Case Study: The Influence of Tunnelling on Slope Stability

Vlachopoulos N. & Vazaios I.
Department of Geological Sciences and Geological Engineering, Queen's University, Kingston, Ontario, Canada

GEOQuébec 2015
Challenges from North to South
Des défis du Nord au Sud

ABSTRACT
Tunneling projects oftentimes involve the design and construction of two adjacent (twin) tunnels with respect to underground infrastructure projects for road, railway or subway systems within mountainous terrain. Currently, there is limited investigation associated with the interaction generated between the excavations of two adjacent tunnels, especially in conjunction with slope stability issues. One of the major drawbacks with regards to such inherently complex, 3D investigations (i.e. a precursor to design) is the increased computational cost required to model the detailed three-dimensional (3D) excavation process utilizing 3D numerical analysis tools. As such, the use of idealized two-dimensional simulation can be employed in order to draw selected preliminary results with respect to the impact of the interaction between twin tunnels constructed within or adjacent to slopes. Despite of being conservative, these slope stability problems can be satisfactorily simulated in two dimensions. In this particular paper, twin tunneling excavation interaction within slopes is investigated utilizing 2D numerical models for tunnels close to the ground surface within weak rockmasses. A case study of the S3 tunnel within the Egnatia Odos European Motorway is also incorporated in order to highlight the very real requirement to take such factors into consideration prior and during the twin tunnel design stage.

RÉSUMÉ
Projets de tunnels impliquent souvent la conception et la design de deux tunnels en parallèle (jumeaux) pour les projets d'infrastructure comme les systèmes routiers souterrains, les ferroviaires ou les métros dans les montagnes. Présentement, il ya des investigations limitée associée avec l'interaction des tunnels en parallèle, et en particulier, en conjonction avec des problèmes de stabilité de la inclinaison. Un des limitations avec ces études de trois dimensions (3D) est le coût associé avec les calculs nécessaire pour modéliser ces systèmes en trois dimensions (3D). En tant que tel, l'utilisation de la simulation en deux-dimensionnelle (2D) peut être utilisé afin d'obtenir des résultats préliminaires pour déterminer l'impact de l'interaction entre les deux tunnels construits à l'intérieur ou adjacentes aux inclinaisons. Avec les limitations, ces problèmes de stabilité de la pente peuvent être simulés de manière satisfaisante en deux dimensions. Dans cet article, l'interaction entre deux tunnels en parallèle adjacent d'inclination et à proximité de la surface est investiguée utilisant des modèles numériques de 2D dans les roches faibles. Une étude du tunnel S3 de l'autoroute Egnatia Odos (européenne) est incorporée avec le but de mettre en évidence les très réels résultats de comportment et les facteurs qu'on a besoin de considérer avant et pendant la phase de conception de deux tunnels en parallèle prés des inclinaisons.

1 INTRODUCTION

There are very real design and construction concerns when designing twin tunnels that traverse mountainous terrain. One of these concerns is the construction of such infrastructure close to mountainous slopes that could be inherently unstable from the onset of construction. As such, much emphasis must be placed on the initial geological engineering site investigations, ground characterization and preliminary analyses.

This paper highlights (through a Case Study and a series of 2D parametric numerical analyses) potential slope instability issues associated with mountainous slopes due in part (but not limited to) to the spatial distribution of geologic formations and geomorphologic features. This is typical in western Greece and the Pindos mountain range (region of Case Study) where competent formations such as limestones are overthrusted by soft, ductile flysch (Fig. 1). Limestone scree typically covers the sheared flysch, allowing water to percolate through. This dampens and further weakens the flysch rock mass, undermining limestone crests and leading to failure (Marinos et al, 2006).

When constructing a motorway through virgin mountainous terrain, it is difficult to predict all of the slope stability risks that may be encountered. For the sites that were identified to have such risks at the mountain scale, many of the slope stability concerns were addressed by moving the tunnel to the other side of the valley where possible. This could not be done in all cases, and therefore difficult tunnels and slopes had to be faced in several sections (Marinos & Hoek, 2001) including the S3 tunnel that is highlighted in this paper.

2 GEOLOGICAL ENVIRONMENT

The overall geology of Greece and that of the Alpine region has traditionally been described in terms of isopic zones and massifs. These zones are groups of widespread rocks that have shared a common history, both in the ancient environments of deposition of sediments (Greece was a shallow, oxygen rich sea during most of the Triassic, Jurassic, Cretaceous and later) and their faulting and folding. The massifs of metamorphic and plutonic rocks are more resistant to folding and faulting than adjacent sediments. .

