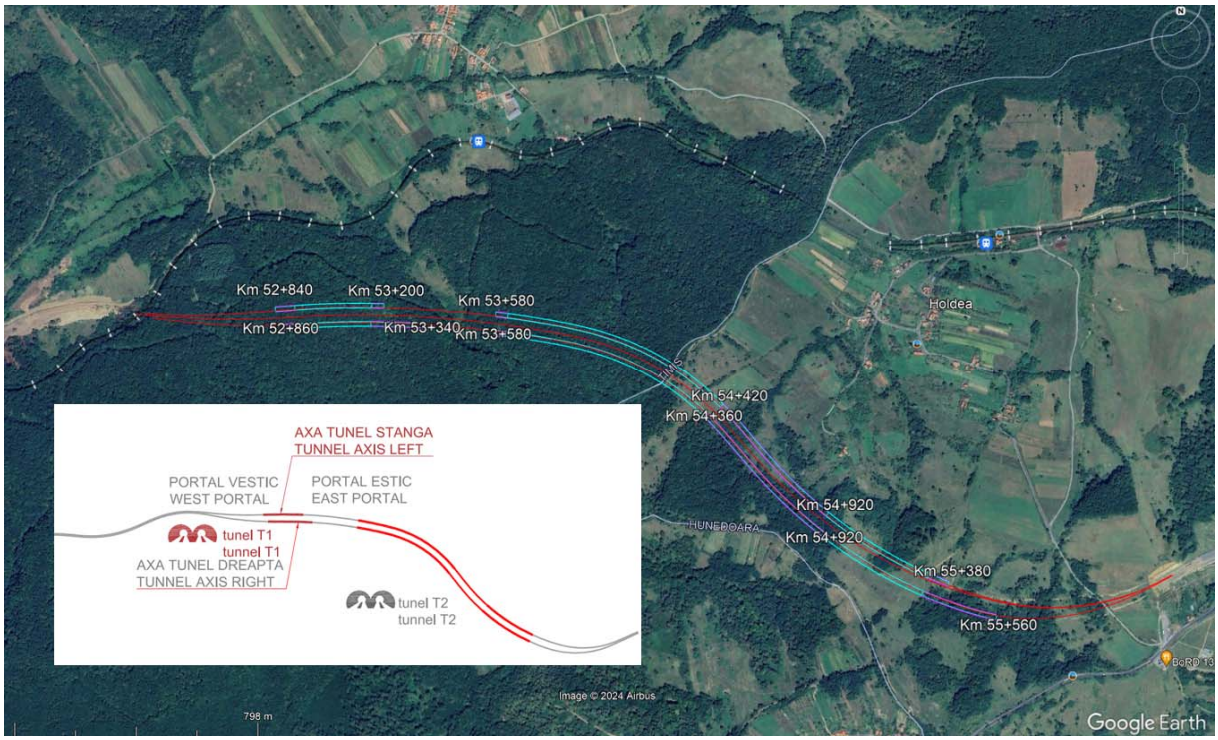


Fig. 1 The track of the tunnels with the topographic characteristics.

The Mesozoic sedimentary deposits in the region are classified into two distinct categories: Rusca Montană - Lunca Cernii basin (Barremian - Danian) and the Metaliferi mountains (Neocomian, Vraconian-Cenomanian and Turonian-Cognac sedimentary deposits). These deposits are further distinguished by the presence of basic Mesozoic eruptive rocks which are associated with the initial alpine magmatism.

The Badenian formations, which are arranged in a transgressive sequence over various pre-Miocene periods, exhibit a substantial, development along the slopes of the Mureș between the localities of Deva - Lăpugi – Coștei. These formations consist of a diverse array of lithologies, including breccias, conglomerates, gravels, sands, marls, clayey marls, coal debris, limestone, gypsum, and pyroclastite.

