Analysis and Performance of the NATM Excavation of an Underground Station for the Athens Metro

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Proceedings: Fourth International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, March 9–12, 1998.

ANALYSIS AND PERFORMANCE OF THE NATM EXCAVATION OF AN UNDERGROUND STATION FOR THE ATHENS METRO

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Paper No 6.11

ABSTRACT

The paper describes the design methods and summarises the results of the analyses for the recent NATM excavation of an underground station of the Athens Metro. The non-linear finite element calculations show that for the excavation method used (twin side-wall galleries and a central pillar), the settlements at ground surface are very sensitive to the assumed values of the ground properties as well as the excavation sequence and construction details; it is shown that the settlements can be reduced significantly by increasing the number of the excavation stages and the density of the temporary support measures.

KEYWORDS

tunnelling, NATM, finite elements, support measures, ground settlements, soft rocks

INTRODUCTION

The design and construction of the Athens Metro has significantly advanced the experience of the geotechnical engineers in underground construction in an urban environment under difficult ground conditions (weathered heterogeneous soft rock). The project, currently in progress, includes the underground excavation of 11.7 kilometres of TBM running tunnels (9.5m in diameter), six NATM station caverns (16.5m wide, 12.5m high and about 110m long) and various access galleries, cross-overs etc. The remaining 6.3 kilometres of running tunnels, fifteen stations and the ventilation and auxiliary shafts are constructed by conventional cut-and-cover methods.

The construction of the underground stations is especially challenging from the geotechnical aspect since movements at ground surface must be minimised to avoid damage of adjacent structures. The difficulties are intensified due to the adverse ground conditions, the frequent presence of deep ancient wells and galleries causing instability and occasionally collapse of the tunnel face and roof (sometimes propagating up to the ground level), in conjunction with the large dimensions of the station caverns (ca. 16.50x110 m in plan) and their relatively low depth below ground level (10-12 m at the crest). For these reasons, the mining of the underground stations is performed in multiple stages using short excavation steps, immediate installation of the temporary support (sprayed concrete, rock bolts and occasionally steel sets) and careful monitoring of the ground movements and the convergence of the tunnel wall. The observed response of the ground to the excavation procedures is compared to the results of detailed finite element analyses of the complete excavation sequence. These analyses also assist in the investigation of the sensitivity of the excavation practices to the variation of ground properties, type, density and redundancy of the temporary support (e.g. because of low adhesion due to incomplete grouting of rock bolts etc.).

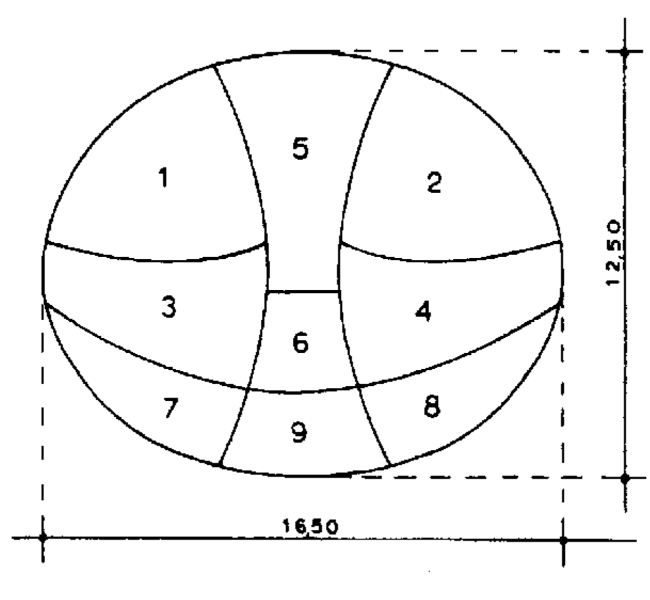

Fig. 1 Station Geometry and Excavation Stages

ODOTECHNIKI Ltd, acting as a specialist geotechnical consultant of the owner (ATTIKO METRO S.A.), has performed a detailed non-linear finite-element analysis of the complete excavation sequence for the Olympion underground station located on Line 2 of the Athens Metro. The excavation of the station was performed in nine stages using two side-wall galleries and a central pillar (Fig. 1). This method was considered the most appropriate to minimise ground movements for the specific soil conditions, since the oval-shaped side-wall galleries are relatively small and excessive compression of the central pillar (which obviously undertakes an appreciable load) is avoided by its lateral confinement using a thick sprayed concrete layer and temporary bolts fixed and tightened with end-plates at both sides of the pillar. The following sections describe the method used in the analysis of the Olympion Station and present the main results of the calculations.

