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Experiences from the construction of the Athens Metro

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ABSTRACT: The paper presents the geotechnical conditions and engineering properties of the soil types (mainly weathered Athenian schist) encountered during the present construction of the Athens Metro. Measured surface settlements along the Line 3 TBM tunnel and a NATM excavated underground station are also presented and evaluated.

1 INTRODUCTION

The construction of the Athens Metro has provided extensive experience on soft ground tunnelling in urban environment. The project consists of two lines (total length 18 km), 21 stations, 29 ventilation shafts and various miscellaneous structures. It was awarded as a turnkey contract to the *Olympic Metro* consortium comprising of 25 German, French and Greek firms, and is estimated to cost about 2 billion ECUs. The project owner is *Attiko Metro A.E.* who employ *Bechtel International* to assist with project management. Construction started in November 1991 and when the system becomes operational (in 1998) it will serve approximately 450,000 passengers per day.

Three methods of construction are used: TBMs bore 11.7 km of the twin-track running tunnels which are located at a depth of 15-20 m (measured at the crown); short auxiliary tunnels and six station caverns are excavated using NATM; the remaining 15 stations, 6.3 km of running tunnels and all shafts are constructed by cut-and-cover methods with struts and/or prestressed anchor tie-backs to support the vertical walls.

Two 9.5m diameter TBMs, one for each line, were designed for the specific ground conditions by Mitsubishi. Each features an open-type, eight-spoke cutterhead fitted with drag bits and single disk cutters to bore through a variety of ground conditions ranging between completely decomposed schist and soft limestone. A 0.35m thick reinforced concrete seven-piece (plus a key) segmental lining is erected under the shield of the machine in 1.50m long rings. Portland cement grout is subsequently injected into the annulus between the ground and the lining.

The stations are of the side-platform type and most of them are 110m long. The NATM underground stations are oval shaped caverns (internal width 15m, height 11m), their crest is located about 15m below ground level and they are provided with a 0.60m thick cast-in-situ reinforced concrete final lining. The cut-and-cover stations are constructed in 15-17m deep excavations supported by bored soldier piles, horizontal struts and/or prestressed anchor tie-backs. NATM excavation (in multiple stage lifts) is adopted when the available space is not sufficient or the existence of buried antiquities precludes open excavation. The excavation sequence varies according to the ground conditions between the classical heading-bench-invert top-down method and the method employing two side-wall drifts with a central pillar. Each of the excavation stages is usually performed in multiple phases with simultaneous installation of the temporary support, in order to minimise the settlements at ground level. The primary support system consists of a 15-30 cm thick steel-mesh reinforced shotcrete shell combined with either steel arches (HEB 140 standard sections or lattice girders) or fully grouted passive rock dowels 4-6 m long.

An essential requirement of the selected excavation and support methods is the limitation of ground movements in order to minimise their effects on neighbouring structures. This is achieved by elaborate design procedures, using elasto-plastic finite-element modelling of the complete sequence of excavation and support installation stages, and comprehensive monitoring of the ground movements, convergency of the shotcrete shell, ground pressure on the shotcrete, pore water pressures etc.

2 GROUND CONDITIONS

Tunnelling for the Athens Metro is performed mostly in the so-called "Athenian schist" a term erroneously used to describe a geological formation comprising a variety of low-level metamorphic and sedimentary (non-metamorphic) weak rocks (Marinos et al, 1971; Dounas & Gaitanakis, 1981; Sabatakakis, 1991). The Athenian schist is a sequence of Upper Cretaceous (Maestrichtian) flysch-type sediments, about 200m thick, formed as materials originating from the surrounding ridges were accumulated in the Athens Basin (a Lower Cretaceous tectonic depression). The sequence consists of thinly bedded clayey and calcareous sandstones, alternating with siltstones (greywackes), slates, shales, marls and limestones. The material has usually a dark grey to green-grey colour and is characterised by a pronounced higher proportion of shale-like sediments compared to the coarse-grained materials. In close association with the flysch sequence are inclusions of ophiolites, a result of underwater (sub-volcanic) magmatism prior to or concurrent with sedimentation. The identified basic flows extend over the entire spectrum of ophiolites including spillitic forms of diabase, dolerites and peridotites (usually altered to serpentines).