With the alpine orogeny (the formation of the Alps), limestone was lifted all over Greece and folded. Therefore, approximately two thirds of the area of Greece is covered with limestone as well as many other karst phenomena. Many heterogeneous rock masses, such as flysch, are also abundant. Greece’s geology is still very active as it is located on a converging plate rim between the European and African plate (Higgins et al., 1996). Figure 1 depicts the tectonic zones and massifs of the Aegean region.

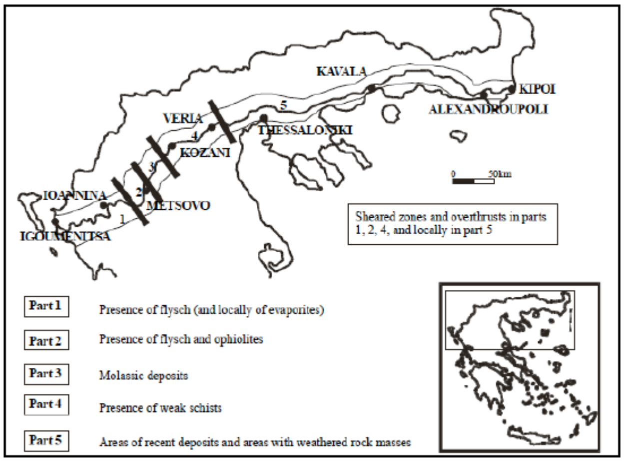

Fig. 1. Alignment of the Egnatia Highway showing spatial setting of the major geological sections along with unfavourable geotechnical characteristics (Marinos & Hoek, 2001).

3 S3 TUNNEL, EGNATIA ODOS

3.1 Egnatia Odos

The Egnatia Odos Highway is a massive construction project that has been completed and commissioned (in 2014) in Northern Greece. The project is an upgrade of an older national highway across the north of Greece. The old alignment follows the ancient Egnatia Road, an 800km route constructed by the Romans for military purposes in the 2nd century B.C.

The new 670 km Egnatia motorway (Fig 1) includes a total of 73 twin road tunnels with an overall combined single carriageway length of 100 km. Sixty (60) of these tunnels are bored or blasted tunnels. The remainder are cut-and-cover (Egnatia Odos AE 2001). As such, over 7% of the overall highway will is carried through tunnels, incurring 30% of the total estimated construction cost. The estimated overall budget of the project was $3.2 billion (US), 60% being funded by the European union and 40% by the Greek government (Silva et al.,2002).

The Egnatia Highway was constructed in order to open up a new, modern and safe motorway connecting the countries of the European Union, the Balkans and the East. The motorway was designed to the specifications of the Trans-European network. It traverses the entire width of Greece, crossing almost perpendicularly the main geotectonic units. Thus, many geotechnically unfavourable characteristics were encountered when deciding how to align the Egnatia Highway.

Also affecting the alignment were various environmentally sensitive areas and locations of high archaeological interest. The great variety of geological / geotechnical situations imposes the need for different approaches in designing the various components of the highway.

3.2 Tunnel Design Methodology

The designers of the Ednatia Odos highway used the principles of NATM and employed a tunnel design methodology to develop excavation methods and support classes for use during construction (Marinos et al, 2006). The steps in this process included:

a. Assess strength and deformability of geomaterials along the tunnel;

b. Calculate deformations and size of plastic zones with no support measures using analytical methods;

c. Preliminary support categories selected using empirical methods such as NATM and RMR;

d. Calculate deformations and size of plastic zones with support measures installed;

e. Identify problem areas (areas of face instability, large deformations, etc…); and,

f. Numerical methods used to confirm and finalize support measures based on failure modes.

Tunnels of the Egnatia Odos highway are twin, two lane tunnels with unidirectional traffic in each tunnel. The final traffic envelope for each tunnel is 8.5 m wide and 5.5 m high, providing adequate space for two lanes, two shoulders and two pedestrian walkways (Fig 2). To accommodate this envelope, an average excavation cross-sectional area of 110 m^2 was required. Over-excavation of the tunnel was needed when large displacements were anticipated. In this case, an excavation area of 110-150 m^2 was required to provide sufficient room to accommodate the final lining (Aggistalis et al, 2004).

Sequence of support installation, sequence of excavation and support of the bench, and the stability of the pillar between tunnels (in the case of very weak rock masses under high overburden) were critical issues during development. Empirical methods were not useful in assessing these due to use of sequential excavation and installation of support measures of face stability.

Numerical models were therefore primarily used for design and validation during construction (Marinos et al, 2006). In an effort to standardize the design and construction of the highway, the Greek government created the Egnatia Odos AE (EOAE) in September 1995.