The Pannonian deposits complete the succession of Neogene sedimentary formations. The two horizons (the lower, which consists of sandy clay blue or greenish-grey clays with irregular intercalations of sands, sometimes coarse, with lenses of gravel and fragments of coal, and the upper - consisting of sands with gravels and rare clay horizons), lie discordantly over the Tortonian formations, or over the crystalline schists.

The Quaternary formations consist of the following series: glacial deposits, proluvial deposits, deluvial-proluvial deposits with reddish clays and alluvial deposits. These deposits are attributed to the Pleistocene and belong to the terraces. The Lower Holocene is attributed to the fluvial deposits of the low terrace, comprising gravel and sand, while the Upper Holocene is attributed to the recent alluvium of meadows, characterized by sand and gravel (Toderas 2022).

From a hydrogeological perspective, the area under investigation is part of the Pannonian and Quaternary sedimentary complex in the alluvial plain of the river Bega. The foundation of the region consists of alternating clays and marls with sand intercalations (Pannonian), which are covered by Quaternary deposits represented by fine and coarse alluvium (gravel, sand, clay, silt). However, minor instances of water infiltration can be anticipated, as indicated by field monitoring during the surface geological investigation.

Based on the construction of the service roads, the tunnel portals and the tunnels experiences that the occurrence of major groundwater inflows is improbable.

The presence of faults was also noted, and the additional investigations were e examined in meticulous detail. The material exhibited signs of fracturing, and in numerous instances, it appeared to demonstrate greater strength than the surrounding rock mass.

Subsequent excavations revealed that the previously identified grey clayey marl layer, in fact, a completely rocky formation.

The tunnels have been excavated in Quaternary (Pleistocene) sediments, with a preponderance of Miocene sediments. The Miocene sediments are characterized by low cohesion, a property that is particularly notable in the tunnel roof, particularly in the top heading area and above the tunnel along its entire length (type FC). Miocene sediments with higher cohesion (the material densely appears as rock mass and the tunnels are crossing continuously this material – type FS) (Deák 2024).

2.1 Integration of the site investigation experiences

In the field of geology, the characterization of rock masses is a critical aspect of geotechnical engineering. This process involves the reduction of measurements taken on intact rock samples to accurately assess the weaker strength characteristics of fractured rock masses. The Hoek-Brown failure criterion and GSI are instrumental in this process, as they provide a framework for evaluating the integrity and strength of fractured rock formations. In the case of soils, it is customary to elevate laboratory values to approximate in-situ undrained conditions. In instances where appropriate in-situ measurement results are available, these should be utilized as a preferred alternative. A comprehensive data analysis reveals that the most sensitive factor in numerical modelling is the Young's modulus (E) (Deák et al., 2024).

In alignment with the designer's perspective, we proposed supplementary solutions. It has been determined that the utilization of physical input parameters, predominantly derived from the laboratory database, renders tunnel excavation impractical. This is due to the fact that the parameters obtained from the laboratory do not accurately reflect the in-situ conditions. The modulus of elasticity, E, is significantly higher and decreases with deformation. However, laboratory tests provide a modulus of E for a high degree of deformation, which is inadequate in this investigated case.

It is imperative to calculate the plasticity parameters under both loading and consolidation conditions, including initial loading conditions, as these cannot be disregarded. In such circumstances, the consolidated undrained triaxial test is the most suitable method for evaluating the soil's structural integrity. In the given context, the triaxial test is the most suitable method for evaluation. For these “real” conditions, the most appropriate test is the consolidated undrained triaxial test. The numerical model provides satisfactory results, provided that the conditions in the field maintain the specified assumptions. These assumptions include the rapid closure of the excavation, which is supported by shotcrete within three hours and the protection of the excavation face at each step.

The implementation of laboratory parameters in the absence of consideration for the initial stress state P_o and undrained conditions renders the execution of numerical calculations unfeasible, as the model exhibits divergence.