GEOTECHNICAL CONDITIONS

Ground conditions at the Olympion Station were investigated by rotary core drilling and sophisticated sampling to minimise the disturbance of the recovered cores. The ground in the area consists of weathered "Athenian schist", overlain by 2-7 metres of man-made fill containing ruins of ancient (and more recent) structures. The main characteristics of the Athenian schist in relation to underground excavation are described by Kavvadas (1996) and Kavvadas et al (1996); in brief, the material is a low-level metamorphic graphitic/chloritic phyllite of Cretaceous age which has subsequently undergone variable degrees of weathering and alteration combined with tectonic folding and faulting. The resulting material is highly heterogeneous (at the scale of a few metres) covering the whole spectrum between a moderately weathered very blocky rock-mass with open gouge-filled discontinuities and a completely weathered engineering soil (lacking rock structure) containing inclusions of the original rock material. The geotechnical description of the cores recovered from the boreholes drilled along the station was performed using the empirical Material Rating (MR) index which is a variance of the well-known RMR index (Bieniawski, 1979) due to the fact that the classification was performed on borehole cores rather than the actual inspection of the tunnel face. More specifically:

1. The effect of groundwater conditions was invariably graded by "10" (damp), based on experience from the excavation of other Metro stations in the Athenian schist.

2. The orientation of the discontinuities was not graded due to the nature of the material (very blocky to completely weathered) and to the difficulties in determining the true orientation of the borehole cores.

While partially justified objections have been expressed regarding the applicability of the RMR (and thus also the MR) index in describing highly weathered and altered rock formations, such as the Athenian schist, which often resemble a stiff soil rather than a rock, it can also be argued that:

1. The MR index can be used with success in a comparative description of the Athenian schist, i.e. in comparing the variation of the rock-mass quality with depth or along the axis of the station, as well as in transferring experience from other stations mined in the same material, regarding the most appropriate excavation and temporary support procedures.

2. In using existing empirical correlations between the MR index and the engineering properties of the rock-mass (e.g. the deformation modulus or the shear strength parameters) extreme caution should be exercised since such correlations are usually based on data from rock-masses of superior quality compared to the Athenian schist. Thus, in the case of the Athenian schist such empirical correlations are used after calibration based on the measured response of the rock-mass.

Regardless of the applicability of the MR index in the Athenian schist, the formation in the area of the Olympion station is highly weathered and extremely heterogeneous with practically random spatial variation of its properties. Fig. 2 shows the distribution with depth of the MR index obtained from nine boreholes drilled along the station. The complete lack of any consistent variation in the horizontal direction is obvious by comparing data from the North part of the station (shown as open circles), the central part (shown as crosses) and the South part (shown as closed circles). Furthermore, the plot does not show any marked improvement of the rock-mass quality with depth, a fact which indicates weathering extending to a significant depth

