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Squeezing rock response to NATM tunnelling: A case study

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ABSTRACT: The squeezing conditions encountered during the Tymfristos road tunnel penetration caused considerable cost overrun along with significant delays in the construction schedule. The project has been completed into three successive contracts. The experiences gained from the extensive convergence monitoring performed in the last contract constitutes the focus of this work. The interpretation of the convergence time histories via close field observation during construction provide insight into the NATM tunnelling concepts in similar ground conditions.

1 INTRODUCTION

The Tymfristos road tunnel construction, 1365 m long, demanded driving through the flysch formation of the Pindos geotectonic unit. This formation at the tunnel site, comprise mainly of claystone and slickensided argillaceous schist, appears intensely folded and tectonized. The maximum height of the overburden is 153 m and the presence of groundwater is restricted to dripping water. From the assessment of the stress field at the depth of the tunnel, in relation to the poor strength-deformation properties of the rock mass (Q values 0.01 to 0.001) (Agistalis 1998), it is inferred that a highly squeezing potential should be accounted of.

The tunnel was designed as a twin sprayed concrete primary lining and cast in place, reinforced, inner concrete lining. The mining of the tunnel commenced in 1992 and was completed in late 1998, worked intermittently, in three different contracts. The excavation took place in either two (top heading and bench) or three stages (top heading, bench and invert). The excavation operation proceeded, according to the NATM, from both portals. The excavated cross section (120 m^2) was subdivided into top-heading (H = 6.0 m), bench (H = 2.20 m) and invert (H = 3.0 m) drifts. The primary support consisted of a 0.40 m thick shotcrete shell, reinforced with two layers of T188 wire mesh, twin IPN 180 steel ribs, placed at 0.80 m centres, as well as 20 fully grouted dowels, 6 ÷ 9 m in length. However, no limitation had been set, concerning the ring closure distance. Tunnel breakthrough, at the height of the top heading section, occurred in Sept.1995, during the second contract.

The stress redistribution in the surrounding rock mass resulted in serious overstressing and subsequent yielding of the primary support, in the second construction period. Various lengths experienced excessive time dependent deformations, which led to substantial closure of the opening.

The final contract (Oct'97-Oct'98), therefore, was associated with the re-excavation of the deformed tunnel top heading sections as well as with the completion of the excavation-primary support of the opening (revised excavated area 130 m^2). The total length of the tunnelling of the last contract was 531 m (ch. 0+809.43 to ch.1+311.96). The experiences gained during the previous construction time periods guided the reviewed design of the final attempt. Top-heading re-excavation (H = 6.5 m), corresponding to an area of approximately 10 m^2, has been performed as a widening of the previously converged cavity. The length of advance of the particular drift was limited to a maximum of 1 m, whereas ring closure of the primary support shell was specified to 2 m, at a maximum. Three different support classes were specified for the changing ground conditions. The primary support consisted of a 0.30-0.40 m thick shotcrete shell, reinforced with two layers of T131 welded wire mesh and Pantex type lattice girders 70/30/D30, complemented by systematic bolting (13/14 fully grouted dowels of 6 to 9 m in length). An extensive convergence monitoring program was also adopted, providing monitoring sections every 15 m.

Against all previous experience, even this final operation proved problematic. Large displacements arose soon after first stage re-excavation, that caused non-uniform deformation and consequent overstressing of the primary support. The accumulation of the amount of deformation give rise to severe instability phenomena, evidenced, among others, by failures of the shotcrete arch, and buckling of the girders. Ring closure following the commencing of such phenomena was to no avail. The present work constitutes an attempt to the identification of tunnel behaviour through monitoring results.

2 INTERPRETATION OF MEASUREMENTS

The mining of the last 531 m (ch. 0+809.43 to ch. 1+311.96) of the tunnel demanded crossing a rock mass of poor to markedly poor strength and deformation characteristics. Works proceeded as two opposite drivings, starting from both edges of the length, under consideration.

