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Slope failure movements controlled by unloading

Glissement de pente contrôlé par déchargement

S. Cavounidis
Edafos Engineering Consultants Ltd, Athens, Greece

M. Bardanis
National Technical University, Athens, Greece & Edafos Engineering Consultants Ltd

ABSTRACT

A cut-and-cover tunnel was designed for a highway in Northwestern Greece. The temporary cut was approximately 30m high and excavated in tectonically disturbed flysch with layers dipping in the opposite direction of the cut. The water table was found very low but rain water easily seeped through the rock mass. According to strength parameters estimated using the Hoek & Brown failure criterion the geometry of the temporary cut provided sufficient factors of safety for overall stability. When the excavation of the cut had proceeded down to the first 10 m, a wedge failure occurred. Support measures were increased at the time and the first surface markers were placed in order to monitor possible large movements. By the time only 4 m of excavation remained, two additional wedge failures and a larger scale instability recorded by the measurements from the markers and surface cracks occurred. Maximum rate of displacement on the markers in the middle of the instability reached 16 mm per day with displacement vectors from markers on the same cross-sections indicating that a deep-seated slope failure surface had formed in the weak rock mass, geometrically helped by the orientation of fully weathered siltstone layers at the toe and discontinuities of tectonic origin at the scarp. Approximate determination of the position of the slip surface allowed back analyses to be performed. Unloading by excavation at the top led to the gradual decrease of the rate of movement to a final stop. The relation between unloading and movement is presented. After this temporary stabilization a new design with flatter slopes and prestressed anchors ensured the stability of the cut.

RÉSUMÉ

Un tunnel réalisé en tranche ouverte a été conçu pour une autoroute au Nord-ouest de la Grèce. La hauteur de la tranchée temporaire était de l'ordre de 30m et l'excavation se formait en flysch tectonisé avec des strates inclinées en direction inverse à la tranchée. La surface de la nappe a été trouvée très basse mais l'eau de la pluie infiltrait facilement dans les massifs rocheux. Selon les paramètres de résistance estimées en utilisant le critère de rupture de Hoek & Brown, la géométrie de la tranchée temporaire accordait un suffisant coefficient de sécurité qui assurait la stabilité globale. Quand l'excavation de la tranchée a atteint les 10 premiers mètres, une rupture en coin c'est produit. A ce point, des mesures supplémentaires de soutènement ont été prises et les premiers repères de mouvement ont été placés à la surface du rocher afin d'observer les grands mouvements éventuels. Lorsque qu'il restait seulement 4 mètres d'excavation, deux coins supplémentaires se sont produit, une instabilité générale a été capturée par les repères et des fissures ont apparut à la surface. La vitesse maximum de déplacement indiquée par les repères au centre de l'instabilité a atteint 16mm par jour et les vecteurs de déplacement des repères aux même sections indiquait la formation d'une profonde surface de rupture. Cette formation dans les massifs rocheux faibles a été facilitée par la géométrie du talus, l'orientation des couches très altérées en grés fin au pied du talus et par les discontinuités d'origine tectonique à l'escarpement. La détermination approximative de la surface de glissement a permit de produire une contre-analyse. Un déchargement par excavation a conduit au décroisement graduel de la vitesse de mouvement et finalement a un état stable. Cet article présente la relation entre le déchargement et le mouvement. Apres cette stabilisation temporaire la tranchée a été redimensionnée avec des pentes moins inclinées et des ancrages precontrainté qui ont rassuré la stabilité.

