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Deep Excavations in Hard Sandy Clays for Stations and Shafts of the Athens Metro Stavros Extension

Richard Kulesza
Bechtel Corporation, San Francisco, CA

Nikos Boussoulas
ATTIKO METRO, Athens, Greece

W. Allen Marr
GEOCOMP/GeoTesting, Boxborough, MA

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ABSTRACT

A 26 m deep excavation in hard sandy clays for the Halandri Station of the Stavros Extension of Athens Metro was completed in 2001. The excavation supporting system and the areas outside the excavation were instrumented. During the design, a finite element analysis of stresses and displacements was performed using soil parameters developed from laboratory and in-situ testing. The measured displacements had a similar distribution with depth as the calculated displacements, but were significantly smaller. An extensive finite element back-analysis was performed, in which the input parameters were varied until the magnitude and the distribution with depth of calculated displacements agreed as closely as practicable with the measured displacements. Comparison was made with displacements measured at other Stavros Extension deep excavations in similar soil. The soil parameters established from the back-analysis can be considered applicable to the design of planned further Athens Metro extensions located in similar type of soil.

INTRODUCTION

Several extensions to the Athens Metro system are currently under construction or were recently completed. The Line 3 Stavros Extension leading towards the new airport required commissioning prior to the 2004 Olympic Games. The location of this extension is shown on Figure 1. While the tunnels and stations of the previous lines were largely excavated in Athens Schist, the major part of the Line 3 Extension passes through stiff to hard sandy clays, believed overconsolidated mostly by desiccation. The largest excavation for Line 3 Extension, and the first constructed, was for the Halandri Station, located as shown on Fig.1. The paper outlines the geotechnical site investigations performed in this area, describes the excavation supporting system, summarizes the pre-construction finite element analyses and presents the results of displacement monitoring performed during excavation. The actual maximum horizontal displacements were about one-third of those computed, despite the use of high-quality soil sampling and testing methods. The availability of the measurements of both horizontal and vertical displacements permitted determination of soil stiffness parameters by back-analysis. The magnitude and distribution with depth of soil stiffness values consistent with the measured displacements are presented.

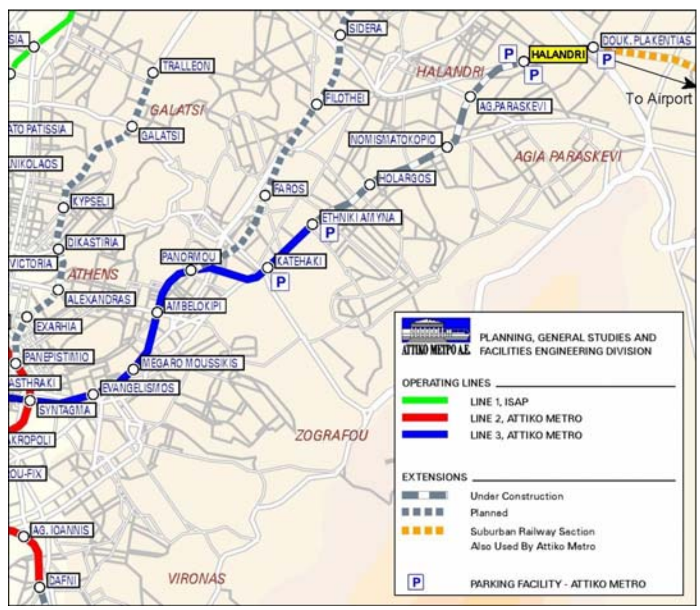

Fig.1 – Plan of Athens Metro Line 3 Extension

BACKGROUND

The Line 3 Extension of the Athens Metro, also known as the Stavros Extension, has a total length of approximately 5.8 km. Between the start of the extension at the existing Ethniki Amyna station and the new Douk. Plakentia station (Fig.1), the line is entirely underground in tunnels constructed by TBM (Tunnel Boring Machine) and NATM ("New Austrian Tunneling Method") techniques. In the vicinity of the Douk. Plakentia Station the line rises to ground surface and continues, merged with a suburban rail line, to the new Athens International Airport at Spata.

The excavation for the Halandri station was the first of several deep excavations for the Line 3 Extension, and the first in the Athens Metro system where stiff to hard sandy clays extended to well below the bottom of the excavation. In all the previously completed Athens Metro stations and shafts, either in the entire excavation or the bottom portion was typically founded in the Athens schist formation. Moreover, the 26 m deep excavation was one of the deepest to-date in the Athens Metro system. Hence, while much local experience was available for tunnels and open excavations in Athens schist, the Halandri site pioneered deep excavations performed entirely in the sandy clays. For this reason, the geotechnical investigation included state-of-art techniques, and a comprehensive instrumentation monitoring system was provided. This included the measurement of horizontal and vertical ground displacements both on the surface and at depth by means of inclinometers and sliding micrometers, as described below.

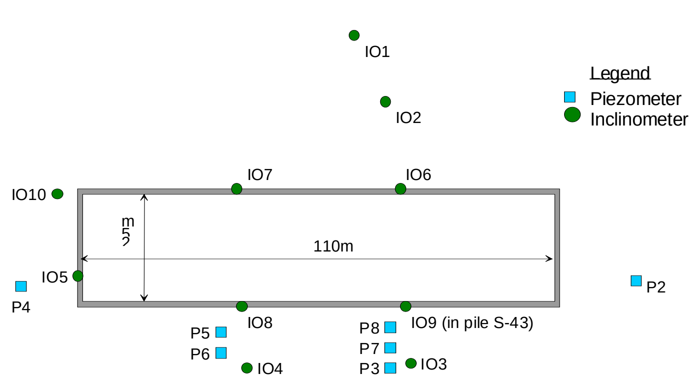

Fig. 2 – Plan of Halandri Station Excavation showing borehole instrument locations