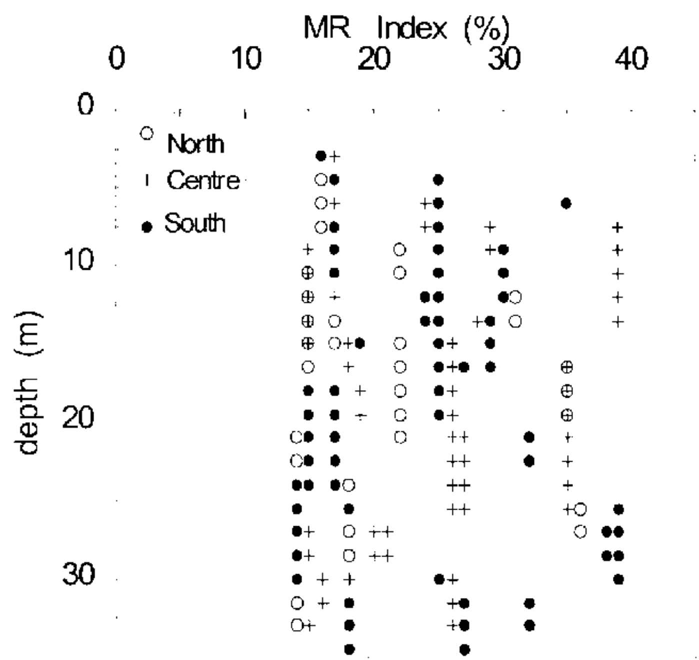

Fig. 2 Distribution of the MR index with depth along the Olympion Station

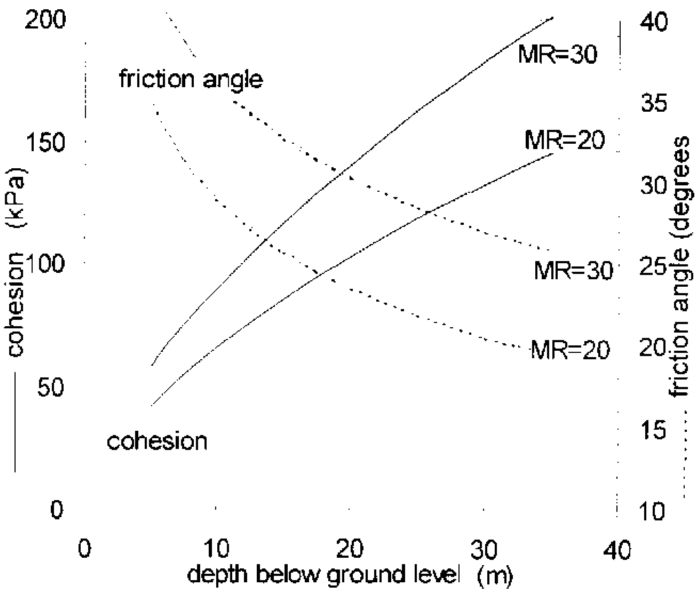

Fig. 3 Distribution with depth of the shear strength parameters for the rock-mass classes C and D-1 (MR=30 and 20 respectively).

due to the intense tectonic disturbance of the material. As a result of the above, for the purposes of the numerical analysis the formation in the area of the Olympion station can be considered as statistically homogeneous at the scale of the dimensions of the excavation.

Because of the variability and generally poor quality of the rock-mass (MR=15-40) it was decided to perform parametric non-linear finite-element analyses of the complete excavation sequence using three alternative sets of material parameters with the following values:

1. Material Rating (MR) index values equal to 30, 20 and 15. These values scan the typical range of the Athenian schist (Fig. 2). The corresponding values of the uniaxial compressive strength (σ_c) are assumed to be: 12.5 MPa, 5 MPa and 3 MPa, recognising that lower MR values usually indicate higher degree of weathering and thus lower strength. The above three sets of parameters are designated by the symbols C, D-1, D-2 in the following. As shown in Fig. 2, soil type D-2 is a very conservative assumption and was only studied in order to obtain a lower bound of the possible response of the ground to the excavation of the station cavern.

2. The modulus of elasticity (E) of the rock-mass. In a non-linear analysis this parameter describes the elastic behaviour of the material, i.e. its response before the onset of plastic deformation, at which point the value of the "deformation modulus" drops significantly. The reduced "deformation modulus" is accounted by the non-linear features of the numerical algorithm independently of the modulus of elasticity (E). Thus, the elasticity modulus describes the initial stiffness rather than the average stiffness. Previous back-analyses of the measured settlements in several NATM excavated stations of the Athens Metro indicates that the modulus of elasticity of the Athenian schist can be obtained using the following empirical relationship:

$$E(GPa) = \left(\frac{\sigma_c(MPa)}{100}\right)^{2/3} \cdot \mathrm{alog}\left(\frac{MR-10}{40}\right)$$