For similar considerations and given individual experience, we primarily recommended the implementation of in-situ measurements. We have directed greater attention toward the results obtained from geophysical measurements and in this field seismic measurements provide valuable insight into in-situ parameters (particularly Young's modulus) and rock mass strength.

The calculation of dynamic E is directly possible if the values of V_p, V_s, and density parameters are known.

The values indicate that the excavated material should primarily consist of rock material, particularly a weaker rock mass, in the depth at which the tunnel is located.

In the interest of conservatism, the parametrization this investigation was based on the OYO self-boring pressuremeter measurement results.

Concurrent with the commencement of tunnel construction, the ShapeMetriX3D photorealistic hardware-software system was initiated for use, and a collaborative effort was undertaken to develop a process for geological mapping and the documentation of excavated tunnel faces and side walls. As the documentation process unfolded, it became evident that characteristic fracture systems are present. Geostatistical analysis of these systems is currently underway. The greyish marl rock mass demonstrates high strength in several areas, and the UMB team was adequately prepared for its excavation by special tunnelling instruments from the project's inception.Tunnel mapping is based on

the RMR89 rock mass rating method, and the GSI values are calculated from the RMR results. RMR values generally range from 25 to 35, though maximum values of 35 to 40 have also been documented in previous engineering geological studies (Fig. 2).