Fig. 2 shows the plan view of the Halandri Station excavation. It was 110 m long by 25 m wide and 26 m deep. The temporary excavation retention system followed the typical design previously used for deep excavations for the Athens Metro. As shown on Figs.3 and 4, at the presently considered section it consisted of drilled RC piles 0.8 m dia. spaced 1.8 m on centers extending to about 6 m below the bottom of excavation. A reinforced concrete (RC) cap beam spans the top of the piles. The piles were supported against horizontal ground pressures by 7 rows of tie-back anchors, up to 33 m long, placed at vertical spacing of 3 to 3.5 m. The holes for the anchors were drilled through sleeves included in the pile reinforcement cage. A nominally 20 cm thick shotcrete layer supported on wire mesh anchored to the piles formed the face of the excavation

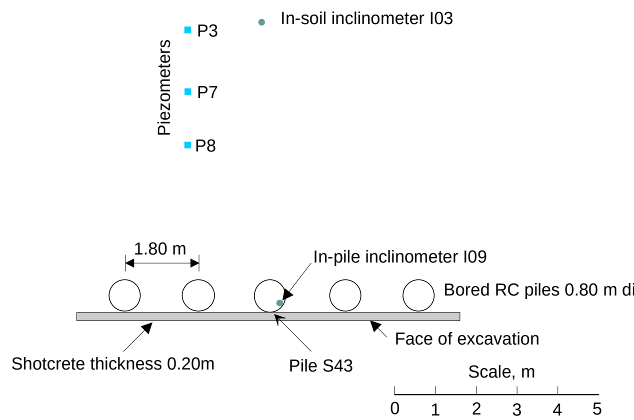

Fig.3 – Plan of retaining structure and borehole instrumentation in the vicinity of pile S-43

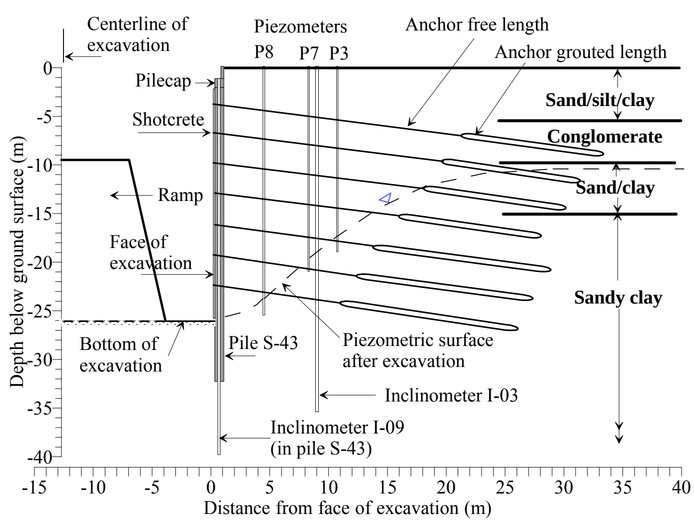

Fig.4– Cross-section of excavation and supporting system in the vicinity of pile S-43

GEOTECHNICAL CONDITIONS

A simplified soil profile is shown on Fig.4. Down to a depth of about 15 m from ground surface, the soils are variable, generally consisting of red-brown, very dense or hard sandy, clayey silts and silty clays, with zones of gravel in silty clay matrix. Layers of cemented material referred to as conglomerate are present at various depths, mostly in the top 12 m to 15 m. The plasticity index of the clayey portions of the soils in the top 13 m averaged about 10, and the moisture content was below the plastic limit. Below this depth, the material was more homogeneous, consisting mostly of red-brown, stiff to hard, broadly-graded sandy clay, with fewer gravelly or cemented zones. The plasticity index was generally in the range of 20 to 25 and the natural water content was near the plastic limit. This type of material extended to the maximum 40 m depth of boreholes in the area, so that the lower boundary of the sandy clay stratum was not known.

The groundwater table before the excavation was at a depth of about 10 m. This low level of the water table is believed partly responsible for the high strength and low compressibility of the upper parts of the soil profile.

Determination of the shear strength and compressibility parameters of the soil deposits was based partly on field testing, and partly on laboratory testing. In the top 12 m to 15 m, where gravelly and cemented zones existed so that undisturbed sampling was not practical, the parameters were estimated from the results of a series of Menard pressuremeter tests performed in one of the boreholes.

The normal method of soil drilling and sampling in the Athens area consists of rotary drilling in which continuous cores are recovered. The cores are placed in wooden core boxes, and portions designated for laboratory testing are wrapped in plastic. In order to obtain soil samples that have undergone minimum disturbance before laboratory testing, a triple-barrel Pitcher sampler was imported from Pitcher Drilling Co. of Palo Alto, California. The soil is cored directly into a thin-walled steel Shelby tube, and the ends of the tube are sealed with paraffin wax after removal of any loose material. A natural water content sample was taken in the field from the bottom of the Shelby tube. The use of this method was very successful in the stiff to hard sandy clays. The local drillers quickly adapted to this technique, consistently recovering high-quality relatively undisturbed samples. To avoid gravelly or cemented material, the borings with Pitcher sampling were located near previous boreholes performed by the continuous coring technique, which indicated the depths suitable for Pitcher sampling.

The locally unprecedented 26 m depth of excavation entirely in the sandy clays made it imperative to verify the stability and to estimate the magnitude of displacement of the excavation retention system prior to construction. It was therefore important to determine as accurately as possible the distribution with depth of the shear strength and compressibility parameters of the soils, including any strength anisotropy effects. It was also important to determine the magnitude and distribution of the earth pressure coefficient at rest K_o, since it has a significant effect on the ground pressure acting on the temporary excavation retaining system as well as on the permanent walls of the station. Further, K_o is an important parameter in the determination of the initial stresses in the ground in a finite element analysis of excavation. A state-of-art testing program specially designed to obtain the site-specific soil parameters was performed by GEOCOMP/GeoTesting of Boxborough, Massachusetts, USA. The testing was carried out on six of the Pitcher samples from depths of 14 m to 30 m, of which four were from the Halandri Station site and two, of very similar material, from the Ag. Paraskevi shaft site.