1 INTRODUCTION

Egnatia Highway is a major highway connecting north-western with north-eastern Greece through very adverse geological conditions, especially in its western part through the Pindos mountain range. In Section 3.1 (M. Peristeri to Anthochori), in the western part of Egnatia Highway, a major landslide was identified on the path of the proposed alignment (D3 landslide area). The landslide is located in flysch with present or old slip surfaces being identified at depths down to 20m. It is a palaeo-landslide extending to approximately 100m perpendicular to the direction of movement and more than 180m parallel to it (Fig. 1). The presence of this landslide on the initially proposed highway alignment and the high costs involved in stabilising it, led to a change in the alignment southwards and uphill of the location of the landslide. Ground morphology and geotechnical properties indicated that a permanent open cut would be unacceptably high for the environmental terms of design for the area and so the construction of a cut-and-cover tunnel was decided (S2 Cut & Cover Tunnel). This was a cut & cover tunnel with one-sided temporary cut, which faced stability problems during its excavation. Herein these stability problems are presented, as well as the way they were dealt with and conclusions from them. A general presentation of the geotechnical problems encountered in the whole of Section 3.1 of Egnatia Highway can be found in Cavounidis et al. (2003).

2 DESCRIPTION OF THE CUT & COVER TUNNEL S2

The Cut & Cover Tunnel S2 is located uphill of the landslide in area D3 of Section 3.1 (Fig. 1). Given that the flysch layer dip direction was opposite to the temporary cut slope direction and strengh parameter estimations on the basis of GSI values led to satisfactory values (c'=50kPa, ϕ'=30°), a relatively steep cut slope was initially designed. The maximum height of the temporary cut was 33 m with the upper 11 m high part having a slope 1 over 1 without any support measures and the lower 22 m high part having a slope of 2 over 1 (vertical over horizontal) with 10cm thick fibre-reinforced shotcrete and 4m bolts as support measures (Fig. 2a). A 5m wide bench separated the two parts of the cut and excavation for the lower part was to proceed

at 5m high stages with placement of temporary support measures in between. The length of the cut was approximately 290m with the central 143m section occupied by the twin concrete tunnels.

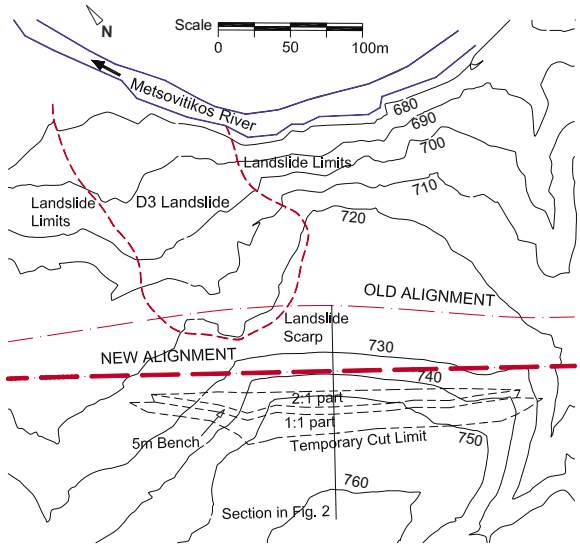

Figure 1. Plan of D3 landslide area, old and new Egnatia alignment and initial cut limits.

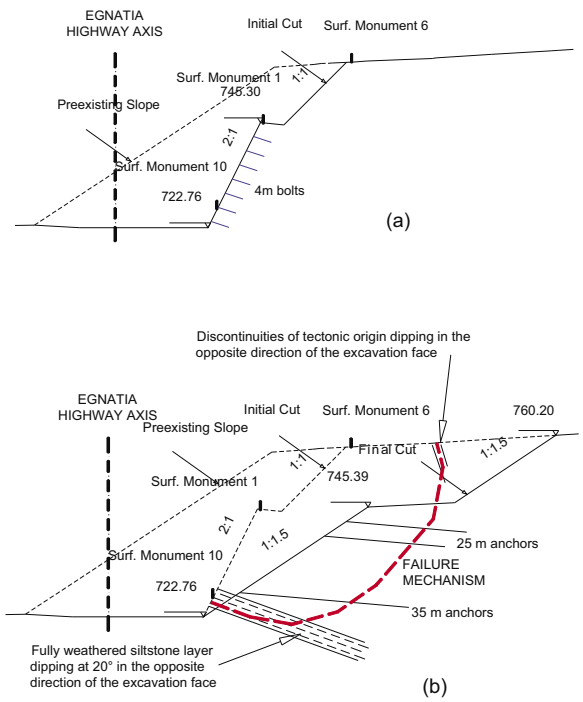

Figure 2. a) Section of the temporary cut with initial slopes, and b) section of the temporary cut with initial and final slopes, fully weathered siltstone band, uphill faults and positions of surface monuments 1, 6 and 10.