This relationship is a modification of the formula proposed by Serafim and Pereira (1983):

$$E(GPa) = \mathrm{alog}\left(\frac{RMR-10}{40}\right)$$

The philosophy of the proposed modification is that the Serafim and Pereira formula (Eq 2) is based mostly on data from rocks with relatively high intact strength (of the order of σ_c=100 MPa) in which the deformability of the rock-mass is controlled by the discontinuities (i.e. by RMR); in weak rocks with low σ_c values, the deformability of the rock-mass is also influenced by the elasticity of the individual rock blocks which can be expressed (indirectly) by the value of σ_c, thus obtaining the modified form of Eq 1. Using the proposed relationship (Eq 1), soil types C, D-1 and D-2 give the following values of the elasticity modulus: 800 MPa, 225 MPa and 120 MPa.

3. For a rock-mass with MR values exceeding 20, the shear strength parameters were calculated using the Hoek-Brown failure criterion with m_i=9.6 and (m, s) divisors equal to (28, 9), values which correspond to a low-disturbance excavation method (by back-hoe tunnel excavators). Since the failure envelope of the Hoek-Brown criterion is curved, the "equivalent" Mohr-Coulomb parameters (c, φ) are stress-level dependent as shown in Fig. 3 for soil types C (MR=30) and D-1 (MR=20). Soil type D-2 was considered to be engineering soil with constant shear strength parameters: c=50 kPa, φ=25°.

FINITE ELEMENT ANALYSES

The excavation of the station cavern was modelled using the finite element computer program SOFiSTiK. A two dimensional (plane strain) model was employed; the effect of the longitudinal tunnel axis, namely the ground movements occurring ahead of the excavation face and before the application of the temporary supports were modelled using the stiffness reduction method, i.e. (i) application of the geostatic stresses, (ii) reduction of the ground modulus to 30% of its initial value in the excavation zone, (iii) application of the temporary support and (iv) removal of the ground within the excavation zone via a de-activation of the finite elements in this area. The analysis attempted to model all the relevant issues as accurately as possible; the rock-mass surrounding the tunnel was modelled as an elastic-perfectly plastic material (Hoek-Brown failure criterion), the sprayed concrete of the temporary lining via beam elements, and the passive rock bolts were modelled as elastic-perfectly plastic springs

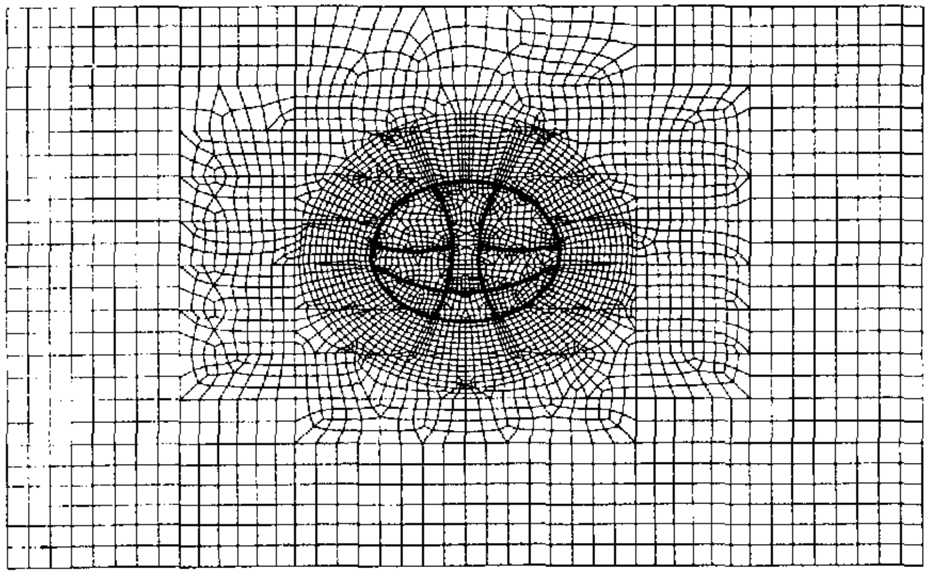

Fig. 4 Finite Element Mesh Used in the Analyses

having a cut-off load controlled by the adhesion along the grout-soil interface (or the yield load of the steel bar, whichever was lower).