The six tube samples were shipped by air courier from Athens to Boston. Prior to opening the tubes, X-ray photographs were taken of each sample to determine the parts of the samples most suitable for laboratory testing.

Four constant rate of consolidation (CRC) tests were performed on four of the samples. These tests gave the compressibility of the soil, the preconsolidation stress and the coefficient of consolidation. CRC tests are better suited to the determination of the preconsolidation pressure than standard, stage-loading oedometer tests. The tests showed that the clay was overconsolidated by a factor of 5 at a depth of 15 m, the overconsolidation ratio (OCR) decreasing to 2.5 at a depth of 30 m. There is no geologic evidence of the overconsolidation having been caused by subsequently eroded formations, leading to the belief that the overconsolidation has been induced by desiccation.

Two triaxial K_o tests were run to determine the value of K_o as a function of the overconsolidation ratio using the Bishop technique. Test 1 was on a sample from a depth of 26 m at the Halandri site, and test 2 was on a sample from a depth of 24 m at the Ag. Paraskevi site. The two samples consisted of very similar soil.

The two tests gave similar results. They indicated the following relationship:

$$K_0 = 0.63\, OCR^{0.30}$$

where OCR is the overconsolidation ratio and equals the preconsolidation stress divided by the existing vertical effective stress. These two triaxial tests gave values of preconsolidation stress similar to those obtained from the CRC tests.

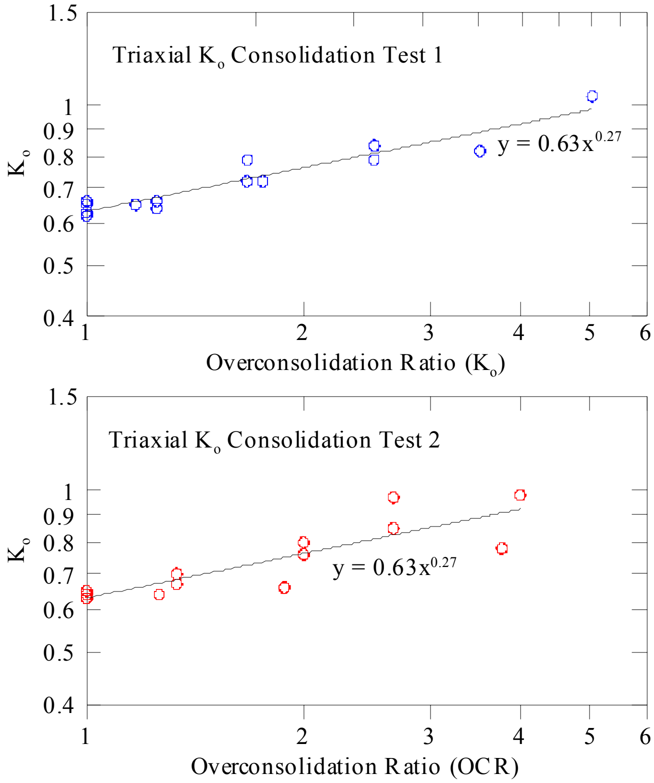

Fig. 5 – Results of Triaxial K_o Consolidation tests to determine the coefficient of earth pressure at rest K_o

Eight K_o consolidated undrained triaxial compression tests (TCK_oU) and two K_o consolidated undrained triaxial extension tests were run to determine undrained shear strength and stress-strain relationships for the clay. The results for extension tests were similar to those for the compression tests, indicating that the clay does not exhibit undrained strength anisotropy at the field stress conditions. The same undrained shear strength applies inside the excavation as outside the excavation.

The laboratory results were converted to field conditions using the recommendations of Ladd et. al (1999) to obtain the following relationship for field strength:

$$\tau_{ff} \quad = \quad 0.25\sigma'_{vc}(OCR)^{0.7}$$

For the effective stress strength parameters, the triaxial compression tests indicated an effective stress cohesion c' of 50 kPa and an effective stress friction angle φ' of 21^o in the stress range applicable to the analysis of the excavation. The two triaxial extension tests indicated a φ' of approximately 25^o with an assumed c' of 50 kPa.

Effective stress triaxial compression tests on very similar soil from the Douk. Plakentias site (Fig.1) were subsequently performed by Geognosi S.A. of Saloniki, Greece. These were TCD (triaxial consolidated-drained) and TCU (triaxial consolidated-undrained with pore pressure measurement) compression tests. The results of these tests are shown on Fig. 6 as plots of p'–q (mean principal effective stress vs mean shear stress). On the same plot are shown the above-mentioned results of the eight TCK_oU tests performed by GEOCOMP/ GeoTesting.

The results of all these tests are very consistent, and indicate a curved failure envelope, which, as shown on Fig. 6, can be approximated to bi-linear envelope. From Fig. 6, in the mean principal stress range of 0 to 540 kPa, the parameters are c' = 0, φ' = 27.5^o, and for mean principal stress above 540 kPa the values are c' = 130 kPa, φ' = 14^o.

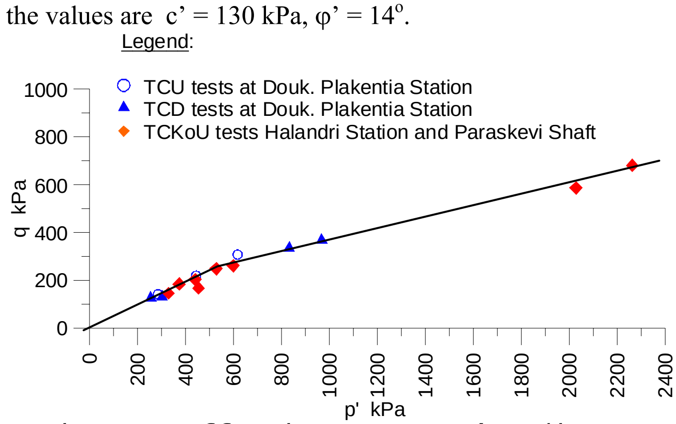

Fig. 6 - Effective stress p'-q diagram for sandy clay

The triaxial tests performed by GEOCOMP/GeoTesting also provided stress-strain data from which values of modulus could be determined. The results are shown on Fig. 7.