3 TEMPORARY CUT EXCAVATION & MOVEMENTS RECORDED

The excavation of the upper part of the slope faced no problems. When the excavation of the cut had proceeded down to the first 10m of the lower 2 over 1 slope part, a small volume wedge failure occurred. Support measures were increased at the time and a dense grid of surface markers was installed in order to monitor possible large movements. By the time only 4 m of excavation remained, two additional wedge failures and a larger scale instability recorded by the measurements from the markers and surface cracks occurred, following heavy rainfall for two days. This rainfall incident (26-27/8/2000) apparently allowed for the creation of temporary perched water tables –no ambient water table was found above the excavation bottom at any stage. Movement rates started climbing rapidly (Fig. 3). The initial rate of approximately 2 mm/day on the markers in the middle of the instability climbed to 16 mm/day with displacement vectors from markers on the same cross-sections indicating that a deep-seated slope failure surface had formed in the very weak rock mass. Surface markers like SM1 and SM6 shown in Fig. 2 recorded movements dipping downwards while surface markers like SM10 in Fig. 2 recorded movements dipping upwards. Markers placed beyond the toe of the slope recorded no movements at all. Azimuth of movements recorded on all markers coincided with the direction of the slope face. Subsequent additional geotechnical investigation indicated that instability was geometrically helped by the orientation of fully weathered siltstone layers at the toe of the cut and discontinuities of tectonic origin at the scarp. Movement vectors on surface markers like SM10 were parallel to the dip direction of the fully weathered siltstone layer but with an opposite direction. A general view of the temporary cut at that stage may be seen in Figure 5.

Approximate determination of the position of the slip surface allowed back analyses to be performed. Assuming the cohesion intercept to be equal to zero given the magnitude of the recorded displacements (maximum value of movements since the beginning of monitoring reached 0.4m), back analyses of the estimated failure surface using the corrected Janbu method of stability analysis yielded residual angle of shearing resistance equal to 25°. Although high, this value is in agreement with values measured in the laboratory by Kalteziotis (1993) on weathered siltstones from Greece.

4 UNLOADING EXCAVATION & STABILIZATION OF MOVEMENTS

Rapid increase of movement rates called for immediate support measures and unloading of the scarp was selected as such while continuing monitoring. Stability analyses on the same sections and failure surfaces that back analyses were performed, determined the size and extents of the unloading excavation. As it can be seen in Figure 4 excavation of approximately 10,000 m^3 proved sufficient to bring movements to a gradual but well defined halt (movement rates dropped even below the values measured before the rainfall of 26-27/8/2000). Given the approximately determined failure surface and the width of the instability, this volume ammounts to approximately 10% of the volume of the sliding mass (estimated at 100,000m^3). Unloading excavations were continued until the movements halted. This relieved immediate danger of collapse of the temporary cut, leaving only the need for final stabilization, given the existence of the failure surface inside the slope.

5 FINAL STABILIZATION MEASURES

Geological and geotechnical investigation that followed movement initiation revealed that despite the favourable dip

direction of flysch layers relative to the slope of the excavation face, the very poor engineering properties of the flysch, especially at the toe of the excavation, were inadequate to ensure the stability of the cut. Inclination of movements on the surface monuments at the toe coincided with the dip of the flysch layers (20°). Additionally, faults of opposite dip direction at the scarp allowed the creation of a rather unusual failure mechanism which, in combination with temporary perched water tables initiated by heavy rainfall, resulted in instability.