Figure 4 shows a typical layout of the finite element mesh used in the analyses (an area 80m wide and 50m high). The top boundary was assumed to be stress-free, the lateral boundaries were constrained in the horizontal direction while the bottom boundary of the mesh was constrained in the vertical direction.

The temporary support system consists of sprayed concrete (class B25) 10-40 cm thick depending on the position (10cm on temporary walls, 40cm above the crest line, 30cm below the crest-line) and passive rock bolts. A relatively low modulus of elasticity (equal to 7 GPa) was used for the sprayed concrete in order to model its slow setting time (compared to the advance rate of the tunnel face). The passive rock bolts consist of a 25 mm diameter rebar (ST IV) pushed into a 56 mm diameter pre-grouted borehole. The length of the rock bolts is 5-6 m in the upper part of the tunnel section and 4.5-5 m in the lower part of the section. Their ultimate load is 0.25 MN for soil types C, D-1 and 0.15 MN for soil type D-2, and their spacing at the tunnel wall is 1.0 metre.

The nine main excavation stages (Fig. 1) were numerically modelled as a sequence of 19 load cases, including the initial application of the geostatic stresses and a two-phase excavation of each stage (phase 1: stiffness reduction as described above, phase 2: installation of the temporary support and completion of the excavation).

CALCULATION RESULTS

Figures 5, 6 and 7 summarise the main results of the analysis. Lines C, D-1 and D-2 correspond to tunnel excavation in the respective soil types, while the line marked as "D-1 TRANS" corresponds to excavation in soil type D-1 but assuming that half of the rock bolts installed in excavation phases 1 and 2 are inactive (to model the possibly incomplete grouting of the boreholes drilled at steep angles in the upper part of the tunnel section).

Figure 5 plots the prediction of the cumulative maximum settlement at ground level (occurring above the centre-line of the tunnel) in each of the main excavation phases. Comparison of the predicted settlement in each excavation phase shows the relatively high contribution of the demolition of the central pillar (excavation phase 5), especially in soil type D, due to the significant stress

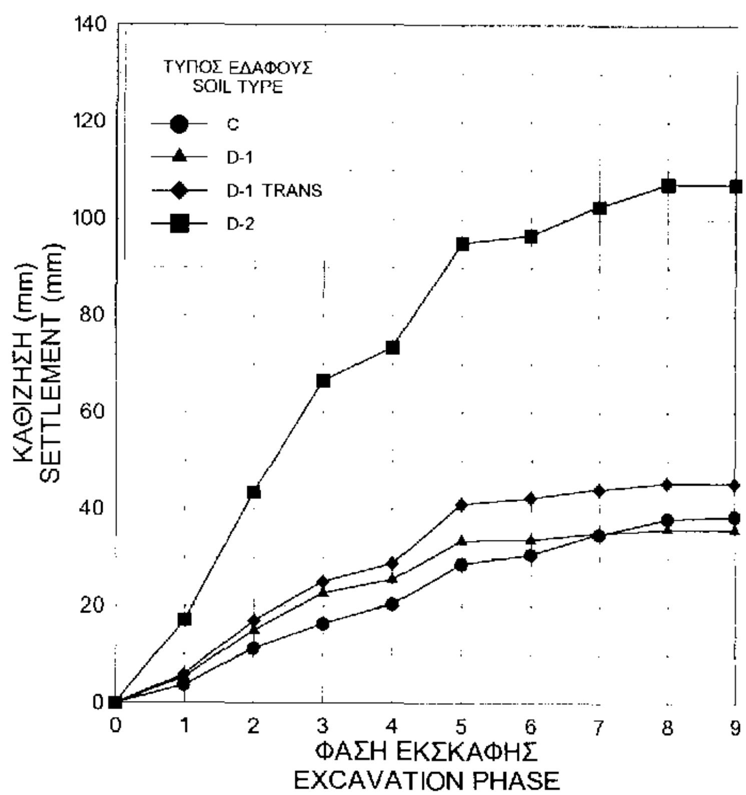

Fig. 5 Finite Element prediction of the maximum settlement at ground surface during the sequence of excavation stages