During the Eocene, the Athenian schist formations were subjected to intense folding and thrusting (Pyrenean orogeny). Further to folding, the entire basin underwent extensive faulting which caused extensional fracturing and widespread weathering and alteration of the deposits. The extent to which the flysch formations responded to the tectonic stresses varies according to their stiffness: clay shales, being more ductile, were intensely folded and developed some schistocity while more brittle rocks like sandstones and limestones were badly sheared and faulted. As a result of the extensive weathering and tectonism, the Athenian schist in many locations is completely decomposed and can no longer be characterised as rock but has the mechanical characteristics of a heterogeneous shale.

The geotechnical investigation for the project included a large number of boreholes (over 500) drilled to depths of 25-35m. The initial campaigns used single tube core barrels with carbide bits. Subsequent investigations used double tube core barrels with stepped diamond bits in order to improve the quality of the recovered samples. However, despite the use of large diameter samplers, it proved difficult to retrieve undisturbed samples of the weak weathered flysch deposits which are more interesting from the engineering point of view since they control the mechanical behaviour of the system. This was due to the appreciable decomposition, schistocity and dense jointing of the flysch. Systematic measurements of the groundwater level were performed in all boreholes and Menard-type pressuremeter tests were executed in several of them.

Regarding the groundwater conditions, tunnelling and deep excavations have shown that the Athenian schist has low permeability, due to the prevailing effects of the clay-like shales, with the following exceptions:

(1) Concentrated seepage along relatively more permeable paths, such as highly fractured sandstones or limestomes and intensely tectonised zones.

(2) Low capacity perched aquifers forming along the interface between an upper more weathered (and thus more permeable) layer and a lower less weathered zone. This feature is more pronounced along the upper limit of the schist formation at its interface with the overlying quaternary or recent alluvia.

The materials sampled from the boreholes were characterised using the so-called MR (Material Rating) index, a variance of Bieniawski's (1979) RMR without the adjustment for discontinuity orientation, because the large number and orientations of the discontinuity sets usually result in a statistically isotropic mass at the scale of the excavations. In rating borehole samples, damp conditions were assumed (MR rating = 10). Since the RQD values in the schist are almost invariably less than 30%, and very often zero (giving an MR rating for RQD less than 4), and the uniaxial compressive strength of the intact shale which is predominant in the formation is low (MR rating less than 4), the values of the MR index in the shale portion of the schist are controlled by the spacing and condition of the discontinuities and the presence of water. Groundwater conditions generally have not caused appreciable problems in NATM tunnelling; the material exposed at the face of the tunnels is occasionally damp (MR rating = 10), usually wet (MR rating = 7) and sometimes dripping (MR rating = 4). The MR values measured on the face of the excavations are usually similar to those obtained from borehole samples because the effects of sample disturbance in the boreholes were mostly offset by