In order for the temporary cut's factor of safety to reach an acceptable level for the construction of the twin concrete tunnels, support measures in addition to the unloading excavation included decrease of the slope to 1 over 1.5 (vertical over horizontal) and three rows of prestressed anchors. The upper two had 25m long anchors at 2m spacings and the lower one 35m long anchors at 2m spacing. Such lengths were required in order for the fixed length of the anchors to lie certainly beyond the failure surface. Locally placed horizontal drainage holes (a total of 25 20m holes were drilled) were also placed in order to cope with possible new perched water tables. After the implementation of these additional support measures, construction of the twin concrete tunnels and placement of fill on top of them continued as initially designed. A photograph of the S2 Cut & Cover Tunnel and the D3 Area landslide as stabilized can be seen in Figure 6.

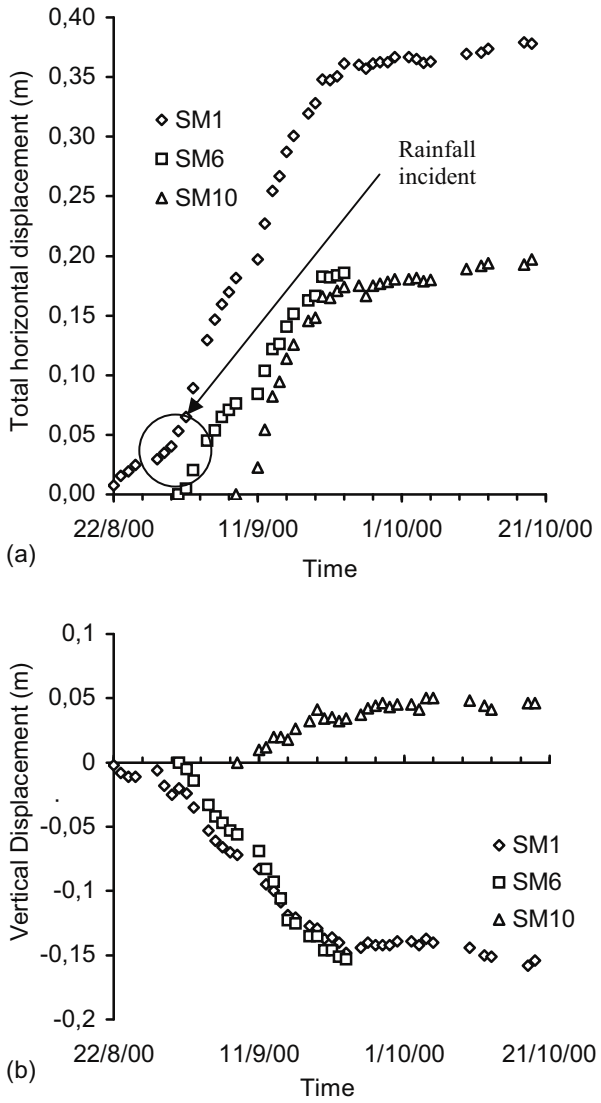

Figure 3. a) Magnitude of movements on the horizontal plane, and b) on the vertical plane (positive outwards and upwards respectively).