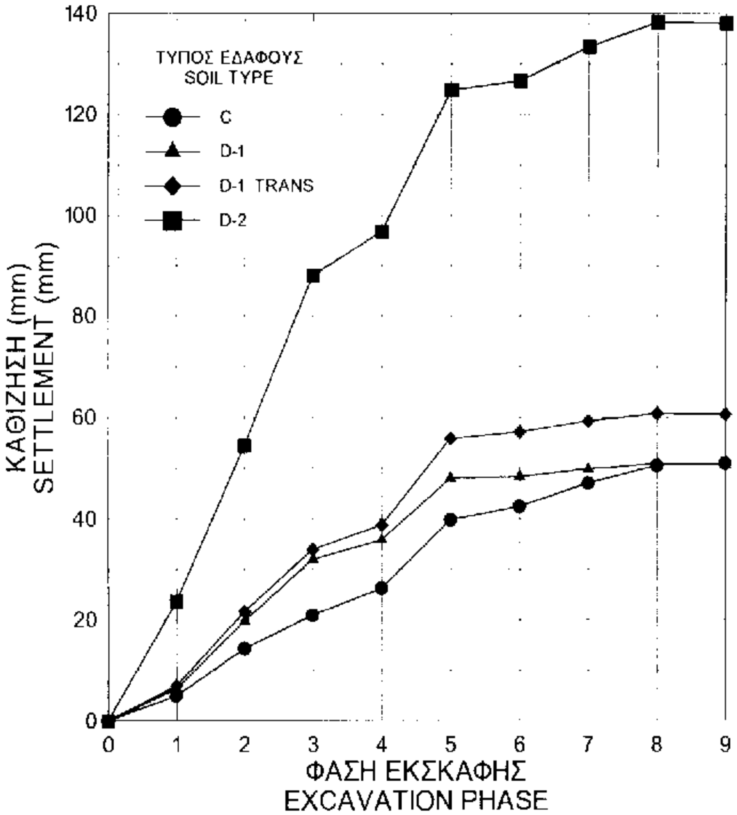

Fig. 6 Finite Element prediction of the settlement at the tunnel crest level during the sequence of excavation stages

concentration in this element during the excavation of the two side-wall galleries. Furthermore, in all cases, almost 90% of the total settlement accumulates before the demolition of the central pillar showing that the strength of the central pillar influences the settlements in all previous excavation phases.

Figure 6 plots the prediction of the cumulative settlement at the tunnel crest in each of the excavation phases. Comparison of the predicted settlements at the tunnel crest and at ground level shows that their ratio is about 1.30.

Finally, Fig. 7 plots the predicted shape of the settlement trough at ground level, for each of the soil types analysed, after the completion of the excavation. The width of the settlement trough is approximately 50 metres, i.e. about three times the width of the excavation itself (16.5 metres). The average (i.e. over a distance of 4m) predicted slope of the settlement trough occurs at a distance of about 12 meters from the tunnel axis and is equal to 1/600 for soil types C and D-1 and equal to 1/200 for soil type D-2. The average predicted slope for soil type C is slightly higher than that for soil type D-1, probably due to the fact that ground settlements in the softer soil are higher but more evenly distributed and extend over a larger distance compared to the stiffer soil. The maximum predicted slope (locally, at a distance of about 12 meters off the tunnel axis) is equal to 1/400 for soil types C and D-1, and equal to 1/80 for soil type D-2 (however, as mentioned previously, soil type D-2 corresponds to a very conservative set of material parameters and was studied