The monitoring system, established for the purposes of the contract in question, was limited to convergence measurements, drawn from a dense sequence of measuring stations (typically every 15 m). The geodetic technique adopted allows for the determination of coordinates of special targets that are fixed to the tunnel circumference. Each station consists of 5 measuring points, 3 of which have been installed at the top heading section (one at the crown and two at the shoulders, close to the tunnel springline, denoted as 2, 1 and 3, respectively). Moreover, it has been specified, in the contract documents, that: the commencement of monitoring, in the due sections, must take place just after the shotcrete shell placing, for the top heading. This demand has been set, because of the previous contract experiences. According to these (Tsatsanifos 1995), abrupt development of field stress gradients in the supported mass is possible to cause undesirable deformations, shortly upon completion of the shotcrete shell. Thereby, the practice, followed, required targets of high accuracy, which were installed, significantly close to the face of reconstruction.

The interpretation of data of the tunnel deformation time histories proved a laborous task. The difficulties are related to the primary support failures, causing stiffness degradation of the shotcrete shell structure. Hence, it was necessary to identify those parts in the strain vs. time plots, that correspond to the uncracked primary support response. The transition strain value of each section has been determined by taking account of the in-situ observations.

The key parameter employed is the inward radial strain in the principal tunnel directions. These directions, for each measuring section, coincide with the crown vertical strain (e_v) and the convergence (e_d) of the diameter at tunnel springline.

The time histories of both strain parameters are given in Figure 1, where the positive sign refers to the inward radial strain for point 2 and the convergence of the 1-3 length. The capital letters S and K are employed to denote the location of the measuring stations installed during the mining from ch. 1+311.96 and ch. 0+809.43, correspondingly. At the same plots the line equal to 2% strain is also presented.

This strain level has been adopted by Hoek (1999) as the one corresponding to a normalized rock mass strength ratio (uniaxial rock mass strength/in situ stress) equal to 1/3. It is noted that weak rock conditions should be assumed for ratios lower than 0.30. Tunnelling under such conditions encompasses the potential of severe stability problems, unless adequate support measures are provided. To this direction, Sakurai (1983) has suggested a critical strain level that serves as a fictive boundary, separating the inherently "stable" tunnels from those tunnels that require special support measures. In this work, the critical strain level (e_{pc}) turns out dependent upon the uniaxial compressive strength of the rock mass (σ_{cm}), according to the equation:

$$e_{pc} = 1.073 \cdot \sigma_{cm}^{-0.316} \qquad (1)$$

From Figure 1, it is obvious that in the majority of the measuring sections, both S and K, the tunnel suffered from vertical rather than from horizontal displacements. Furthermore, S stations correspond to that tunnel length where serious instabilities appeared during mining. In contrast to this, K stations exhibited significantly lower strain development, whereas no signs of overstressing appeared during excavation and primary support. Based on these plots as well as on field observations, a gross identification of tunnel behaviour can be achieved.

A length of, approximately, 200 m (ch. 1+056.71 to ch. 1+248.67) represents excavation via shear zones consisting of a soil like material. The transition into the shear zone is estimated (see Fig.1) between stations S5 (ch.1+259.90) and S6 (ch. 1+248.67). Extensive geological face mapping showed a foliated / laminated material with poor surface conditions (Hoek et al. 1998). More specifically, GSI value of about 20 and uniaxial compressive strengths of intact material ranging from 5 MPa to 10 MPa are representative of the zone under consideration (Tsatsanifos 1995). Based on these values, the uniaxial strength of the rock mass has been derived as: σ_{cm} = 0.30 MPa, whereas the Mohr-Coulomb shear strength parameters were calculated as c = 0.11 MPa and $\varphi = 16^o$, respectively. The procedure followed for the determination of the above strength characteristics is based upon a statistical correlation between the non-linear, generalized Hoek–Brown criterion and the Mohr-Coulomb failure law, and is described, analytically by Hoek &