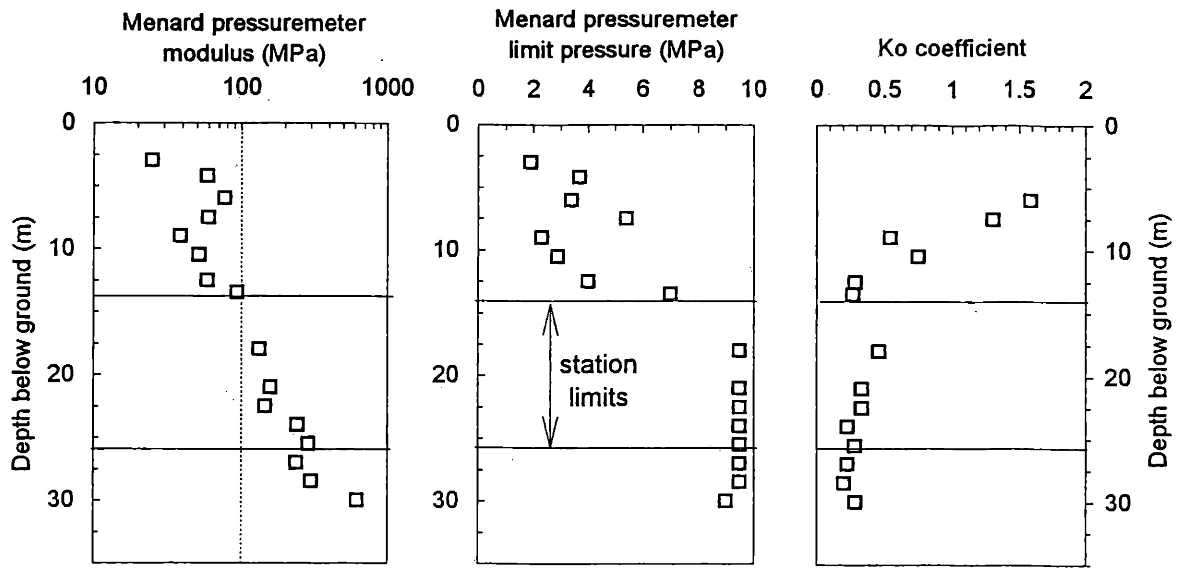

Figure 1: Menard-type pressuremeter test results from boreholes at the Omonia underground station.

the somehow more adverse groundwater conditions encountered during excavation: damp conditions were assumed in rating borehole samples, whereas the actual conditions on the face were usually wet or dripping.

Although serious questions have been raised regarding the applicability of the MR index in soft ground tunnelling (since this index is based on RMR

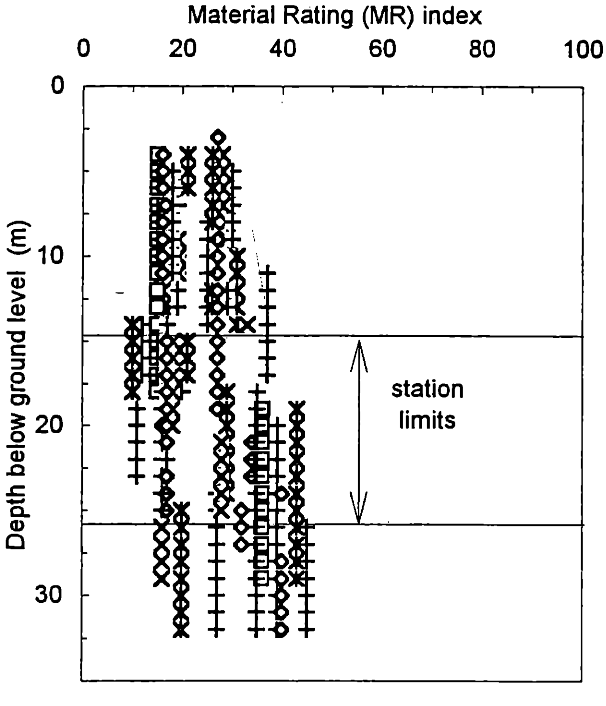

Figure 2: Distribution of the MR index with depth. Results from ten boreholes in the area of the Omonia station.

which is derived largely from experience in rock tunnelling), the MR is a useful tool in this specific project, mainly in correlating the engineering behaviour of the schist at various locations and in building up a database of geotechnical parameter sets, excavation procedures and support measures for tunnelling in the Athenian schist. This database is certainly not directly applicable in different ground conditions.

Figure 1 presents the results of Menard-type pressuremeter tests performed in two boreholes at the Omonia underground station which was excavated at a depth of 14-26m below ground level. There exists a tendency of the Menard modulus (E_M) and the corresponding limit pressure (P_L) to increase with depth and the Ko value to decrease with depth as the material becomes less weathered. The average values of the pressuremeter parameters in the zone of influence above the crown of the station are E_M=70 MPa and P_L=4 MPa while those at the face of the excavation are somehow higher (E_M=130 MPa and P_L=8 MPa). The few values of the modulus exceeding 500 MPa correspond to thin layers of sandstone and limestone which, however, only had a minor beneficial influence in the response of the ground to the excavation of the station cavern.

Figure 2 presents the results of the Material Rating (MR) index obtained from the cores of ten boreholes drilled in the area of the Omonia station. As mentioned above, this index is analogous to Bieniawski's RMR and ranges between 0-100. Measured values of the MR are between 15 and 45,

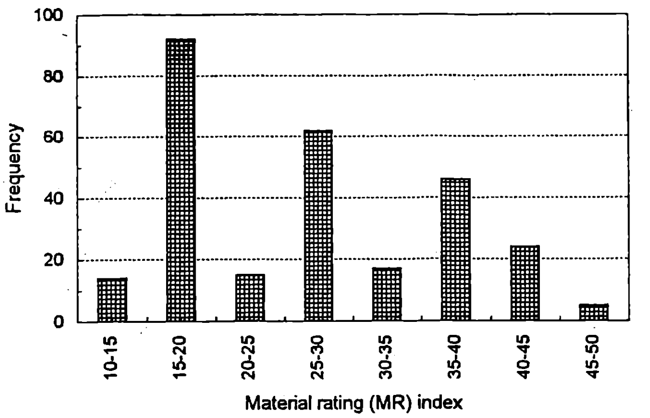

Figure 3: Frequency distribution of the MR values from ten boreholes in the area of the Omonia station.

