Deep Supported Excavation in Difficult Ground Conditions in the City of Patras, Greece — Measured vs. Predicted Behavior

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DEEP SUPPORTED EXCAVATION IN DIFFICULT GROUND CONDITIONS IN THE CITY OF PATRAS, GREECE - MEASURED VS. PREDICTED BEHAVIOR

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ABSTRACT

The technical characteristics of a deep supported excavation project using anchored diaphragm walls and the measured (by inclinometers) behavior of the soil retaining system are presented. The measured behavior is then compared with the predicted behavior using a finite element model of the excavation. The comparison shows a good agreement in a location where the soil profile is well defined. However, differences in the magnitude of the displacements were observed when the information on the soil profile was incomplete due to the variability of the deposits on site.

INTRODUCTION

The deformation of ground masses surrounding deep supported excavations constitutes a subject of increasing interest to soil engineers. When these excavations are conducted in the urban environment (Gould et al., 1992) it is of outmost importance to be able to predict the resulting ground movements, to compare them to the allowable values and evaluate the possibility of causing damage to nearby structures (Boone, 1996). The reliable prediction of such ground movements has to overcome, among other things, the difficulty of the inherent variability of soil conditions (geometry and mechanical properties) and the incomplete knowledge that a soil engineer can obtain from a limited number of boreholes. Therefore, well-documented case histories of instrumented deep supported excavations (Whitman et al., 1991) can provide significant insight into the mechanics of the problem and be used as a guide in the design of future projects (Whittle et al., 1993, Hashash and Whittle, 1996, Koutsoftas et al., 2000, Leonidou et al., 2001).

Clough and O'Rourke (1990) made a comprehensive presentation of experience gained by previous excavations. These investigators used the measured data to establish recommendations for the evaluation of the expected lateral and vertical deformations of the wall and ground for different soil conditions, retaining systems and depth of excavation. More recently, Long (2001) presented an extensive database of case histories of wall and ground movements due to deep excavations worldwide. The database was used to reassess the guidelines proposed by Clough and O' Rourke (1990) and to provide help to geotechnical and structural engineers in the preliminary design work of deep ground excavations.

In this paper the case of a deep supported excavation performed in difficult ground conditions is presented. The case history includes the results of measurements of lateral wall movements during the excavation as well as the results of finite element analyses. By comparing the measured vs. the predicted behavior conclusions are drawn regarding the capability of numerical analyses to be used in the design of deep supported excavations.

DESCRIPTION OF THE PROJECT

The site of the project presented in this paper is located in the southern part of the city of Patras, Greece, near the old railway station of St. Andreas, on the coastal road, Fig. 1. A currently abandoned alcohol production factory was located in the site for many years. Development consisted of the construction of a three-story entertainment complex (which included movie theatres, restaurants, and other entertainment services) and a two-story underground parking garage for the visitors of the complex. Masonry storage buildings, along the south side of the site, were considered of historical importance and needed to be protected and developed. The deep excavation works lasted approximately four months, from May to September 2000. The excavated area had a rectangular shape with dimensions 126.4m by 62.8m, Fig. 2.

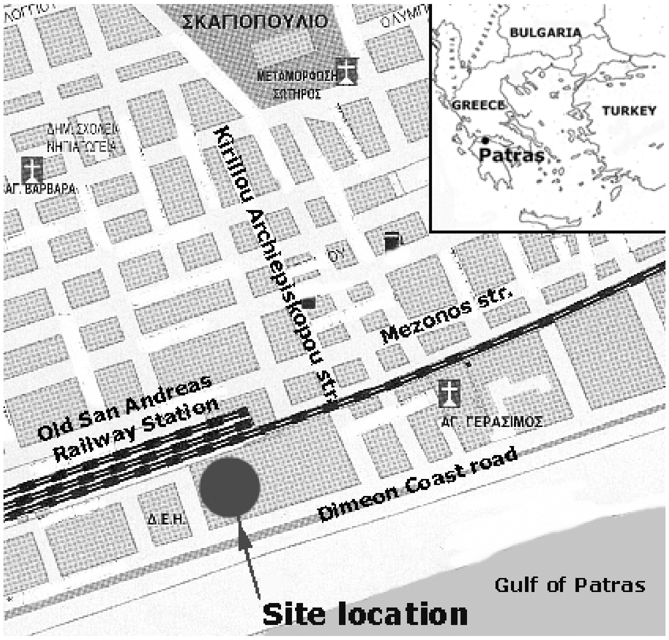

Fig. 1. Part of the map of the city of Patras with the location of the excavation site.