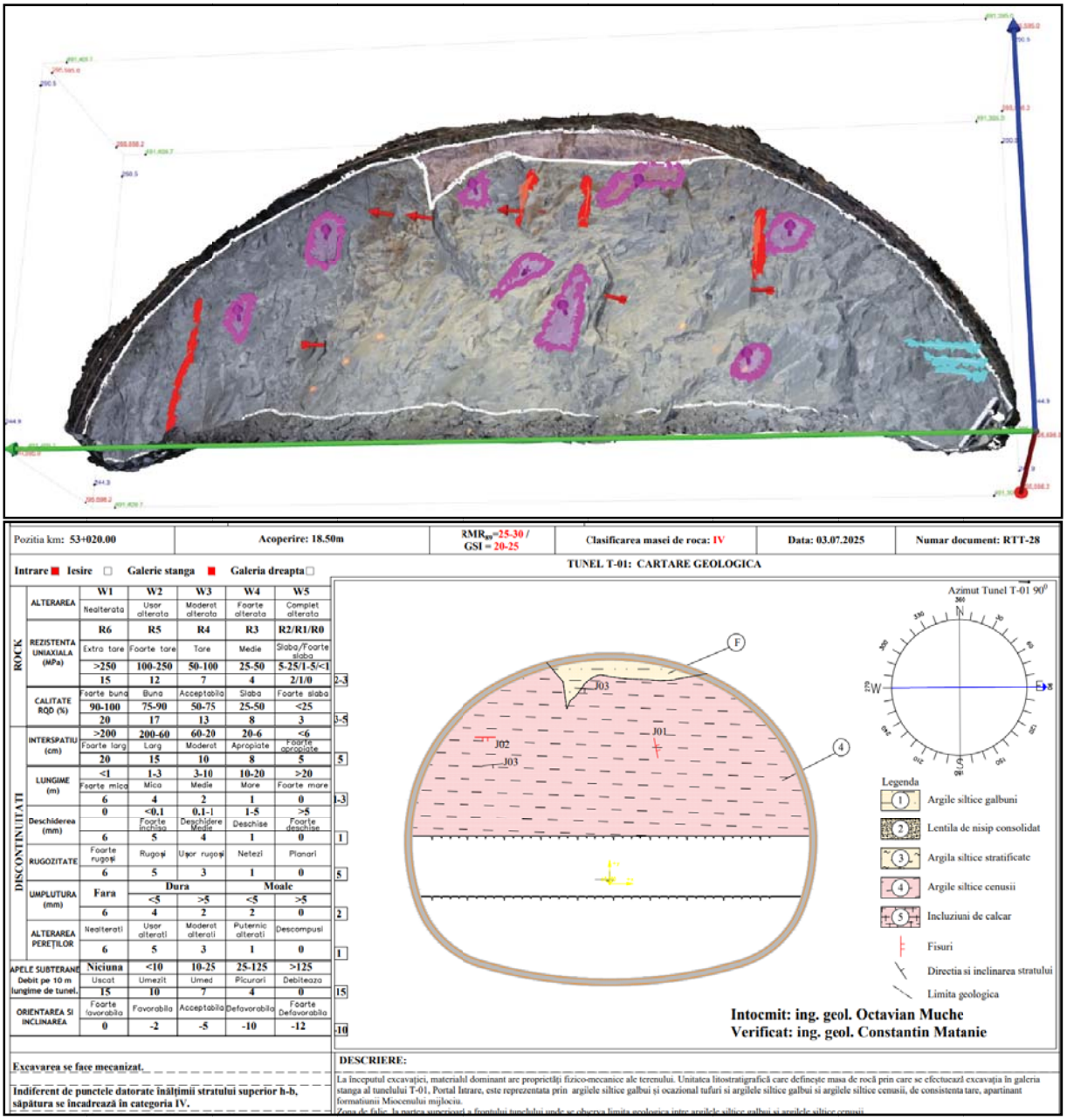


Fig. 2 A detail of the complex tunnel mapping is shown, including a table of RMR values and a photorealistic ShapeMetriX3D model of the excavated tunnel area.

3 Excavation method and tunnel support

In the course of this project the New Austrian Tunnelling Method (NATM) philosophy and approaches were used. The tunnels exhibit minimal overburden, with thicknesses ranging from 2 to 35 meters.

Initially, the designer specified three types of tunnel support systems. Subsequently, based on our recommendations, two additional types of tunnel support were introduced. The TunnelTech team further recommended two basic tunnel support types, in addition to the original three schemes and verified them in detail during the numerical modelling. In the event of more favourable rock mass conditions, an additional five support variations were provided (Deák et al. 2024).

Finally, based on the initial experiences and in-situ measurements obtained during the excavation of the tunnel, six classes of tunnel support were defined. These classes were determined with consideration for the stability of the rock mass, the overburden and the “design as you go” principle.

The primary structural elements that support the main tunnel are as follows: top heading and bench, pipe umbrella, left - right, partial face excavation, with middle wall and temporary top heading invert concrete. The cycle length is 15 m, and the excavation step is 1 m. Additional support is necessary: Pipe umbrella (D=114 mm, L=15m, e=40 cm, overlap 4 m); Pipe roof with spiling rods (D=48.3 mm, L=6m, e=30 cm, overlap 2 m);Systematically bolting (IBO R32, L=9 and 6 m, spacing 1 m, 16 pcs/excavation step); Primary lining (Shotcrete C25/30, v=40 cm, lattice girder every 1 m, wire mesh comprising two layers Q335).

4 Numerical modelling

The designer employed Plaxis 2D software's 2D plain strain finite element modelling (FEM) exclusively. The Phase2, RS2 and RS3 finite element modelling codes were utilized to verify the outcomes, rather than to revise the designed tunnel support solutions.

Two constitutive models were utilized in this study: 1. The designer elected to utilize the Hardening Soil Model (HS) exclusively. The Elastic-Perfectly Plastic (EPP) model is a theoretical framework that aims to predict the behavior of materials under strain. The EPP constitutive model with conservative feedback to lay the underground work for the new tunnel support variations. Additionally, the control phase was resolved by employing the hardening soil model. After traversing several meters of subterranean passage, it became evident that the bedrock manifested as a remarkably fragile rock mass, exhibiting an elastic-perfectly plastic behavior (Deák et al. 2024).