Based on the lab data, the E_{50} stiffness can be computed using:

$$E_{50}/\sigma'_c \quad = \quad 900\bullet(OCR)^{-0.95}$$

where E_{50} = secant modulus at 50% of maximum deviator stress (a reference value in PLAXIS finite element analysis – see below), σ'_{vc} is the vertical effective confining stress and OCR is the overconsolidation ratio.

For a hyperbolic stress-strain formulation these equations need to be modified to fit the requirements of the hyperbolic model, which was found to fit the site soil. They become:

$$s_{ult} \quad = \quad 0.26\sigma'_{vc}(OCR)^{0.7}$$

$$E_i/\sigma'_c \quad = \quad 1800\bullet(OCR)^{-0.95}$$

$$R = 0.95$$

where s_{ult} is the peak undrained shear strength, E_i is the initial tangent modulus and R is the failure ratio (Duncan and Chang, 1970).

These parameters assume that the clay remains undrained during the duration of construction of the excavation. This assumption was checked by computing consolidation times for the clay. Using coefficients of consolidation measured in the laboratory, the average strength loss due to consolidation of the clay during a six month construction period was estimated to be less than 1%. With a suitable factor of safety against global instability, this would be an insignificant loss of strength. Undrained strength parameters for the clay would control global stability for the anticipated 6 months excavation period.

As explained above, because of the impracticality of undisturbed soil sampling in the top 12 m to 15 m of the soil profile due to the presence of gravelly lenses and cemented zones, the geotechnical parameters in the upper part of the soil profile were determined taking into consideration the results

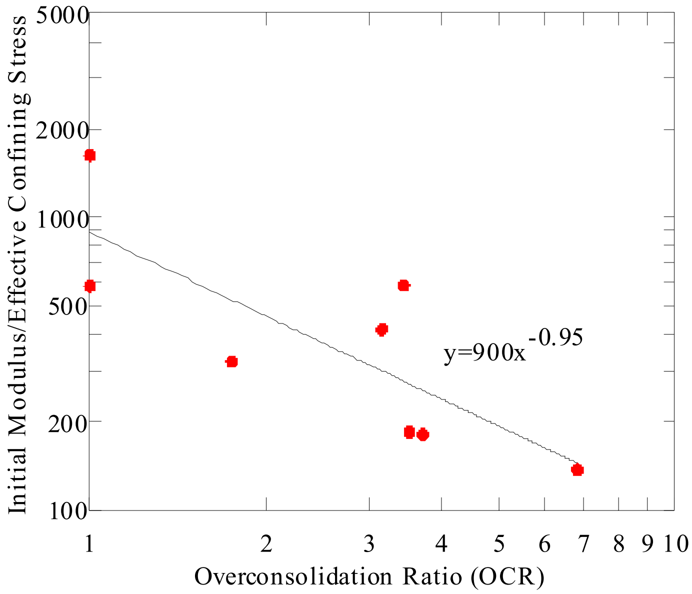

Fig. 7 – Initial Modulus vs Overconsolidation Ratio

of Menard pressuremeter tests. These tests, performed by Geognosi S.A., were interpreted in detail by Dr J. Hughes of Hughes In-Situ, Vancouver, Canada. The resulting strength, modulus and K_o values were considered in the selection of parameters for the finite element analyses described below.

Table 1 summarizes the source of parameters for each layer used in the soil model for the pre-construction finite element analysis. The unloading-reloading modulus E_{ur} was set equal to three times the E_{50} for each material. The primary layer controlling performance of the excavation is the clay below a depth of 15 m.

Based on measurements from laboratory testing, the total unit weight of all the soil layers was taken as 21.5 kN/m^3.

PRE-CONSTRUCTION FINITE ELEMENT ANALYSIS

Structural Model

The excavation and support of the cut-and-cover station box was modeled using the finite element program PLAXIS (Brinkgreve and Vermeer, 1998). As earlier described, the excavation support system was to consist of 800 mm diameter reinforced concrete piles placed at 1.5 to 1.8 m on-center spacing with a face covering of 200 mm of shotcrete. The wall was to be supported with seven rows of tiebacks at 3 to 3.5 m vertical spacing. The project structural engineers provided an estimate of moment of inertia for this geometry of 0.0314 to 0.0119 m^4/m with the variation depending on the quality of the bond between the shotcrete and piles. The lower value was selected as more representative of actual construction conditions. At the analyzed section, the tie-back anchor design loads were up to 800 kN.

Table 1: Source and Values of Parameters for FE Model in Pre-Construction Analysis

Layer	Depth m	Soil Model	Para-meter	Source
Sand/Silt/ Clay	0-6	Drained	c'=20 kPa ϕ'=40^o	Estimated using pressuremeter data
			K_o=0.5	Ditto
			E=90000 kPa	Ditto
Conglomerate	6-10	Drained	c=4000 kPa	Ditto
			K_o=1.0	Ditto
			E= 300,000 kPa	Ditto
Sand/Clay with gravel	10-15	Un-drained	s_u=400 kPa	Ditto
			K_o=1.0	Ditto
			E= 150,000 kPa	Ditto
Clay	15-50	Un-drained	s_u: See Note	Lab tests
			K_o: See Note	Lab tests
			E: See Note	Lab tests and Pressuremeter

Note: Below 15 m, the values varied in accordance with the above-given values of OCR and equations derived from laboratory testing. The distribution of modulus E_{50} with depth adopted in the pre-construction finite element analysis is shown on Fig. 8. Between 15 and 18 m it was taken as 216 kN/m^2, and between 18 m and bottom of model at 40 m it progressively increased from 193 to 230 kN/m^2 in accordance with the above-described relationship established from laboratory testing.