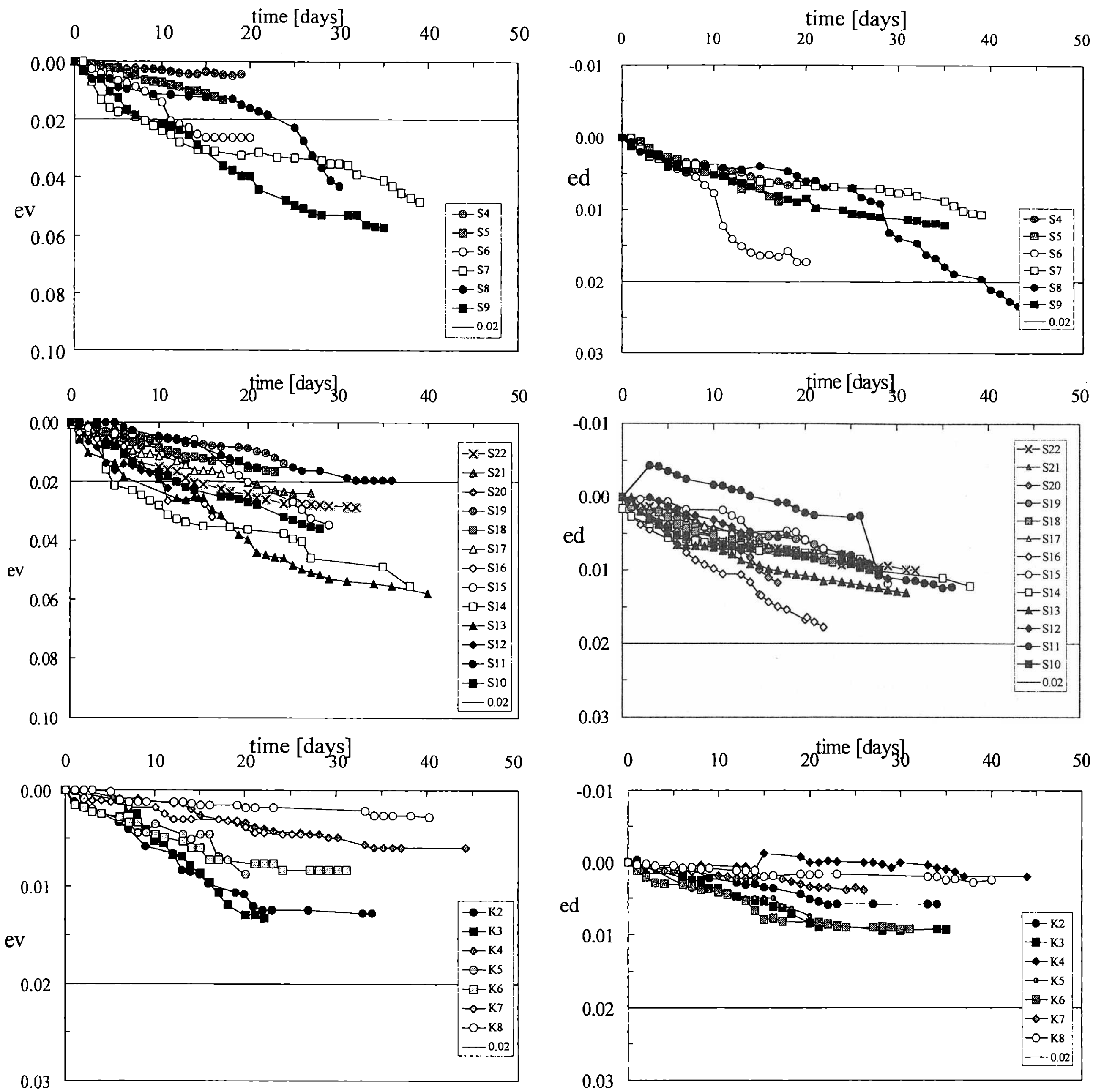

Figure 1. The variation of e_v and e_d radial strain parameters versus time for S and K measuring sections.