GEOTECHNICAL DATA

The geotechnical investigation consisted of drilling three boreholes with soil sampling up to a depth of 20.0m, and conducting laboratory testing, Fig. 2. The soil profile consisted of alternating layers of silty-sandy Clay (CL) and silty-clayey Sand (SC) with varying thicknesses. These layers were underlain by a sandy Gravel (GC) layer, encountered at a depth ranging from 13.0 to 17.5m. The investigation indicated that there was significant variability in the relative depths and thicknesses of the soil units, Fig. 3. The location of the water table, measured during the investigation and a month later, had a maximum depth of approximately 2.0m below the ground surface. Artesian pressure yielding $1m^3$/hr of water was measured in borehole B-1.

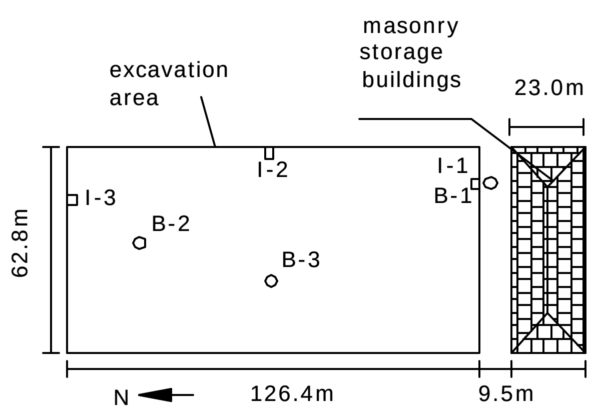

Fig. 2. Plan view of the excavated area with the location of boreholes indicated with a circle and of the inclinometers with rectangles.

DESCRIPTION OF THE EARTH RETAINING SYSTEM

An excavation depth of 7.5m was required for the operation of the underground two-story parking garage. Anchored, reinforced concrete diaphragm walls were used to retain the deep excavation and the decision to use them was based mainly on the minimization of the effect of the excavation to the nearby masonry structures and on the availability of equipment. The total height of the slurry walls was 13.0m and their thickness 0.60m. At a depth of 2.2m the wall was tied by 3 prestressed steel S1670/1860 tendon anchors with a diameter of 0.6'', having a total length of 19.5m and a grouted length of 13.0m. The horizontal spacing of the anchors varied from 1.7 to 2.2m. The anchors were prestressed to forces of 173kN/m to 232 kN/m based on the results of three in-situ prestressing tests.

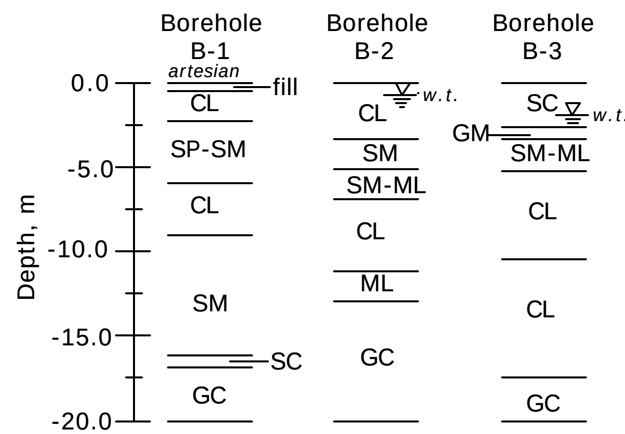

Fig. 3. The soil profiles at the location of the three boreholes indicate significant variability in the depth and thickness of each soil unit. The water table is also indicated (in borehole B-1 artesian pressures were measured).