An interaction study was conducted for twin tunnels, with the T1 52+940 scenario serving as the model. This study assumed that the left tunnel is excavated first. The resolution of this issue entailed the implementation of the same methodology employed in the earlier single tunnel models, albeit with the caveat that this was only feasible after the construction of the initial tunnel. The earthquake static load constitutes the final step in the analysis, as the dynamic earthquake loads were previously assessed through a distinct modelling process. It is evident that there is a noticeable interaction even at this distance between the tunnels. However it would be worthwhile to model interaction in three dimensions The most significant displacements occurred during the construction of the second tunnel. However, neither the tunnel environment nor the surface subsidence is at risk as a result (Fig.3).

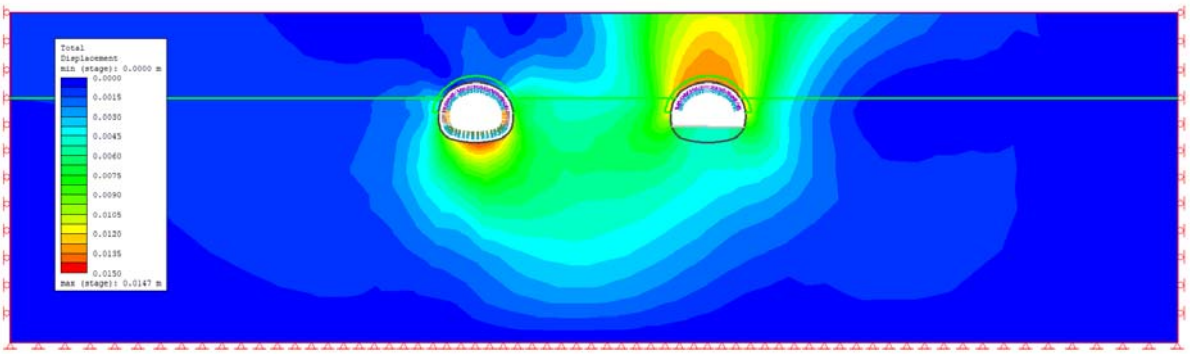


Fig. 3 Numerical modelling of the possible interactions, T1 52+940 – Total displacement – maximum values in the Stage 14 (15 mm). Distance between the two tunnels axis is 50 m.

Finally, during tunnel excavation process, in-situ measurements (i.e., surveyor data and data from the complex geotechnical monitoring section) were obtained, confirming that the dimensions of the pillar specified in the design were appropriate. The current wide axis spacing between the two tunnels was also selected to enable the two tunnel construction teams (Euro-Asfalt and UMB), to construct the tunnels in sections from either direction, independently of each other.

It was evident from the excavation of the initial 25 meters that the EPP constitutive model provided realistic results during the calibration, modelling, and back analysis. This finding was subsequently validated by measurements obtained from the complex geotechnical monitoring section (Fig. 4).

It is therefore recommended that the hardening soil (HS) constitutive model be disregarded, as its application results in displacements that exceed those observed in reality (see Table 1). As was evidenced during the site visit and the integration of the site investigation material during the design phase, tunnels entered a weak rock mass material (with visible joint systems and 5-10 MPa UCS_i) rather than an overconsolidated soil mass.