Brown (1998). The aforementioned values prove similar to the set of parameters ($c = 0.10$ MPa, $\varphi = 16^{o}$) assumed for tunnel deformation back analyses (Tsatsanifos 1995).

The squeezing potential of the medium is controlled by the ratio of the uniaxial compressive strength (σ_{cm}) of the rock mass to the maximum in situ stress level. For a crude approximation of the latter term, the weight of overburden (γ·h) is employed. For the case considered, the ratio becomes equal to 0.1, which is a figure indicating a highly squeezing material (Tsatsanifos 1995). In addition, a critical strain e_{pc} = 1.57% is obtained from the empirical equation (1), by using $\sigma_{cm} = 0.30$ MPa.

The results of the characteristic line computations, assuming $\gamma = 20$ kN/m^3, are displayed in Figure 2. In this graph both the tunnel convergence and the thickness of the plastic zone are associated with the decreasing support pressures (p_i) from the in situ stress level (p_o) to zero. It can be seen that the convergence for the unsupported 13 m span cavity is approximately 38%, whereas the plastic zone is expected very large. It is also estimated that the non-linear response of the surrounding mass begins for 30% deconfinement ($1-p_i/p_o$). The plastic zone respective to strain level 1% extends, about, 6.0 m far from the tunnel profile. Moreover, 2% convergence and a considerable amount of yielding around the

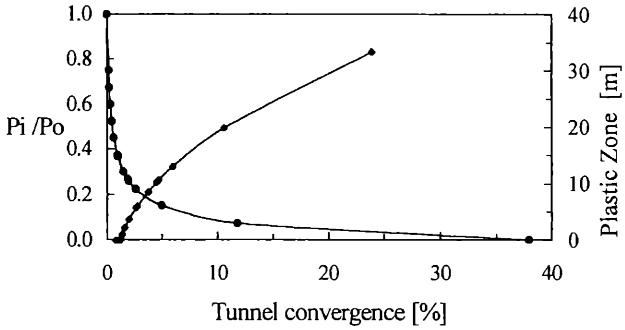

Figure 2. Characteristic line computation results for the material of the shear zone.

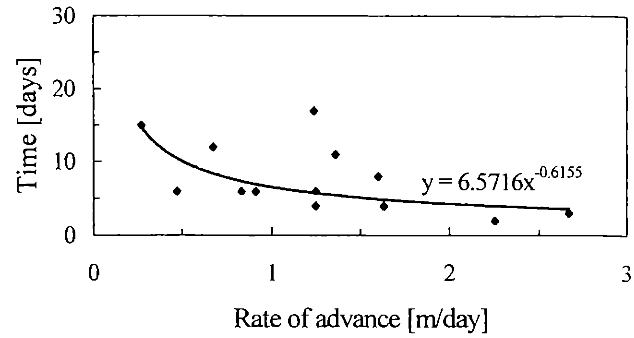

Figure 3. Time required to reach e_v=1% vs. the rate of heading advance for S convergence stations.

tunnel may be conducted should 75% deconfinement is allowed. Notwithstanding the inherent limitations of such an approach, the results, in question, are indicative that the mining operations should focus on special measures for the excavation and consequent support.

Close everyday examination of the primary support behaviour, during construction, revealed that signs of shotcrete failures appear at strain levels close to 1%, provided that this strain accumulation happens shortly upon completion of the re-excavation and support of the top heading in the section. Time dimension refers to the ability of the shotcrete top heading arch to conform, without cracks to strain accumulation. The ability of the heading shotcrete shell to sustain the observed overstressing, could be drawn from Figure 3, for the S sections. The graph displays the variation of the time (t), required to develop crown vertical strain level 1%, with the rate of heading advance (v).