Prior to the excavation, a permanent system of pumping wells was installed and the water table was drawn down to a depth of 20.0m. This significant water table lowering was considered necessary to avoid the occurrence of bottom blowout due to high artesian pressures.

The construction of the diaphragm walls was made in panels 6.0m wide, along the perimeter of the excavation. Construction of each panel was performed in the following stages:

1. Construction of guide walls to assist in the trench excavation of the diaphragm walls.
2. Excavation of the trench in segments, using a grab auger to a depth of 13.0m and bentonitic fluid to support the trench during the excavation.
3. Placement of the steel reinforcement cage of each panel in the trench. The steel cage is manufactured on site, lifted and placed in the trench using cranes.

4. Concrete pouring using two tremie pipes for each excavated panel.
5. Installation to a depth of 2.2m, upon concrete curing.
6. Construction of the anchors through the diaphragm walls at the locations indicated in the design.
7. Prestressing of the anchors.
8. Excavation to a final depth of 7.5m

The ICE manual (1997) specifications were followed in the construction of the guide walls and the properties of the bentonitic fluid. Further details and photos on the excavation process are given by Zeccos (2001), which is also available online by the Geoengineer website at http://www.geoengineer.org . The procedure was initiated at the western part of the site and extended to the East, in a way that the above stages could take place at the same time in different areas along the project reducing the construction time.

MEASURED BEHAVIOR

To evaluate the behavior of the diaphragm walls during the excavation, three inclinometer casings (I-1, I-2, I-3) were embedded in the diaphragm walls by fastening them to the reinforcement cages before concrete pouring in the locations indicated in Fig. 2. Inclinometers I-1, I-2 and I-3 were installed in the southern, eastern and northern side of the excavation, respectively. Unfortunately, the access ramp to the bottom of the excavation was located in the vicinity of the inclinometer I-3 and resulted in acquiring meaningless data, which will not be presented here.

At each inclinometer location, measurements were taken at three different stages of construction:

- Initial conditions, after the construction of the slurry wall and prior to any excavation.
- Excavation to a depth of 2.2m, prior and/or after the prestressing of the anchors
- Final stage of excavation to a depth of 7.5m.

The measurements in the intermediate stages showed very small deformations, which are also within the accuracy of the equipment. The maximum deformations occurred after the excavation reached the maximum depth of 7.5m and are shown in Fig. 4 & Fig. 5 for inclinometers I-1 and I-2, respectively.

FINITE ELEMENT ANALYSES

Finite element analyses were performed to estimate the expected displacements of the wall and compare them with the behavior measured by the inclinometers. For this purpose, the commercially available Finite Element Program PLAXIS was used (Plaxis, 1998). For each inclinometer location, both the in-situ soil conditions, and the construction procedure (staged construction) were simulated.

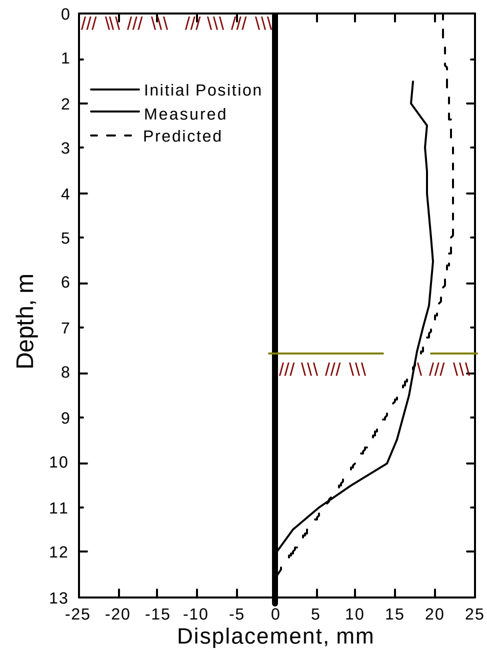

Fig. 4. Measured and predicted displacements at the location of inclinometer I-1.