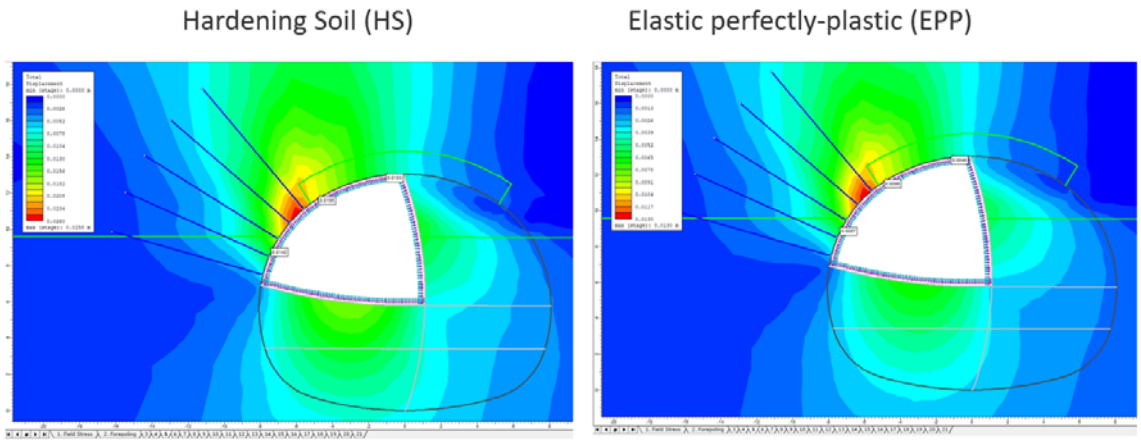


Fig. 4 Numerical modelling results at T1 52+872 section – Total displacements.

Table 1. Surveyor measurements and the numerical modelling results at the T1 52+872 section.

	Surveyor measurement (mm)	**Modell total displacement HS (mm)**	**Modell total displacement EPP (mm)**
Top prism	13	10.3	5
Middle prism	8	19.5	9.9
Lower prism	5	14.5	6.7

5 Geotechnical tunnel monitoring

In 2024, TunnelTech Ltd. and TEHNOSTRADE Srl. formalized a contractual agreement encompassing the acquisition, delivery, installation, commissioning, and maintenance of three sophisticated geotechnical monitoring segments. In addition to the aforementioned sections,we have furnished a comprehensive surveyor background, replete with specialized measuring instruments and proprietary software. The system under consideration integrates not only surveyor data but also all working measuring systems and devices, thereby providing real-time interfaces and data analysis (Fig. 5).

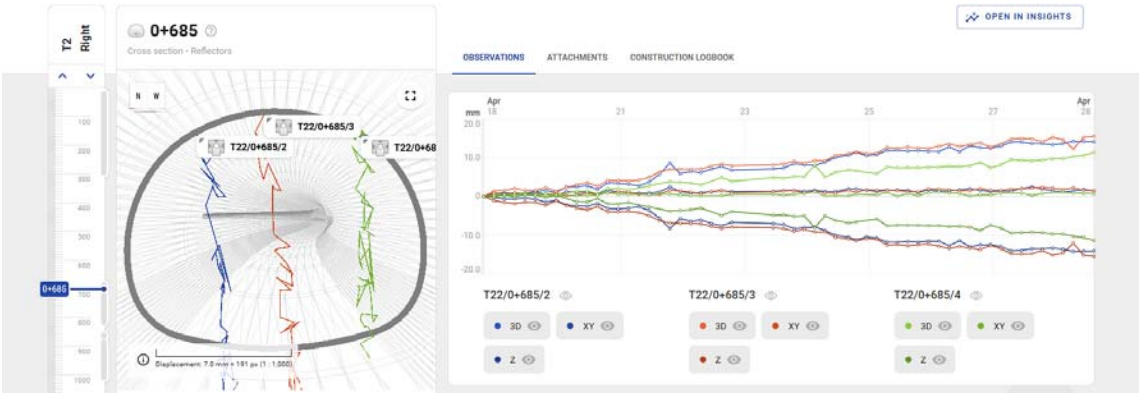


Fig. 5 Real-time visualization of the surveyor data measured by the automatic total station.

A variety of geotechnical monitoring systems are in use, including multipoint borehole extensometers (MPBX) and NATM pressure cells (Deák et al. 2023; Deák et al.2024). At the time of publication, the installation of a measuring section had been completed.

In the case of the 1st geotechnical monitoring section, three MPBX extensometers (three groutable anchors at two extensometers with depth of 3m, 6m, 9m and one extensometer with hydraulic anchors

– borros type, with depths of 3m, 6m and 12 m) and six NATM pressure measuring cells (three radial and three tangential cells) were installed (Fig. 6).