The stiffness of the shotcrete was estimated to be an E of 14*10^6 kPa after curing. For the pile concrete, the project design value of Young's modulus of $30*10^6$ kPa; however this value applies to high quality material with no cracks. Experience from other projects showed that construction of this type of piles generally does not produce the highest quality and the concrete develops cracks. For this reason, for the purposes of the analysis the concrete modulus was reduced to $15*10^6$ kPa. This value was used for both the pile concrete and the shotcrete. The analyses were made with a worst-case pile spacing of 1.8 m. This produces the most flexible wall and results in the highest tieback forces. These values result in EI of the wall of $0.285*10^6$ kN/m^2 and EA of $7.2*10^6$ kN/m. Poisson's Ratio of 0.25 was used for the wall.

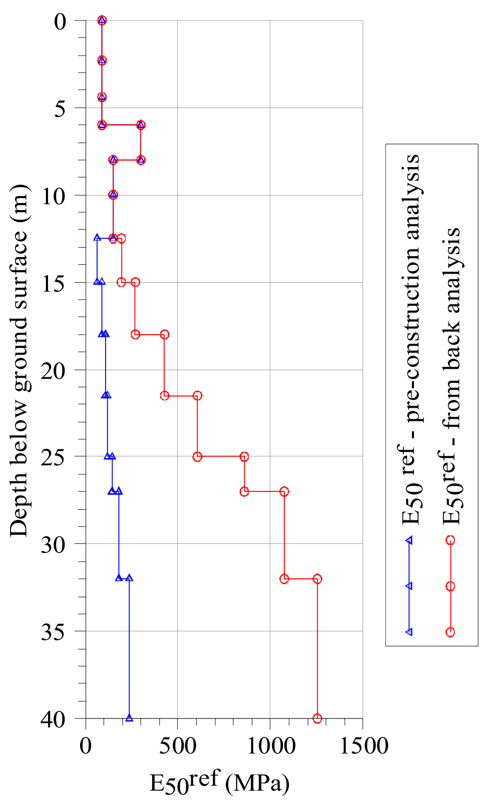

Fig. 8 - PLAXIS Reference Modulus vs Depth

As shown on Fig.4, the anchor system consisted of 7 rows of tiebacks with an unbonded portion and a bonded portion. The design anchor capacity was converted to a force per unit length of wall using an anchor spacing of 1.8 m. Using an E for the tie steel of $14*10^6$ kN and the area of steel, the 4 tendon anchors, used in the higher-levels of tiebacks, had an EA of 61,800 kN/m and the 5 tendon anchors, used in the lower levels of tiebacks, had an EA of 77,200 kN/m. The stiffness of the grout is negligible compared to the steel and was not included in this calculation. The unbonded portion of the tie is free to move separate from the surrounding soil. This behavior is best modeled in PLAXIS with a bar element. A bar element in effect places a spring between two nodes that couples the displacement of those two nodes together by the spring's stiffness. The soil between those two nodes is unaffected. The bonded portion of the tie interacts with the surrounding soil. Once the shear stress at the perimeter of the bond exceeds the shear strength of the soil, the bond will begin to slip. This is best modeled in PLAXIS with a "geotextile" element. This element is misnamed for this application. It really is a plane element with unit thickness that can only take tension. To permit the adjacent soil to slip along the bond, interface elements were added to both sides of the geotextile element. This combination produces a plane that ties both ends of the bonded portion of the tie with a spring but allows slip along the plane if the displacement of the geotextile element exceeds that required to fail the adjacent soil.

The combination of bar elements and geotextile elements produces a model of each tie where the force in the tie is constant along the unbonded portion and decreases along the bonded portion with distance away from the wall. This is a correct model of actual behavior. The PLAXIS analyses were performed assuming that each tieback would receive a prestress load of 80% of its design capacity.

Analytical Model

In the PLAXIS analysis, a two dimensional (plane strain) model was employed. The strain hardening (HS) soil model was chosen for all the soil layers. This model employs a hyperbolic stress-strain relationship, taking into consideration the typically non-linear stress-strain properties of soils. It is based on the Duncan-Chang model (Duncan and Chang 1970) but is defined within the elasto-plastic framework. The strains are calculated using stress-dependent stiffness, with E_{50} defines as reference stiffness modulus for the first loading (stiffness at 50% deviator stress at failure) and $3E_{50}$ for unloading/reloading. E_{50} determines the magnitude of both the elastic and the plastic strains.

In combination with a Poisson's ratio v_{ur}, the elasticity modulus E_{ur} determines the soil behavior under unloading and reloading, where the subscript ur stands for "unloading/reloading". The secant virgin loading modulus E_{50} and hence the unloading modulus E_{ur} are dependent on the stress level. Failure yield criterion is defined by Mohr-Coulomb parameters; effective cohesion c', effective angle of shear resistance φ' and shear dilatancy angle ψ. The above-described laboratory test data fit a hyperbolic stress-strain relationship very well

The sequence of construction for an excavation affects the deformations that occur. For the deformation calculations to be representative it is important that the analysis sequence duplicates the most significant elements of the construction sequence. PLAXIS has a staged construction feature that permits the analysis to follow actual construction steps. Elements are removed to simulate excavation. Their removal involves removing their stiffness from the model and transferring any stress carried by the removed elements to the remaining elements. Elements may be added to simulate walls and tiebacks as the construction progresses. The following sequence was used in the pre-construction analyses (It was assumed that groundwater table behind the wall of the excavation remained at a depth of 12 m for all stages of the analysis):

- compute initial stresses for existing conditions with water at -12m, using the soil unit weight and values of K_o determined from the testing program.
- activate the wall for its full depth
- excavate to 4m depth
- activate Level 1 tieback and prestress
- excavate to 7m depth
- activate Level 2 tieback and prestress
- excavate to 10m depth
- activate Level 3 tieback and prestress
- excavate to 13m depth and lower water table inside excavation to 14m
- activate Level 4 tieback and prestress
- excavate to 16.5m depth and lower water table inside excavation to 17.5m
- activate Level 5 tieback and prestress
- excavate to 20m depth and lower water table inside excavation to 21m
- activate Level 6 tieback and prestress
- excavate to 23m depth and lower water table inside excavation to 24m
- activate Level 7 tieback and prestress
- excavate to 27m depth and lower water table inside excavation to 28m

The maximum horizontal displacements at the excavation wall, at the completion of excavation was calculated to be 112 mm. This occurs at the top of the wall. The maximum calculated moment in the wall was 488 kNm/m occurring at a depth of 25 m. The maximum vertical displacement (ground settlement) was calculated as 93 mm.