This organization was responsible for managing the design, construction, maintenance, operation and exploration of the highway. The EOAE compiled design guidelines for all engineering disciplines involved in construction and coordinated with all project and construction managers during development (Aggistalis et al, 2004). As tunneling knowledge in Greece prior to this project was limited, extensive use of external consultants was employed during both the design and construction phases. Drs. Hoek and Marinos were employed as consultants on this project, completing quarterly reviews at each of the tunnel locations that were utilized for this investigation (Aggistalis et al, 2004).

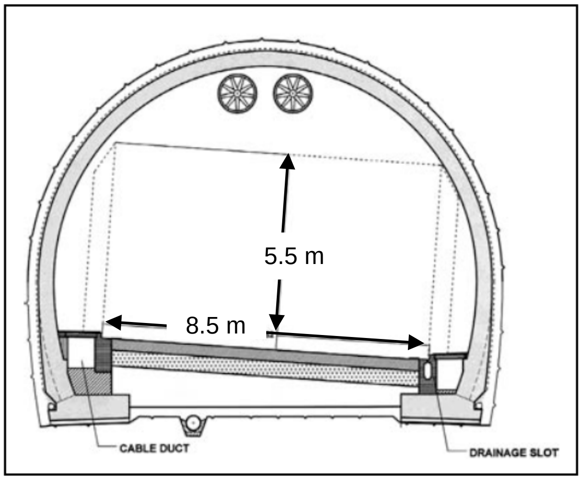

Fig. 2. Eganatia Odos highway tunnel; typical cross-section (Aggistalis et al., 2004)

3.3 S3 Tunnel

The Sigma 3 tunnel is a shallow twin bored tunnel, 230 m long and 12 m wide with a 30 m axial spacing between the two tunnels. It was constructed through a rock slope composed of sheared phyllite thrust on top of crystalline limestone (Fig. 3). While no signs of instability were noted during the geotechnical assessments, significant slope stability issues were encountered during construction (Koronakis et al, 2004).

During construction, a crack measuring approximately 60m in length into the tunnel extended along the final invert near the entrance portal (Fig.3c) (Hoek & Marinos, 2001). Ongoing down slope movements of up to 270 mm were also recorded in the tunnel and inclinometers in the slope (Marinos & Hoek, 2006). These movements and the cracking in the invert suggested a shallow slide had occurred around entrance portal and the tunnels were moving translationally downhill (Hoek & Marinos, 2001).

To prevent propagation of the slide, a pile wall was placed uphill and tied back with long, stressed anchors. Stressed anchors were also placed inside the tunnel while grouting was used to improve the foundations for the anchors as well as strengthen the pillar and the surrounding rock mass (Fig. 4). This increased the safety factor for the shallow slip surface (Marinos & Hoek, 2006).

Follow-up studies showed that an older, deeper landslide surface exists below the original failure surface. This was determined after the discovery of tension cracks and a 1.8 m wide trough structure in the slope above the tunnels. A younger red clay layer dipping into the mountain was also discovered 15 m over the floor of the

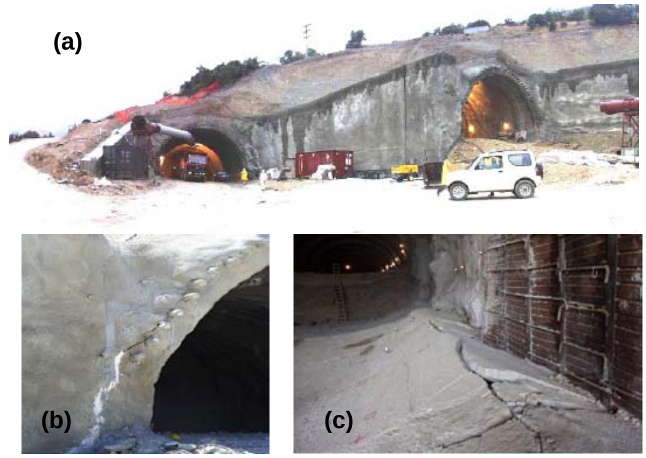

Fig. 3. Photos from S3 Tunnel Construction; (a) Eastern Portals; (b) cracks within shotcrete adjacent to tunnel walls; (c) upheaval of invert.