with the higher values (>40) corresponding to the intercalated layers of sandstones and limestones. There is no clear tendency of the MR values to increase with depth neither a consistent variation in the lateral direction along the length of the station, indicating that the deleterious effects of weathering and tectonism extend over the whole investigated area. Figure 3 shows the relative frequency of the MR values in the same boreholes (275 measured values). The marked concentration of values in the range 15-20 is representative of the completely decomposed schist and corresponds to about 30% of the total values.

For NATM design purposes, four types of ground conditions were analysed, all corresponding to the weak rock - stiff soil range (Table I)

TABLE I
Soil types for the design of NATM temporary support of underground stations

Soil type	MR range
A	40-50
B	30-40
C	20-30
D	< 20

Shear strength parameters for soil types A-C were obtained using the Hoek-Brown failure criterion with material parameter m_i=9.6. The (m,s) divisors were assumed equal to (28,9) corresponding to mild excavation procedures (by back-hoe excavators in multiple stage drifts and early installation of the temporary supports). According to Hoek-Brown theory, the shear strength parameters (c,φ) vary with depth as they depend on the effective stress level. This variation was exploited in the finite element analyses used to design the temporary support systems. For soil type D, the shear strength parameters used for NATM design varied among stations (depending on local conditions) and generally were in the range c=10-60 kPa and φ=25-28°.

3 TBM TUNNELLING

Although ground conditions from the geological aspect are relatively homogeneous, until the present time (January 1996) the two TBMs have encountered soils with very different engineering properties and their performance has varied accordingly. Line 3 TBM, operating since October 1994, has bored 2.7 kilometres without appreciable delays at an overall average penetration rate of 5.8 metres per day based on an 18-hour-per-day working shift (including all stops and down-times). It is noted that daily advance rates of 18 metres were often sustained for several days under favourable conditions. Ground settlements at surface level usually ranged between 3-8 mm and occasionally reached 10-12 mm.

Line 2 TBM, operating for a slightly longer period, has bored only 650 metres at an overall average penetration of 1.6 metres per day. The low performance of this machine is due to large and occasionally uncontrollable overbreaks which have caused major delays while freeing the machine and grouting the cavities (which sometimes reached ground surface). The principal reason for the observed ravelling tendency of the ground seems to be insufficient cohesion in the intensely weathered and highly tectonised zones, in conjunction with the large muck openings of the TBM cutterhead which cannot adequately control muck-flow (the cutterhead operates in the open air i.e., under atmospheric pressure). The effect of soil cohesion (c') on the stability of the tunnel face can be illustrated via the Overload Factor (OF=σ_{vo} / c_u) for a tunnel at a depth of 21m (where σ_{vo}=460 kPa). Assuming effective shear strength parameters c'=40kPa, φ'=28°, a K_o value of 0.40 and a pore pressure parameter A_f = -0.25, the computed equivalent undrained shear strength is c_u=230 kPa and corresponds to an overload factor OF=2. For reduced strength parameters (c'=20kPa, φ'=25°, A_f = 0) in the intensely weathered zones, the computed undrained shear strength drops to 115 kPa and the overload factor doubles (OF=4). This significant increase in the overload factor can explain the

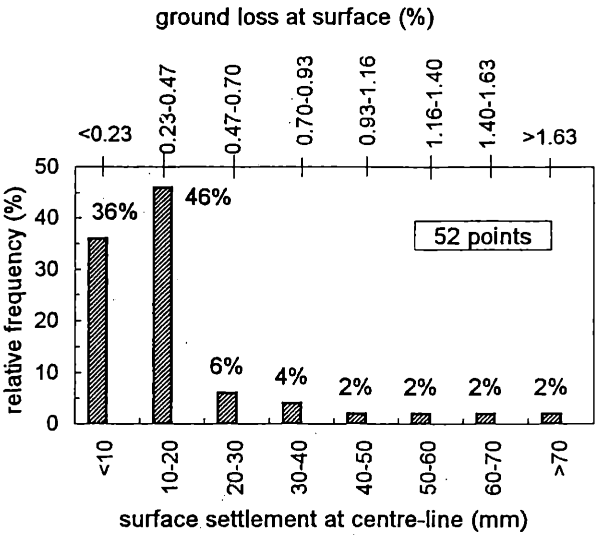

Figure 4: Frequency distribution of the measured surface settlements above the tunnel centreline and ground loss at surface level. Line 2 TBM tunnel.

observed face instabilities.