As expected, the time to reach 1% inward strains at the monitored section is reduced by increasing the rate of top heading advance. The line fitting to data is described by the equation: $t = 6.57 \cdot v^{-0.615}$. It has to be born in mind, that this relationship constitutes a combined result of the interaction between the shell structural element, the surrounding rock mass and the supporting rock mass volume beneath the foundations of the shell.

Figure 4 presents the vertical displacements (V) along with the corresponding convergence (D) plots of the top heading measuring points, versus time, at S13 (ch. 1+171.56) as well as at K2 (ch. 0+849.54). Downward vertical displacements are plotted as positive, whereas positive D values are related to convergence. The curves show a characteristic behaviour of an ongoing failure due to the rapid overstressing of the primary support. The times respective to bench excavation, 1, as well as to ring completion, by invert excavation, 2, are also shown on the graphs. From the vertical displacement plots, it is evident that the shotcrete shell foundations of the top heading undergo severe non-uniform settlements. The excessive deformability of the rock volume, beneath these theoretical footings imposes bending to this structural element in the longitudinal direction.

At this point it is useful to examine the contribution of ring closure to control these phenomena. Note that, as a NATM fundamental concept, ring closure performed by invert excavation and support, is suggested when difficult tunnelling conditions are anticipated. Hence, contract documents specified ring closure following each 2 m heading advance, at a maximum. During construction, yet, this fundamental clause, has not been followed, because of the delays incident to the construction schedule.

The effect of ring closure cannot be considered apart from parameters such as: the time lap between excavation of the top heading and invert closure, the distance from top heading to closed invert as well as the previous straining of the section. Careful examination of the deformation-time histories showed, that invert closure at strain levels close to 2% cannot offer to the halting of straining. As an example, at S13 section (Fig. 4), invert closing occurred 6 days after primary support of the top heading has been placed. At that time, the top heading drift spanned only 5m ahead of the measuring section, whereas crown inward strain amounted to 1.8% (see Fig. 1). It seems, that rock mass mobilization and consequent shotcrete shell stiffness degradation rendered ring closure ineffective, in this particular case. Contrast to this, the deformation time history of K2 (ch. 0+849.54) section portrays, that ring completion even 21 days after top heading support at the section (with the top heading drift spanned 20 m ahead of the measuring section), at maximum strain level of 1.2%, provided the desired stabilization of the shotcrete shell straining.

3 CONCLUSIONS

The construction of Tymfristos road tunnel constitutes the most significant manifestation of rock squeezing behaviour experienced in Greece.