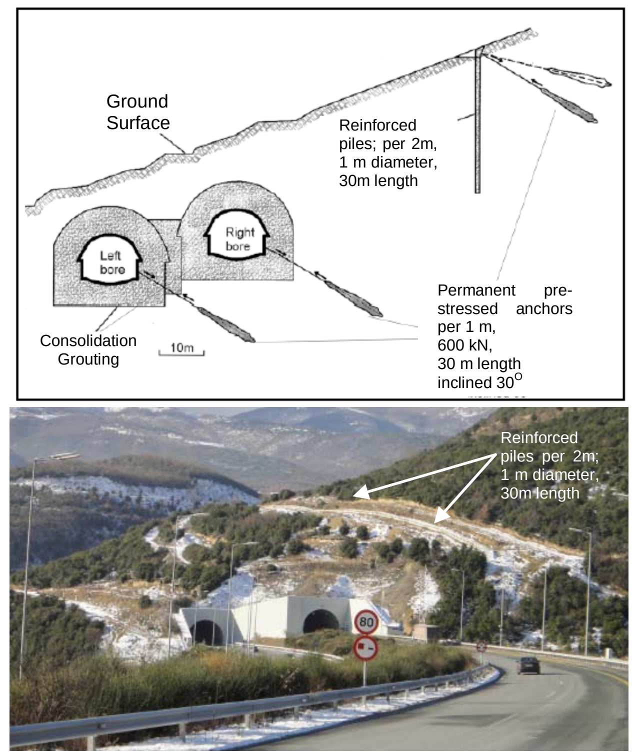

Fig. 4. Support Measures for S3 Tunnel; (a) figure of support measures implemented (modified after Koronakis et al., 2004) and, (b) Photo of reinforced slope/twin tunnel portals.

torrent. This clay layer is overlain by older phyllites, which likely occurred due to an old slide (Hoek & Marinos, 2002). It is believed this surface developed due to the rapid draining of an ancient lake at the mountain toe, resulting in a weakening of the surrounding rock mass. The loss of the water also removed a natural buttress from the slope, further degrading the stability (Langford, 2010). Placing a stabilizing buttress at the base of the ridge was considered a wise, precautionary measure.

Slope stability modeling showed a considerable increase in stability when a buttress was added (Hoek & Marinos, 2002).

3 NUMERICAL MODELLING

In order to highlight the significance and impact of a tunnel passing through a slope as in the case of the S3 Tunnel of the Egnatia Odos project, relevant numerical analyses were conducted. Usually in tunnelling projects, the stability of the openings and their interaction in the case of twin tunnels is of pronounced importance. However, when the topography dictates for a tunnel to pass through a slope, then another crucial parameter that has to be accounted for is the stability of both the underground openings and the stability of the slope as well. In such cases, numerical tools can be proven valuable as they can provide added insight into the complex interaction between the tunnel(s) and its impact on the stability of the slope, especially when the excavation takes place within weak rockmasses.

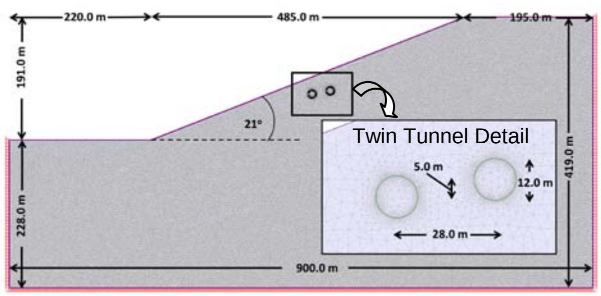

Fig 5. Numerical model configuration for the S3 Tunnel of the Egnatia Odos Project developed in Phase2

The finite element code Phase2 (Rocscience, 2014) was used in order to conduct two-dimensional (2D) numerical simulations. A finite element slope stability analysis in Phase2 can be performed by employing the Shear Strength Reduction (SSR) method, which has advantages over the traditional limit equilibrium methods; for instance, the non-requirement of erroneous assumptions concerning the shape or location of the failure surface. In Figure 5, the major geometrical features of the numerical model are illustrated. The model consists of 48,442 6-node, triangular elements and 97,385 nodes. The rockmass material is assumed to have an elasto-plastic behaviour according to the Mohr-Coulomb failure criterion. Inset within Figure 5, one can observe the details associated with the twin tunnels. The tunnel support, however, is assumed to have an elastic response in these analyses. Regarding the material property assumptions, these are summarized in Table 1.

The simulation was conducted into seven stages in order to account for the 3D effects of the excavation and the de-stressing taking place around the excavation area (i.e. taking into account the pre-convergence effects as the excavation face approaches a specific location). In the first stage, the slope geometry was created and a geostatic stress state was assigned to the rockmass material. In the second stage, the material within the excavation boundary of the left-hand side tunnel was removed and it was substituted by equivalent induced stresses representing the in-situ stress state. In the third stage, the induced stresses were reduced by one-half in order to simulate the de-stressing effect of the excavation getting closer to the cross-section under investigation. In the fourth stage, the induced stresses were removed and the tunnel support was activated. Steps from to two to four were repeated for the right-hand side tunnel as well. Based on the SSR method, at the final stage of the simulation (both tunnels excavated and supported) the Strength Reduction Factor (SRF) was estimated in order to investigate the stability of the slope as the twin tunnels passed through.