Following these steps a stability analysis was performed in PLAXIS to determine the factor of safety against a global stability failure. PLAXIS does this by incrementally decreasing the strength parameters of each soil by a constant until the deformations become very large. The inverse of the strength reduction constant equals the factor of safety against global stability failure.

The global stability analysis in PLAXIS gave a factor of safety of 1.45. It showed a failure mechanism consisting of an active wedge from the toe of the wall through the soil behind the wall and a passive wedge from the bottom of the wall inside the excavation.

The analysis indicated that the factor of safety in the four lowest levels of anchors would be above unity but below 1.5. To verify tie-back behavior, load cells were installed on 12 of the anchors to monitor the actual loads, and all the anchors were proof-tested to verify the capacity.

EXCAVATION AND CONSTRUCTION OF THE SUPPORTING SYSTEM

The excavation progressed in stages, determined by the levels of tieback anchors. The soil conditions encountered in the excavation generally confirmed the soil profile shown on Figure 4.

The conglomerate layer required mechanical breaking to assist ripping operation. The excavated material was trucked from the bottom of the excavation over the ramp shown on Fig.4. The very steep sides slopes of the ramp were factual, permitted by the high strength of the soils. The walls of the excavation were covered with sprayed shotcrete as the excavation progressed, after installation of each row of anchors. All the anchors were proof-tested in accordance with the applicable DIN standard. Readings of the load cells installed on 12 of the anchors showed that the actually developed lock-off loads ranged from 50% to 98% of the design load, averaging 77%. Starting with the fourth row, some of the anchors did not sustain the design load or did not satisfy the proof-testing displacement criteria. In such locations additional anchors were installed. The inadequate performance of some of the anchors was attributed to defects during installation. The ramp was removed after the excavation was complete.

INSTRUMENTATION AND MONITORING

Fig. 2 shows the locations of borehole instruments installed for the monitoring of the excavation and its supporting system, and of the areas outside that could be affected by the excavation. The presence of a major traffic artery diverted around the north side of the excavation necessitated the placement of most of the borehole instruments on the south side, as shown on Fig. 2.

There were a total of 10 inclinometers, of which 5 were placed inside piles. Combined inclinometers/sliding micrometers were installed behind the excavation, permitting the measurement of both horizontal and vertical displacement profiles.

A total of 7 vibrating wire piezometers sealed at several levels provided an indication of the ground water drawdown curve caused by the excavation.

Load cells installed on 12 of the tie-backs provided information on the loads carried by the anchors.

Surface instruments consisted of 3-dimensional optical targets placed on the top of the pile cap beams spaced 15 m to 20 m apart, on some of the nearest building, and on the concrete center divider of the diverted highway. Also, surface settlement points were installed along the highway curbs immediately north of the excavation, and on nearby buildings.

The vibrating wire piezometers permitted the monitoring of the drawdown of the piezometric surface as the excavation progressed, without a time lag that may have occurred in these low-permeability soils had standpipe piezometers been employed. The piezometric surface at the end of excavation, interpreted from the piezometric readings, is indicated on Figure 4.

The left graph of Fig. 9 presents the profiles of horizontal displacements measured in the inclinometers I-09, located in

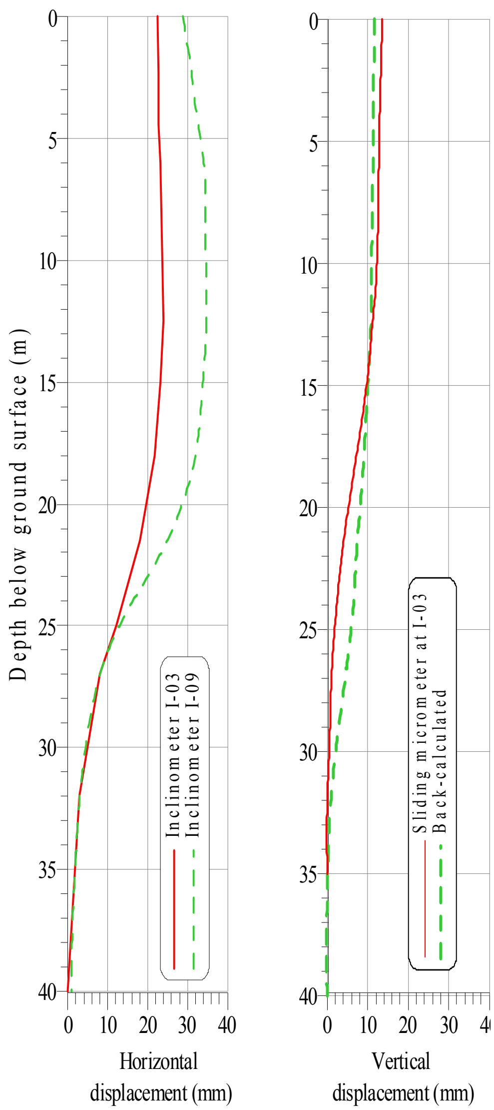

Fig. 9 – Horizontal and vertical displacements in inclinometers I-09 (in pile S-43) and I-03 (7 m behind piles

pile S-43, and I-03, located 7 m behind the piles. The measured horizontal component of displacement towards the excavation of 30 mm was approximately the same as displacements of 31 mm and 28 mm measured at the nearest optical targets located on top of the pilecap beam, and 28 mm measured at a small building located about 5 m behind the excavation. The vertical displacement of 13 mm measured at I-03, shown in the right graph of Fig. 9, is similar to the 12 mm settlement measured at the above building.