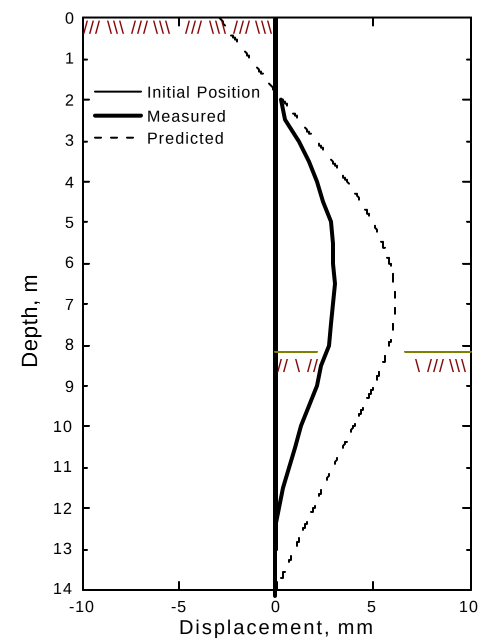

Fig. 5. Measured and predicted displacements at the location of inclinometer I-2.

Inclinometer I-1 was located very close to borehole B-1 (Fig. 2) and thus the soil profile was relatively well known. The finite element model incorporated the layering and properties of each unit based on the soil profile, the SPT blow count and the laboratory data, as shown in Table 1. The stress-strain behavior of soil materials was described using an elastoplastic model and the Mohr-Coulomb failure criterion. The plane strain finite element model used in the analysis is illustrated in Fig. 6. The FE mesh consisted of 664 15-noded elements, and had a height of 25.0m and a length of 135.0m.

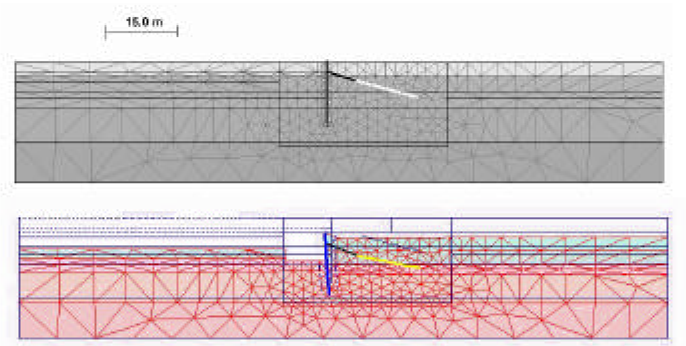

Fig. 6. a) Finite element model for the analysis of soil stability at the location of inclinometer I-1.
b) Deformed shape of the mesh at the final stage

Table 1. Soil Profile and layer properties used for the location of inclinometer I-1

Layer	Soil	γ_d (kN/m^3)	E (kPa)	ν	c (kPa)	Friction angle, φ
1	Clay1	16.0	5000	0.35	25.0	0
2	Sand1	18.0	20000	0.3	3.0	30
3	Clay2	17.0	5000	0.35	20.0	0
4	Sand2	18.0	20000	0.3	3.0	30
5	Sand-gravel	19.0	40000	0.3	2.0	35

The predicted deformations by FE analysis are also shown in Fig. 4. The predicted and the measured behavior were similar in shape and magnitude, as summarized in Table 2. The shape of the deformation of the wall indicates that the tiebacks did not "hold" the wall too well. However, the performance of the tiebacks is considered adequate since the predicted deformation of the wall without the tiebacks was much greater and the resulting deformations did not have any effect on the masonry buildings.

Table 2. Comparison of measured and predicted behavior for the location of inclinometer I-1

	Measured	Predicted
At the anchor's level (depth –2.1m)	17.6mm	21.8mm
Maximum Displacement	19.6mm (in depth –5,5m)	22.4mm (in depth –4.0m)

In the location of inclinometer I-2, the soil profile was not known very accurately and had to be inferred based on (i) the exploratory boreholes B-2 and B-3, located at approximately equal distances (≈40.0m) from the inclinometer, and (ii) the observations made during the excavation by the grab auger. The soil profile and properties are listed in Table 3. Of interest is the existence of the sand-gravel layer at a depth of 3.0m. This layer seems to be responsible for the greater resistance of the tiebacks in this side of the excavation. This fact is also clearly observed in the deformed shape shown in Fig. 5.