Table 1. Geotechnical parameters used for simulation process. [1] Rockmass properties, [2] Rockmass residual properties, [3] Concrete liner (after Koronakis et al. 2004)

	[1]	[2]	[3]
Young's modulus E (GPa)	0.30	0.30	30.0
Poisson's ratio ν	0.30	0.30	0.20
Unit weight γ (kN/m^3)	26.0	26.0	25.0
Friction angle φ (^{0})	27	25	-
Cohesion c (kPa)	30.0	0.0	-
In-situ stress ratio K_o	1.0	1.0	N/A

The aforementioned numerical analysis model was applied for 12 analyses which included: (i) 2 analyses examining the stability of the slope without the excavation of the tunnels and the stability of the tunnel after the tunnels have been excavated at their original location according to the proposed design, (ii) 5 analyses in which both tunnels were translated horizontally in the rockmass and moved away from the slope ground surface, and (iii) 5 analyses in which the tunnels were translated vertically. These groups of analyses were conducted in order to examine the influence of the twin tunnel location relative to the slope and its impact on the stability of the slope.

4 RESULTS AND DISCUSSION

In the previous section, the basic configuration of the numerical model was discussed along with the major assumptions of the simulation and the material properties. The results of the parametric analysis (including the total displacement magnitude and the maximum shear strain) will be discussed in this section.

The first step of the numerical analysis was to determine the stability of the slope prior to the excavation of the tunnels in terms of the factor of safety. As illustrated in Figure 6, the total displacement and maximum shear strain contours exhibit a potential shear failure surface (dashed line) and the SRF for this case was estimated at 1.21, indicating that the slope prior to the tunnel excavation is stable but its factor of safety is relatively low due to the weak rockmass, although the slope angle is relatively small. The slope SRF will serve as the reference case and it will be used to compare the results of the rest of the performed analyses.

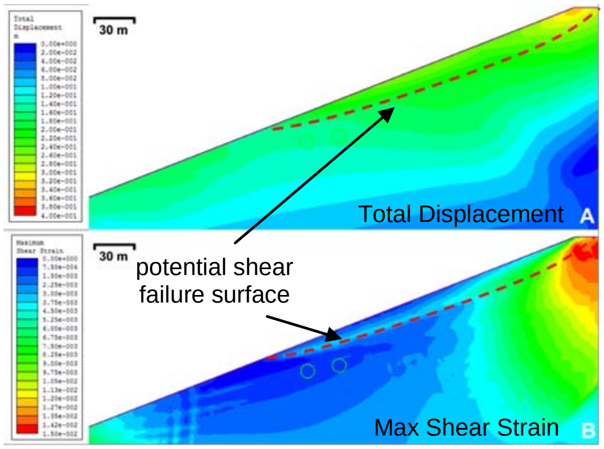

Fig 6. (A) Total displacement and (B) maximum shear strain contours of the slope prior to the excavation of the tunnels

The next step was to simulate the excavation of the tunnels with their location relative to the slope according to the actual S3 tunnel design of the project. In this case, it was determined that the critical SRF was equal to SRF=1.15. The results from this simulation are shown in Figure 7 which depicts the total displacement and maximum shear strain contours. The excavation of the tunnels reduces the factor of safety of the slope due to the loss of shear resistance with the material removal. Additionally, after the excavation of the tunnels, the location of the potential shear failure surface is certainly influenced by their presence, passing through them.

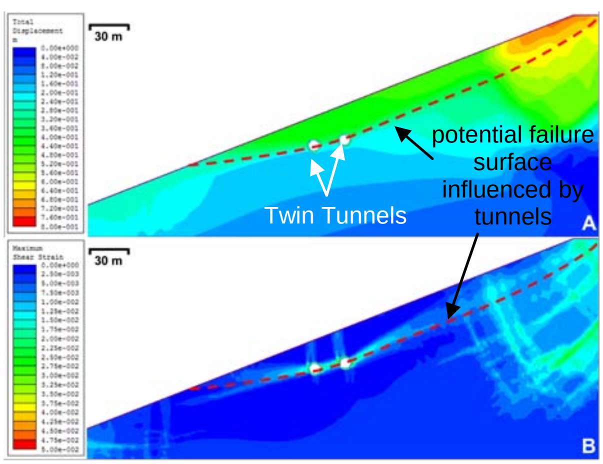

Fig 7. (A) Total displacement and (B) maximum shear strain contours of the slope after the excavation of the tunnels. The tunnel location relative to the slope was determined based on the actual design specifications of the project.