The three remaining inclinometers installed in piles, located as shown on Fig. 2, had provided horizontal displacement profiles very similar to those shown for I-09. The maximum displacements were in the range of 25 mm to 30 mm, occurring at a depth of 14 m to 17 m.

Fig. 10 shows horizontal displacement profiles measured in inclinometers installed in excavation-supporting piles at the Ag. Paraskevi and Nomismatokopio Shafts. The locations of these sites are shown on Fig. 1. The soil conditions at these sites are very similar to those at the Halandri Station. The somewhat smaller displacements may be partially caused by smaller plan dimensions of these excavations. However, the distribution of the displacements with depth is very similar, with maximum displacement occurring at 14 m to 17 m depth. These observations show that the general uniformity of the soils, indicated by laboratory testing, is reflected in the similarity of ground response to excavation at these sites.

The plots show that most of the horizontal as well as vertical displacement has developed below about 15 m depth. This is consistent with the above-discussed high strength and stiffness of the upper layers of soil. The soil mass in the top 12 m to 15 m in effect is acting as a rigid mat, experiencing very little internal strains, and moving horizontally and vertically as a rigid block. The recorded movements were mostly due to the displacements occurring in less stiff materials existing at greater depths. Most of the horizontal and vertical movement developed between the depths of 17 and 25 m. This behavior of the site soils was favorable for the construction of the Metro tunnels.

Measurements along the tunnel alignment passing through this soil type showed that surface settlements rarely exceeded 10 mm and were uniform.

As indicated above, the measured displacements had a similar distribution with depth as those obtained from the pre-construction analysis, but were significantly smaller. However, in comparing the measured displacements with those calculated in the pre-construction analysis, several factors need to be considered. Firstly, based on the low permeability of the soils, the piezometric surface was assumed to remain at 12 m depth throughout the excavation, while in actual fact, a significant drawdown has taken place. The calculated horizontal displacements would have been significantly smaller had this drawdown been applied in the analysis. Secondly, the actual depth of excavation was 26 m, while 27 m was assumed during the design stage. The stepped analysis has shown very significant increase in displacements during the final stages of the excavation. Thirdly, the presence of the ramp throughout the excavation period reduced the tendency for bottom heave, which reduced the displacements of the ground behind the wall. Fourthly, the bottom part of the excavation was performed in sections, with the pouring of the bottom foundation mat following soon after the completion of excavation at each section. This, similarly as the presence of the ramp, reduced the tendency for bottom heave.

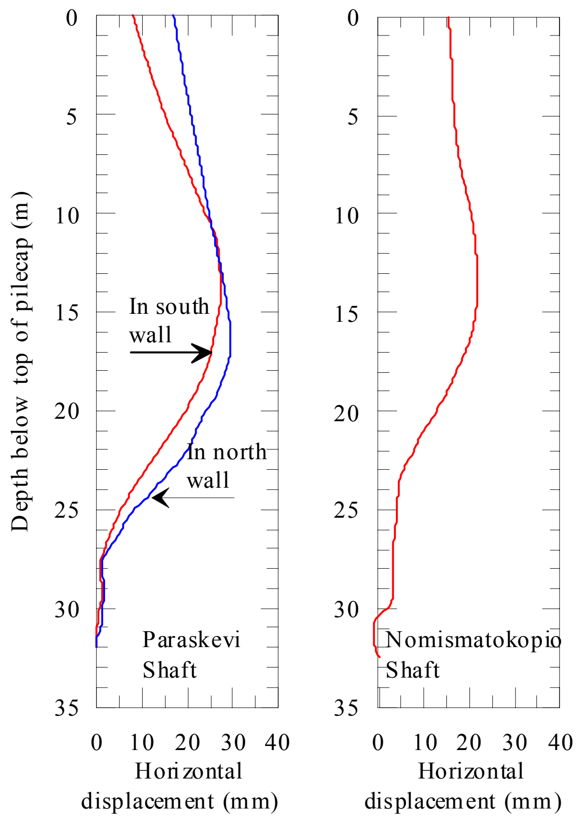

Fig. 10 – Horizontal displacement profiles at the Ag. Paraskevi and Nomismatokopio Shafts

BACK ANALYSIS OF CROSS SECTION AT PILE S-43

The availability of the distribution with depth of both horizontal and vertical displacement measurements provided a unique opportunity for a back-analysis. The cross-section selected for this analysis was in the vicinity of pile S-43. In addition to the borehole instruments shown on Figs. 3 and 4, there were optical targets on the pile cap and on the shotcreted excavation wall, and load cells on two of the adjacent piles.

The back-analysis was performed using the PLAXIS program. The input soil stiffness parameters were varied to achieve as close as practicable agreement between the calculated and the measured displacements.

The total width of the finite element model was 150 m and the depth was 60 m. The model contained 3394 6-noded elements. The piles were modeled as beams with a flexural rigidity EI = 285×10^3 kN/m^2 and a axial stiffness of EA = 7.2×10^6 kN/m, with Young's Modulus of E = 30.0 GPa. Examination of the piles exposed by the excavation showed them to be sound with no observable defects, so that this design value of the modulus was adopted for the back-analysis.

Anchors were modeled with two elements. The fixed part of an anchor was modeled with a slender element, defined only by normal stiffness, while the anchor rod was modeled by the two node elastic spring element with constant spring stiffness.

Due to the nature of the encountered materials (clayey soils with permeability ranging from 10^{-7} to 10^{-8} m/s) and the relatively short period (less than 6 months) available for excavation and support works, undrained conditions were assumed to prevail during construction.

In order to limit the number of variables, and in view of the fact that, in strong soils, the displacements are controlled much more by soil stiffness than by soil strength, the strength parameters used in the pre-construction analysis described above were retained, while the stiffness of the soil was varied. This was justified by the known fact that, in laboratory testing, soil stiffness is much more sensitive to sample disturbance than soil strength. Further, in the present case the shear strength properties were fairly well defined by the laboratory testing supported by pressuremeter tests and were quite consistent. The adopted shear strength was as shown in Table 1. The total unit weight for all the layers was taken as 21.5 kN/m^3.