Squeezing takes effect immediately after the

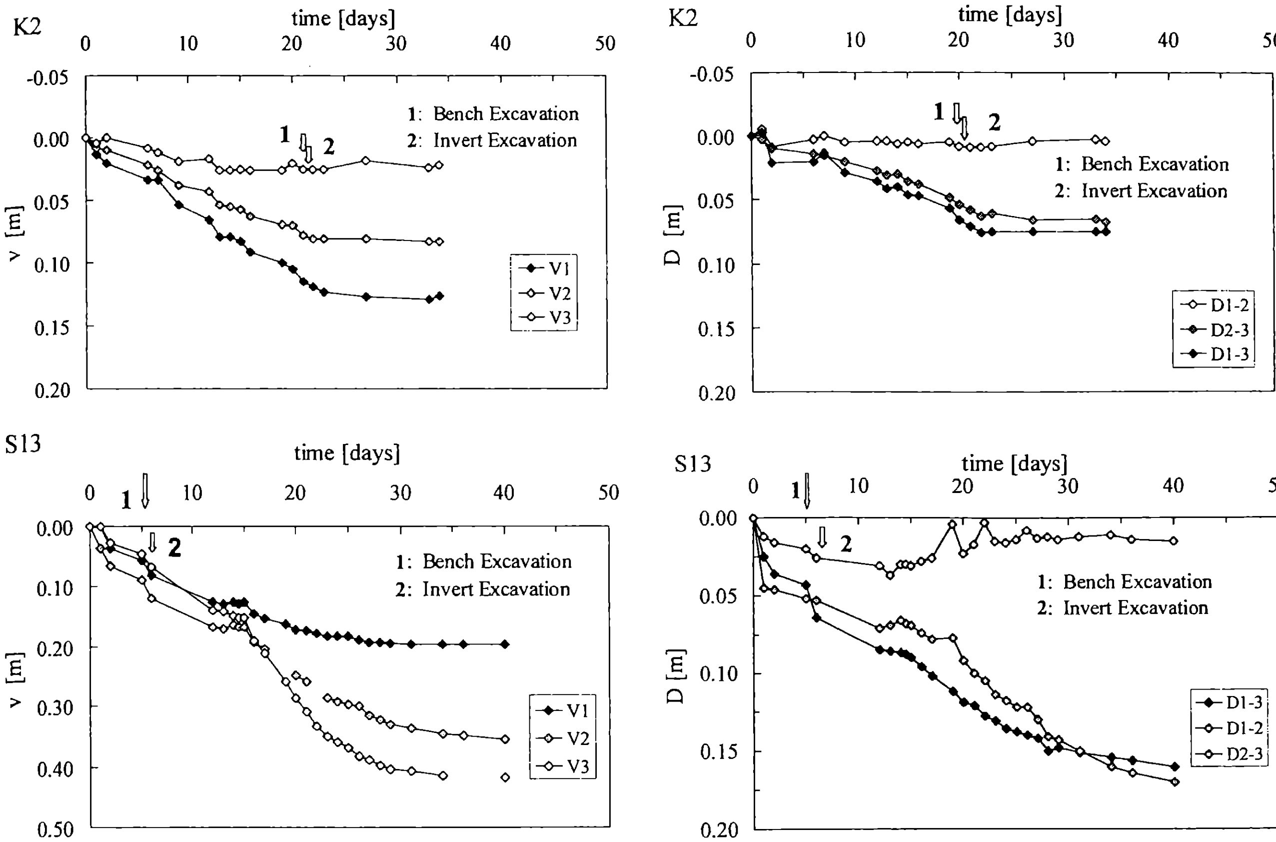

Figure 4. The vertical displacement and convergence time histories at K2 (ch. 0+849.54) and S13 (ch. 1+171.56)

shotcrete primary support placement. It is perceived as an abrupt development of severe, long lasting time dependent deformations around the opening. Such kind of deformation resulted in local failures of the freshly placed shotcrete shell. Furthermore, these instability phenomena were facilitated by the excessive deformability of the medium beneath the theoretical footings of top heading shell structure.

The conclusions presented herein are associated with the contract aimed at the re-excavation the top heading failed sections and the completion of the excavation-primary support of the overall cavity for a length of 531 m.

Concerned with the nature of the developed deformations, it is revealed that the shotcrete shell of the top heading suffers from vertical rather than from horizontal displacements. These are attributed to the bending induced in this structural element due to the poor load-deformation characteristics of the supporting medium.

Based upon the support deformation versus time histories and the close field observations of the response of the primary support, it seems that signs of overstressing appear for radial, inward strains close to 1%. The level in question, in this particular case, is reached at the early stages of shell construction and is related to the top heading advancing rate.

Severe cracking propagation and signs of instability of the primary support become obvious at strain levels close to 2%. This threshold value agrees well with the critical strain boundary between "stable" and "unstable" tunnels, suggested by Sakurai.

Ring closure at this strain level proves not effective at all, once the continuity of the primary support is already disturbed and its stiffness is far from that required. On the contrary, the completion of the shotcrete ring at lower strain levels leads to the limitation of deformations.

4 ACKNOWLEDGEMENTS

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