This is a dissimilar possible failure mechanism than seen in Fig 6, making it apparent that the there is, indeed, an influence and interaction between the excavation of the tunnels and the slope stability.

Additional analyses were performed in order to further investigate the impact of the tunnel excavation on the stability of the slope by translating the tunnels horizontally and vertically. In Figures 8 and 9, the total displacement and maximum shear strain contours are illustrated after translating the tunnels horizontally 2.5D and 4D relative to their initial location. Their estimated SRF was 1.17 and 1.18 respectively, exhibiting a slight increase in the factor of safety of the slope, as the tunnels are moved deeper in the rockmass. However, this increase in the factor of safety is not significant (especially in terms of such geotechnical works), as the presence of a weak rockmass combined with the tunnel excavation dominate the response of the slope in this case.

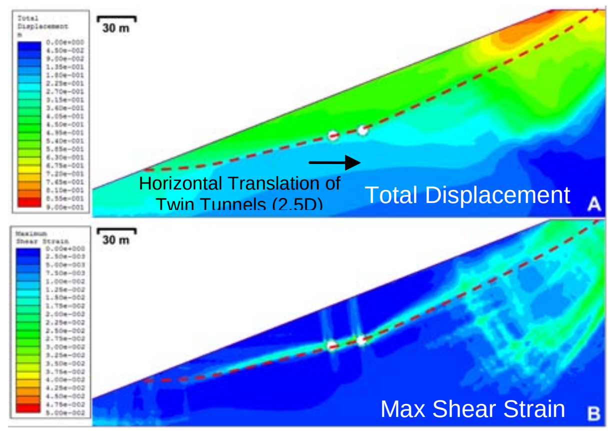

Fig 8. (A) Total displacement and (B) maximum shear strain contours of the slope after the excavation of the tunnels. The tunnels have been translated horizontally a distance of 2.5D relative to their initial location.

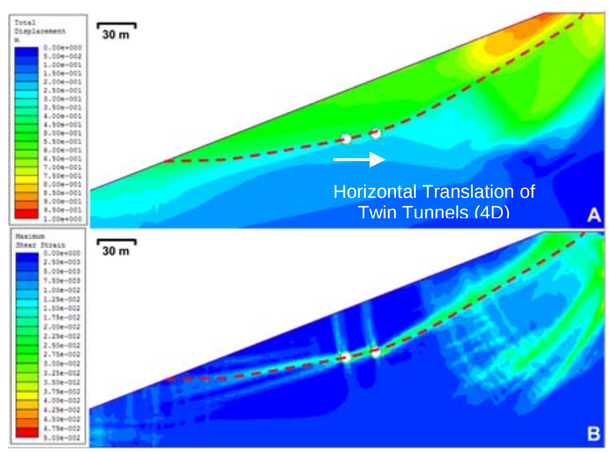

Fig 9. (A) Total displacement and (B) maximum shear strain contours of the slope after the excavation of the tunnels. The tunnels have been translated horizontally a distance of 4D relative to their initial location.

The second group of this parametric study included numerical analyses in which the tunnels were translated vertically relative to their initial position. In Figures 10 and 11, as in the previous cases the total displacement and maximum shear strain contours are illustrated for a vertical translation of 2.5D and 4D respectively. The estimated SRF values are 1.18 and 1.21, from which it can be inferred that as the tunnels are placed deeper vertically and the overburden practically increases, the interaction between the tunnels and the slope decreases. This leads to higher safety factors and for the last case the SRF value is approximately equal to the value for the case of the slope prior to the excavation of the tunnels. Additionally, in Figure 11 it can be observed that there are two distinct potential shear failure surfaces (one highlighted with a dashed line and one with a dotted line) with the one closer to the ground surface resembling the potential failure mechanism of the slope before the tunnels are excavated.

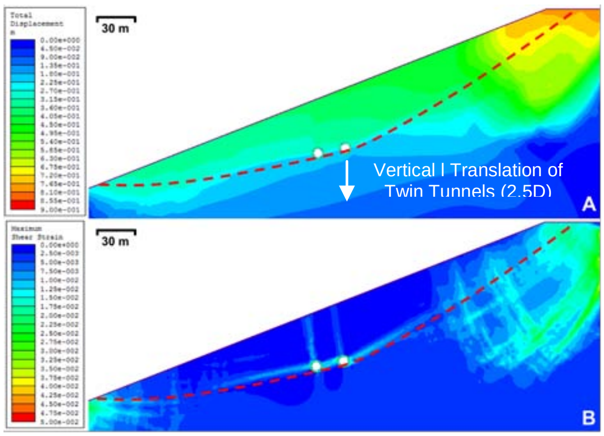

Fig 10. (A) Total displacement and (B) maximum shear strain contours of the slope after the excavation of the tunnels. The tunnels have been translated vertically a distance of 2.5D relative to their initial location.