Figure 4 presents the frequency distribution of the recorded surface settlements above the centreline of the Line 2 TBM tunnel and the corresponding estimated ground loss at surface level (i.e., the volume of the settlement trough at surface divided by the excavated volume). These values represent the most adverse TBM tunnelling conditions in the project, at least until the present time, since surface settlements along the Line 3 TBM tunnel were significantly lower (3-12 mm). The maximum observed settlement was 119 mm while 82% of the measured settlements were less than 20 mm. Figure 5 presents the shape of the surface settlement trough obtained from measurement points offset with respect to the axis of the Line 2 tunnel by a distance (x) normalised with the tunnel radius R=4.75m. The settlements are normalised with the corresponding

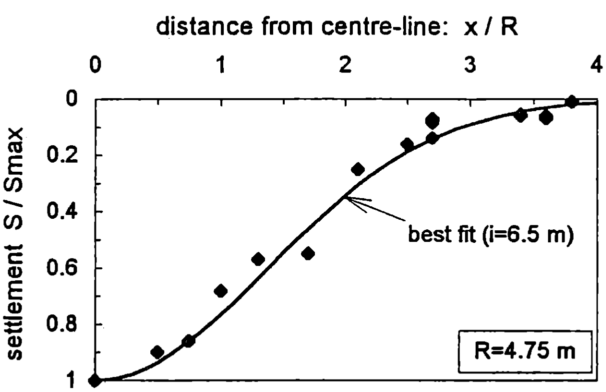

Figure 5: Normalised shape of the surface settlement trough. Line 2 TBM tunnel.

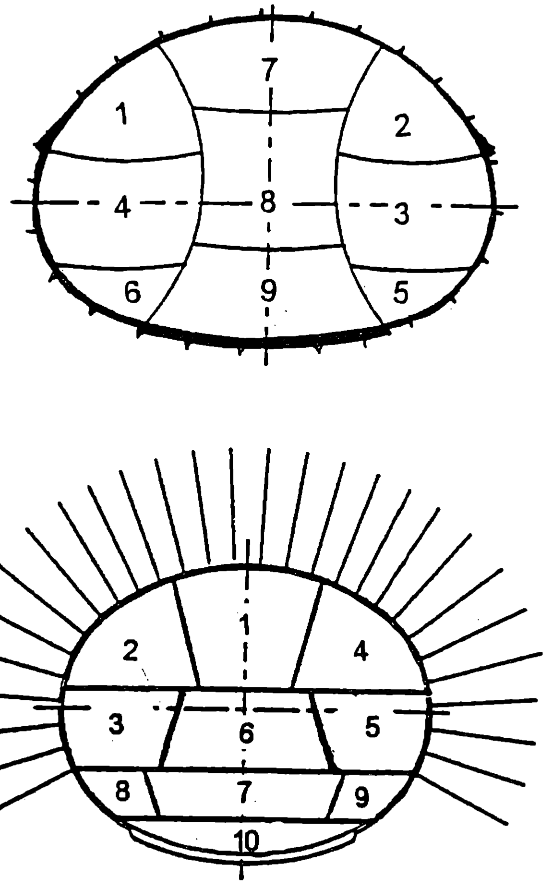

Figure 6: Typical excavation sequence of NATM underground stations. Width=16.5m, height=12.7m.

maximum surface settlement ($s_{\max}$) at the tunnel axis in order to obtain a unique shape curve. The Figure also plots the mathematical curve:

$$s / s_{\max} = \exp\left(-x^2 / 2i^2\right)$$

which gives the best fit of the measured values for i=6.5m, where (i) is the distance from the centreline to the inflection point of the trough curve. The volume of the settlement trough (ΔV) per unit length of the tunnel for this curve is given by the formula: $\Delta V = 2.5 * i * s_{\max}$. This formula was used for the estimation of the ground loss in Figure 4.