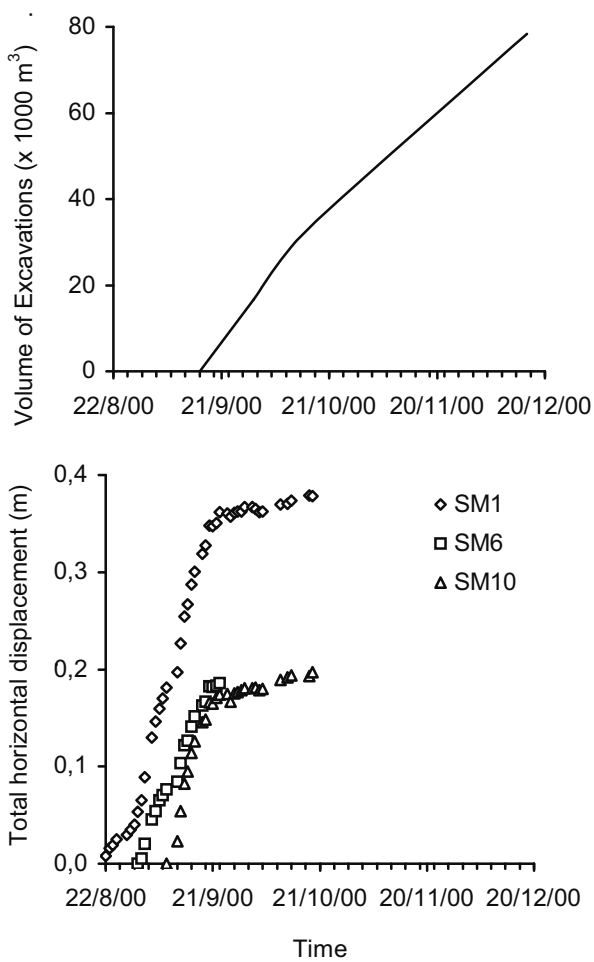

Figure 4. a) Uphill excavation volume, and b) magnitude of movements on the horizontal plane.

6 SUMMARY & CONCLUSIONS

A temporary cut of a cut & cover tunnel was excavated in tectonically disturbed flysch. Favourable dip direction of flysch layers relative to the direction of the excavation face and sufficiently high strength parameter values estimated from GSI values determined slopes in the initial design. Poor engineering properties of fully weathered siltstone layers located at the bottom of the excavation along with discontinuities of tectonic origin at the scarp and temporary perched water tables due to heavy rainfall favoured a deep-seated failure mechanism. Monitoring by surface markers allowed timely, low cost intervention by means of unloading excavations. An excavated volume of approximately 10% of the estimated sliding mass volume brought movements to a halt relieving immediate danger of catastrophic failure and allowed completion of the works.

Dip direction of soil layers opposite to the direction of an excavation face is generaly acknowledged as favourable conditions for excavation. The slope failure of the temporary cut of the S2 Cut & Cover Tunnel is an example of the possible exceptions to this general rule. Layers of very poor engineering properties at the toe of the slope, especially in combination with favourable or even unfavourable –as in the S2 tunnel case- dip direction discontinuities at the scarp can lead to the initiation of, hard to anticipate, failure mechanisms. Monitoring and careful study of movements recorded even with low cost surface markers can be a safe tool allowing timely intervention if instability is initiated. In the case of the S2 Tunnel the main stabilization measure selected was low cost unloading

Figure 5. General view of the temporary cut face with local failures on the left. The sliding mass extended from the left limit of the local failure to the right limit of the shotcrete-covered area.

Figure 6. General view of the S2 Cut & Cover Tunnel and the D3 Area Landslide downhill of it as constructed.

excavations. Approximately 10,000m^3 of material excavated were needed in order to stabilize a mass of approximately 100,000m^3, an excavation volume amounting to approximately 10% of the sliding mass volume.

Furthermore, engineering behaviour of so disturbed "rock" masses as in the case of the heavily tectonically disturbed flysch of the temporary cut at S2 Cut & Cover tunnel is more likely to be governed by the soil layer with the poorest properties. GSI-based strength predictions –even in up-to-date formulations for very disturbed flysch- favoured in engineering geology may correctly predict average rock-mass parameters. Yet, if there is continuity of soil layers and discontinuities with poor properties, these average strength parameters will not be the governing ones. The governing parameters will actually lie closer to those measured by common soil mechanics laboratory tests on the poorest fully weathered layers. Given this, the key to careful design is the early identification of possible continuity of these layers with poor properties. This continuity provided, kinematic feasibility is the next question to be answered. The S2 Cut & Cover Tunnel temporary cut instability is a spectacular example of a kinematicaly feasible, yet geometrically "unfavourable" failure mechanism, created by the coincidential continuity of layers with poor properties and discontinuities. Geotechnical investigations to gather the data needed to indicate the actual existence of such a failure mechanism and the need to design against it remains a challenge for geotechnical practicioners.

ACKNOWLEDGEMENTS

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