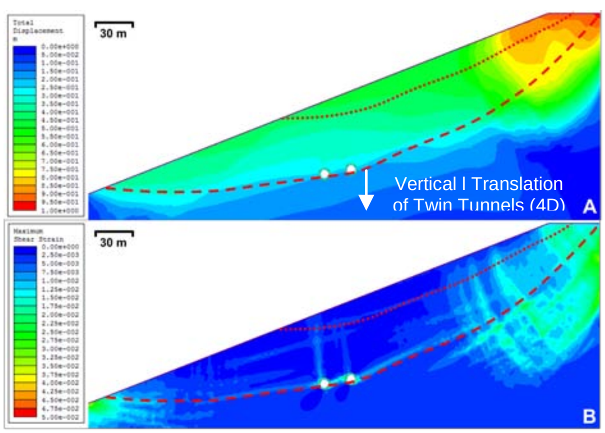

Fig 11. (A) Total displacement and (B) maximum shear strain contours of the slope after the excavation of the tunnels. The tunnels have been translated vertically a distance of 4D relative to their initial location.

Summarizing the results of all the numerical analyses preformed, in Figure 12 the ratio between the SRF of each analysis and the reference SRF which is the SRF of the slope prior to the tunnel excavation is illustrated. From this figure it can be inferred for the horizontal translation of the tunnels, the slope-tunnel interaction is not truly affected and the SRF/SRF_{ref} values are within a range of 0.96 to 0.97. However, when the tunnels are translated vertically, the SRF gradually increases and it becomes approximately the same as the reference SRF as the tunnels go deeper within the rockmass due to the decrease in the slope-tunnel interaction and the arching effect due to the higher overburden.

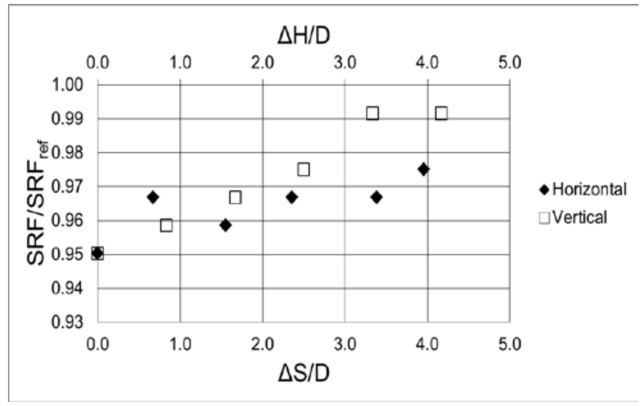

Fig 12. The influence of the horizontal and vertical translation of the tunnels on the SRF values and the safety factor of the slope. D is the diameter of the tunnels, ΔS the relative horizontal translation of the tunnels to the initial tunnel location and ΔH the relative vertical translation.

Despite not having a significant variation in the SRF values by translating the tunnels vertically and horizontally, a definite trend can be observed in both cases. The shallow angle of the slope leads to a small variation of the SRF values for the various scenarios, despite the poor mechanical properties of the rockmass. The analysis herein is hinged on the geometries of the gradual slope and thus the expected variances in SRF were predicted to be minimal. The trend itself is of importance. In the case of a steeper slope, it is anticipated that the tunnels would yield a wider range of SRF values due to the translation of the tunnels. However, even in the case of a slope with a shallow angle, a trend in the SRF/SRF_{ref} ratio can be observed. Thus, it can be inferred that the stability of the slope is a function of the geometrical features of the slope itself combined with the specific placement of the tunnels. Due to the given slope geometry, the horizontal translation of the tunnels had minimal influence on the SRF values, as the tunnel overburden did not change dramatically. However, the vertical translation of the tunnels led to an increased overburden. High overburden results in the redistribution of the stresses due to the tunnel excavation (arching effect) and it does not act as "deadweight" over the tunnels. This is reflected on the estimated SRF/SRF_{ref} values which get higher as the tunnels go deeper due to the minimization of the tunnel influence on the slope.