The analysis was performed in similar multiple steps as listed above for the pre-construction analysis. The ground water table was progressively lowered at each excavation step using estimated curves of the drawdown of the piezometric surface,, reaching the level shown on Fig. 4 at the last step of excavation.

Initially, the modulus values used for the different layers in the preconstruction analyses, shown on Fig. 8, were increased by a single factor. However, to further improve the agreement between the measured and calculated displacements, the modulus was varied in accordance with the distribution of E_{50}. The values of the remaining stiffness parameters used in PLAXIS, namely the oedometer modulus E_{oed} and the unloading-reloading modulus E_{ur} which were assumed to be proportional to E_{50}, were varied accordingly.

The input stiffness parameters were varied in an iterative process with an aim to provide the best possible agreement between predicted and observed deformations of the wall. Finally, a single set of parameters was selected, giving the best agreement between the two. Care has been taken that the parameters are realistic.

Fig. 11 compares the measured and back-calculated displacements at inclinometers I-09, located in pile S-43, and I-03, located 7 m behind the piles, as shown in Figs. 2, 3 and 4. The iterative analysis focused on achieving a reasonable agreement between the measured and calculated displacements in the wall pile inclinometer I-09. As shown on

Figure 11, the two displacements are very close to a depth of 15 m, below which the calculated displacement exceeds the measure. The trend below 25 m depth suggests that below the bottom of excavation the soil has an even higher stiffness than was assumed. There was some indication of this in small pits locally excavated below the bottom of the general excavation. For the inclinometer I-03 located 7 m behind the piles, the calculated displacements exceed the measured ones by up to 50% above a depth of 14 m, suggesting that the stiffness of soil in the top 10 m or so is even higher than was assumed. However, below about 20 m depth, the calculated displacements are significantly lower than measured below about 20 m depth range, and underestimate the displacement below this depth.

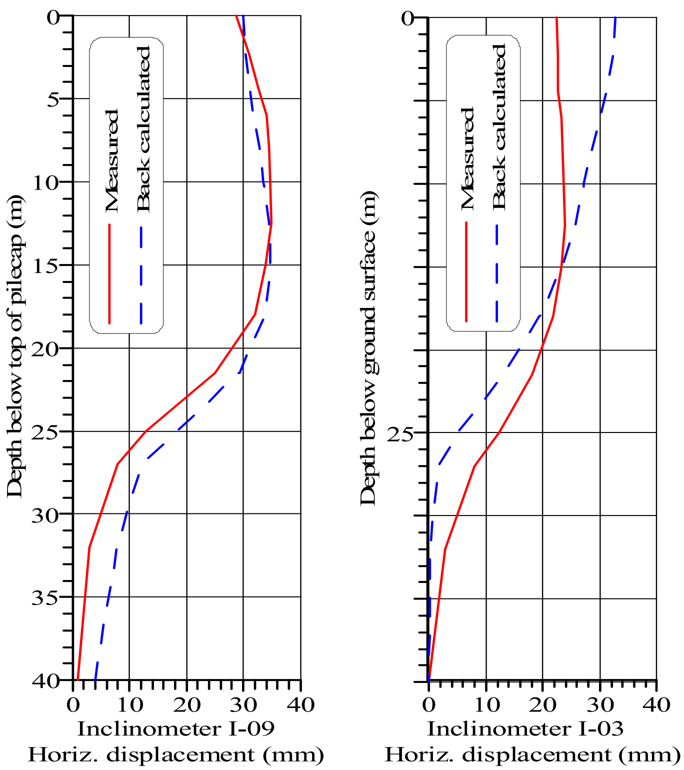

Fig. 11 - Measured and back calculated horizontal displacements at inclinometers I-09 and I-03

In the back-analyses, to minimize the number of variables, the modulus in the horizontal direction was taken equal to that in the vertical direction. The inconsistency in the relationship between the measured and calculated displacement at inclinometer I-03 may be caused by depth-dependent stiffness anisotropy of the soil.

A comparison between the measured and back-calculated vertical displacement profiles at the location of inclinometer I-03 equipped with sliding micrometer is shown on Fig. 9. There is excellent agreement from ground surface to a depth of about 14 m, below which the calculation overestimated the vertical displacement. As mentioned earlier, this may be caused by soil stiffness below 15 m even higher than was assumed.

CONCLUSIONS

1. Instrumentation that included inclinometers installed in piles supporting the excavation, and sliding micrometers measured the displacement profiles for the excavation supporting system and adjacent areas. The results were consistent, indicating uniformity of engineering properties of the soils, which resulted in the observed relatively high degree of uniformity of soil response to excavation.
2. Similarity of the horizontal displacement profiles at several sites along the Athens Metro Line 3 Stavros Extension located in the stiff to hard sandy clays indicated that the back-calculated soil parameters may be applicable to a preliminary estimate of ground displacements caused by future excavations in similar soils.
3. Despite state-of-art soil sampling and laboratory testing techniques and advanced analytical methods, the pre-construction analysis overestimated the horizontal displacements by a factor of about three, and the vertical displacement by a factor of six. However, the small vertical displacement is likely caused by the very stiff soils in the uppermost 12 to 15 m of the soil profile, which act as a rigid mat. Note, from Figs. 9 and 11, that the uppermost 15 m to 17 m of soil moved both horizontally and vertically almost as a rigid block, with little horizontal and vertical strains within the block.
4. Possible reasons for the over-estimate of displacements in the pre-construction FE analysis include (1) a general difficulty of determining the strength and compressibility parameters for hard desiccated soils; (2) lack of information on the stiffness anisotropy of the soil; (3) not taking into account the negative pore water pressures that were likely present in the 10 m to 12 m thick soil zone located above the water table; and (4) conservatism in selecting the soil parameters to predict excavation performance for design purposes that would not be exceeded in construction.

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