4 NATM UNDERGROUND STATIONS

Figure 6 presents the typical excavation sequence and temporary support measures employed in the construction of the six NATM underground stations. The top part of the figure shows the two side-wall drift method with a central pillar, used e.g. at the Omonia and Olympion underground stations. Temporary support consists of a thick double-mesh reinforced shotcrete shell (25-30cm on the external walls, 20cm on the pillar) and lattice girders or steel sets at a spacing of about one meter. The bottom part of the figure shows the method used at Ambelokipi station (classical top-down heading-bench-invert multiple phase excavation). Temporary support in this case consists of a shotcrete shell (about 20cm) and passive bolts (25mm fully grouted rebars, L=4-6m, at a spacing of 1.0-1.5m).

Table II presents the magnitude of the peak (at the centre-line) surface settlements at three locations along the Omonia underground station as they accumulated during each excavation stage (see also Figure 6).

TABLE II
Max. surface settlements (mm) at the Omonia Station

	Excavation Phase	CH 3297	CH 3341	CH 3353
side wall drifts	1	8	9	8
	2	7	8	8
	3	3	5	5
	4	1.5	4	5
	5+6	1.5	2	2
central pillar	7+8	10	11	8
	9	1	3	4
	Recesses	-	12	10
	Time dependent settlements	10	9	10
	Total	42	63	60

A significant portion of the total settlement (about 25%) occurs during the removal of the central pillar. The time dependent settlements, which are mainly due to the dissipation of the excess pore water pressures built-up during the excavation (consolidation settlements), amount to an additional 15-20% of the total. Finally, the excavation of the reeesses (after the completion of the station cavern)

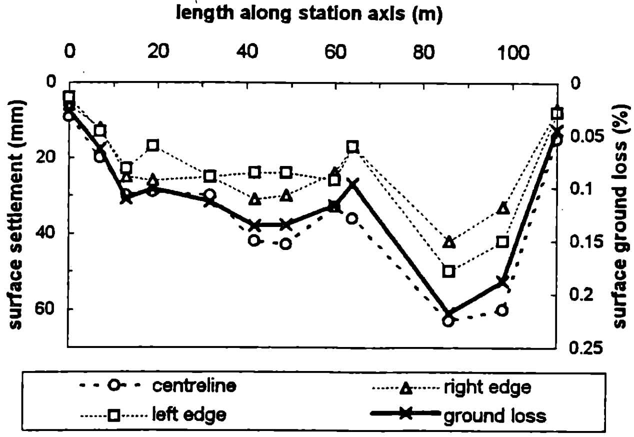

Figure 7: Surface settlement distribution and ground loss along the Omonia underground station.

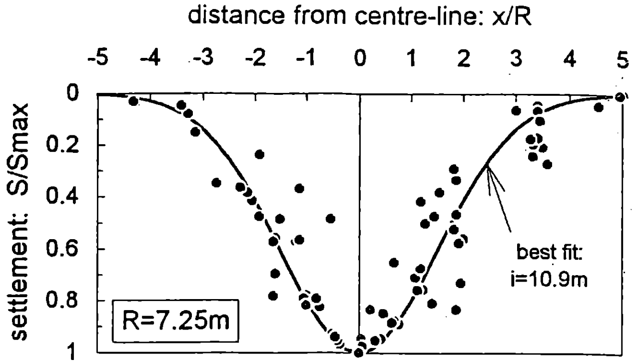

Figure 8: Normalised surface settlement trough at Omonia underground station.

towards the eastern end of the station added an additional settlement (about 15% of the total value).

Figure 7 presents the measured surface settlements along the Omonia station, at the centre-line and at the left and right edges (8m away from the axis). It also presents the distribution along the station of the ground loss at the surface which is in the range of 0.1-0.2%. The appreciable increase of the settlements towards the end of the station is attributed partly to poor ground conditions and partly to the effect of the two recesses excavated in this area after the completion of he station cavern. Figure 8 presents the normalised shape of the surface settlement trough at Omonia station.

CONCLUSIONS

The paper reviews the design methods and construction practices of the Athens Metro project. The efficiency of TBM tunnelling has varied significantly and roof collapses of appreciable size often occurred. NATM construction of the station caverns using various excavation and support techniques has proved to be easily adaptable to the local variability of the terrain.

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