5 CONCLUSIONS

Developing an appropriate design methodology has significant advantages when constructing tunnels in areas of highly variable rock conditions. To do so, a thorough understanding of the geological, geotechnical and stress conditions must be obtained. Empirical methods supported by analytical and numerical models can provide significant assistance during this process. It is important that the methodology applied is versatile and is able to adapt to different rock conditions and tunnel geometries. In most of the Greek highway tunnels, it was clear that the design methodologies applied were largely successful. Excavation and support methods were suitably adapted for poor rock conditions and the use of face support techniques significantly improved stability and reduced deformations. Yielding support also proved to be a suitable alternative design in areas where large deformations were expected. The design methodology also considered contractor knowledge and ensured the design could be changed easily if different rock conditions were encountered. It is clear that these projects have greatly advanced the tunneling knowledge in Greece.

That being stated, it was demonstrated in this paper that that preliminary 2D analysis is crude enough at the design stage in order to predict slope stability issues associated with tunnelling within slopes. The low factor of safety associated with the initial slope conditions could have been an alarm for the designers. This case study also highlights the requirement for thorough site investigations and ground characterization.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the support provided by Natural Science and Engineering Research Council of Canada (NSERC) and the Canadian Department of National Defence., As well, appreciate the support of Egnatia Odos S.A. and Dr. Marinos.

REFERENCES

Aggistalis, A., Georganopoulos, C., Kazilis, N. & Game, R.C. 2004. Problems and solutions in squeezing ground: cases from Egnatia motorway tunnels, Shotcrete: More Engineering Developments, Taylor & Francis Group.

Chatziangelou, M., Christaras, B., Dimopoulos, G., Soulios, G. & Kilias, A. 2002. Support of unstable wedges along the Platamon railway tunnel under construction in northern Greece, Engineering Geology, 65, 233-243.

Higgins, M.D. and Higgins, R. 1996. A Geological Companion to Greece and the Aegean, Cornell University Press, Ithica, New York, 1996.

Hoek, E. & Marinos, P. 2001. Report by Panel of Experts (Geotechnical/ Tunnelling) on Egnatia Odos Highway Project – Section 5.2: S3 Tunnel, Egnatia Odos A.E., Thessaloniki, Greece.

Hoek, E. & Marinos, P. 2002. Report by Panel of Experts (Geotechnical/ Tunnelling) on Egnatia Odos Highway Project – Section 5.2: S3 Tunnel, Egnatia Odos A.E., Thessaloniki, Greece.

Hoek, E. & Marinos, P. 2005. Report by Panel of Experts (Geotechnical/ Tunnelling) on Egnatia Odos Highway Project – Section 3.5.1: the “Big Cut”, Egnatia Odos A.E., Thessaloniki, Greece.

Hoek, E., Marinos, P., Kazilis, N., Angistalis, G., Rahaniotis, N. & Marinos, V. 2006. Greece’s Egnatia Highway Tunnels. Tunnels & Tunnelling International, September 2006, 32-35.

Hoek, E., Marinos, P. & Marinos, V.P. 2005. Characterisation and engineering properties of tectonically undisturbed but lithologically varied sedimentary rock masses, International Journal of Rock Mechanics & Mining Sciences, 42, 277- 285.

Koronakis, N., Kontothanassis, P., Kazilis, N. & Gikas, N.

2004. Stabilization measures for shallow tunnels with ongoing translational movements due to slope instability, Underground Space for Sustainable Urban Development, Proceedings of the 30th ITA-AITES World Tunnel Congress, May 2004.

Lambropoulos, S. 2005. Tunnelling on the Egnatia Odos motorway: procedures adopted, managerial choices made and lessons learned. Symposium on Design, Construction and Operation of Long Tunnels, Taipei, 2005.

Langford, C. 2010. Greece field course notes, dated 13-19 December 2010, 1-69.

Marinos, P. & Hoek, E. 2001. Appropriate use of geological information in the design and construction of the Egnatia motorway tunnels, report to Egnatia Odos A.E., dated 7-8 December 2001.

Marinos, P. & Hoek, E. 2006. The construction of the Egnatia highway through unstable slopes in northern Greece, proceedings of the XI Conference on Rock Mechanics, Turin, 2006.

Marinos, P., Hoek, E., Kazilis, N., Agistalis, G., Rahaniotis, N. & Marinos, V. 2006. The tunnels of Egnatia Highway: design in a variety of rock masses under difficult geological conditions. 5th Hellenic Conference on Geotechnical and Geoenvironmental Engineering, Xanthi, 2006.

Rocscience, 2014. Phase2, v 9.0. https://www.rocscience.com/rocscience/products/rs2

Silva, R.D., Kazilis, N. and Rawlings, C. 2002. The design and construction of tunnel portals of the Egnatia motorway project, Greece. Proc. NARMS 2002, Toronto: UofT Press. p1491-1498.

Tsiambaos, G., 2010. Rock falls Tempe 2009 committee findings. 6th Hellenic Conference on Geotechnical Engineering, Volos, 2010.