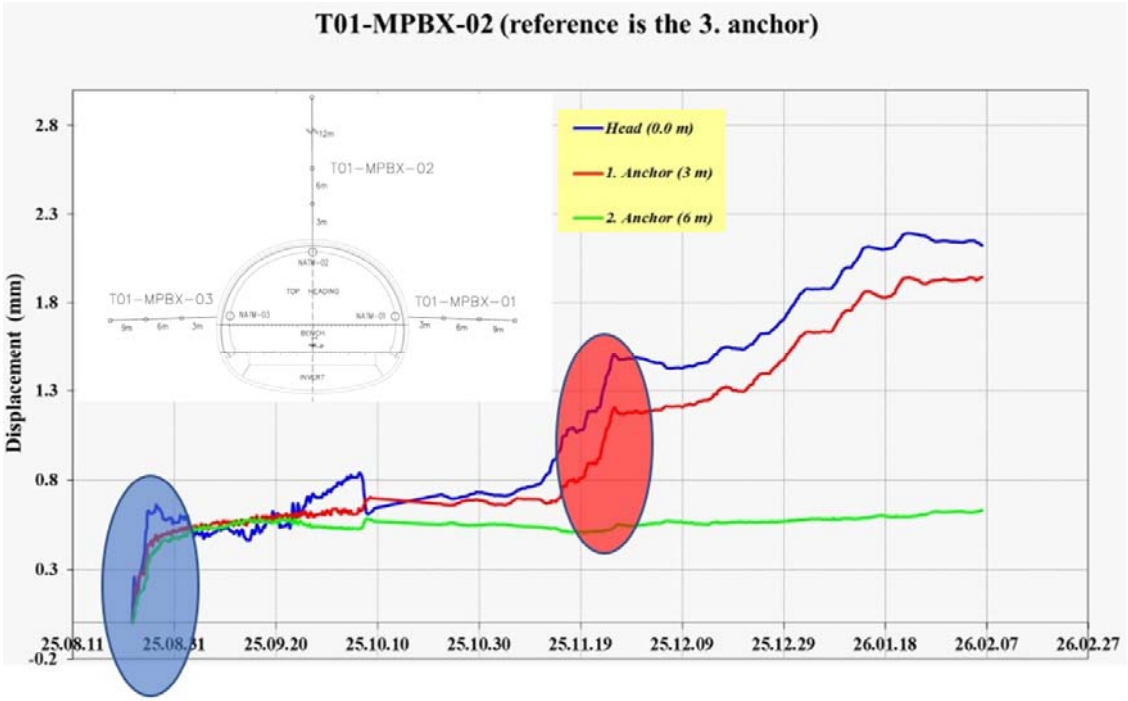


Fig. 6 Measurement results of the T01-MPBX-02 extensometer from the tunnel roof. The displacements from the blue area representing the continuation of the excavation after the installation of the instruments. The increase of displacement steps in the red area started at the moment of breakthrough in the shortest T1 tunnel (the monitoring section was positioned 58 m distance from the breakthrough).

6 Conclusions

Prior to the licensing process, a comprehensive review of the initial draft tunnel design was conducted, accompanied by empirical observations of the excavated rock mass's actual behavior. These observations are concisely outlined in this paper. According to the subsequent analysis and in consideration of the communication between the Romanian and Bosnian tunnelling teams, the designer modified the original design, necessitating the use of six different tunnel support types. This adjustment was made in response to several of our recommendations and observations. A comprehensive numerical modelling analysis has yielded findings that offer significant insights into the behavior of excavated weak rock masses. The study has identified the Young's modulus (E) as the primary sensitive factor in these models. The constitutive model employed for the analysis has demonstrated an exceptional capacity to simulate the behavior of such rock masses, exhibiting an elastic-perfectly plastic response. These observations were corroborated through the integration of in-situ geotechnical and surveyor measurements.

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