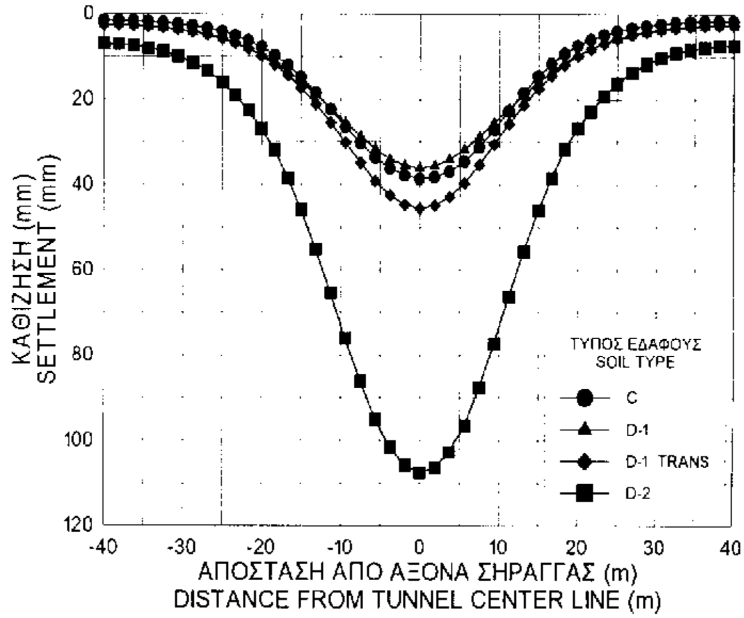

Fig. 7 Finite Element prediction of the settlement trough at ground surface (perpendicular to the axis of the station) after the completion of the excavation

only to establish an absolute upper bound of the possible ground settlements).

The magnitude of the maximum predicted settlement at ground level ranges between 38mm for soil C and 107 mm for soil D-2. As mentioned above, the set of material parameters corresponding to soil type D-2 is very conservative and was used mainly to investigate the stability of the station cross-section under the most adverse possible ground conditions. Actual measurements obtained during the construction of the station show that the maximum observed ground surface settlement was 18mm while the average ground surface settlement along the station axis was 15mm. However, most of the monitoring locations were located on adjacent structures or even on the pavement of the street and their measurements were obviously influenced by the stiffness of these structural elements, resulting in lower measured values compared to the predicted green-field conditions. Furthermore, in several occasions the actual temporary support measures installed were significantly stiffer than those used in the predictions (e.g. steel sets were incorporated in the sprayed concrete layer, additional rock bolts were installed at selected locations in order to adapt to local ground conditions, etc.).

In addition to the ground settlements, the analytical model also predicted the axial forces developing in the passive rock bolts and showed that for soil types C and D-1 they do not reach their ultimate capacity. On the contrary, in the case of soil type D-2 a significant portion of the rock bolts reaches the ultimate load and deforms plastically. This is due to the increased loads on the temporary support (caused by the poor ground quality) as well as the reduced value of the adhesion at the grout-soil interface.

CONCLUSIONS

The paper presents the analytical method and the main results of the calculations used in modelling the excavation of the Olympion underground station of the Athens Metro. The analysis was performed with the finite element method (computer program SOFiSTiK) and modelled the complete excavation and support installation sequence as well as the non-linear elasto-plastic behaviour of the ground surrounding the station. A parametric analysis was performed using three alternative sets of ground properties (soil types C, D-1 and D-2). Of the three sets, soil type D-2 represents a very conservative estimate of the ground properties and was included in order to obtain an absolute lower bound of the possible ground response.

The analyses show that the excavation is stable and the settlements are within acceptable limits for soil types C and D-1; in the case of soil type D-2 the magnitude of the maximum predicted settlement at ground surface exceeds 100 mm and several of the rock anchors reach their ultimate location and deform plastically. The largest portion of the ground settlement occurs during the removal of the central pillar, due to the significant stress concentration in this element during the excavation of the two side-wall galleries. Comparison of the predicted settlements at ground surface with the measured values during the actual excavation of the station indicates that the measured settlements are lower than the predicted. This is probably due to a more favourable response of the Athenian schist at the large scale, the additional support measures installed during the excavation of the station (increased thickness of the shotcrete shell, additional rock bolts, installation of steel sets, etc.) as well as a systematic bias of the measured settlements (compared to the green-field predictions) mainly due to the stiffness of the structural elements on which the settlement monuments were attached.

ACKNOWLEDGEMENTS

The author would like to express his appreciation to his colleagues for their assistance in the collection of the data included in the paper and to ATTIKO METRO S.A. for the permission to publish the data. The arguments and conclusions in this paper express the personal opinion of the author and not necessarily the official view of ATTIKO METRO S.A.

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