The FE mesh consisted of 538 15-noded elements, and had a height of 25.0m and a length of 101.0m. The deformed mesh of the plane strain finite element model used in the analysis is illustrated in Fig. 7. The predicted deformations of the FE analysis at the location of inclinometer I-2 are also shown in Fig. 5. Even though the shape of the measured and the analytically predicted behavior is similar and relatively small, the maximum displacements predicted in this case are almost double the measured ones. The results for the location of inclinometer I-2 are summarized in Table 4.

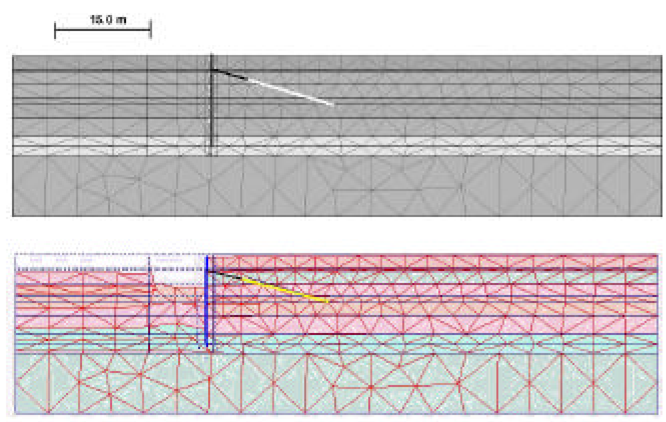

Fig. 7. a) Finite element model for the analysis of soil stability at the location of inclinometer I-2.
b) Deformed shape of the mesh at the final stage

Table 3. Soil Profile and layer properties used for the location of inclinometer I-2

Layer	Soil	g_d (kÍ/m^3)	Å (kPa)	í	c (kPa)	Friction angle, ö
1	Fill	17.5	15000	0.3	10.0	20
2	Sand-gravel1	19.0	40000	0.3	2.0	35
3	Sand1	18.0	20000	0.3	3.0	30
4	Clay1	17.0	7000	0.35	20.0	0
5	Clay2	16.0	5000	0.35	25.0	5
6	Sand-gravel2	19.0	40000	0.3	2.0	35

Table 4. Comparison of measured and predicted behavior for inclinometer I-2

	Measured	Predicted
At the anchor's level (depth –2.1m)	0.317 mm	0.498 mm
Maximum Displacement	3.06mm (in depth –6.5m)	6.16mm (in depth –7.0m)

DISCUSSION AND CONCLUSIONS

The performance of the tiebacks in the present project was adequate for all practical purposes. However, the two different patterns of deformation at the location of inclinometers I-1 and I-2 suggest that the anchors provided more resistance in the case of the latter location. The increased resistance could be attributed to the existence of the gravel layer at the depth of 3.0m, which was observed during the excavation process and can also be inferred by the boreholes.

For the location of inclinometer I-1, where the soil profile was known, the predicted and measured deformations were similar in shape and magnitude, suggesting that the Mohr-Coulomb criterion can provide a fairly accurate prediction of the behavior for this type of projects and soil conditions. For the location of inclinometer I-2, the site conditions had to be inferred and the predicted behavior is similar to the real one in shape but is approximately twice as much, though still relatively small. The less accurate prediction in this case must be attributed to the great variability of the soil conditions making difficult to know the actual soil profile in the location of inclinometer I-2. The significant differences in the soil profiles (Fig. 3) among the boreholes illustrate this variability.

For both soil profiles, the pattern of the deformation was insensitive to changes in soil material properties. On the other hand, the magnitude of the deformation was influenced by the assigned properties of soil units, although not significantly.

Clough and O'Rourke (1990) suggested that for stiff clays, residual soils and sands, there exists a linear relation between the maximum lateral wall movement and the depth of excavation. More specifically, the horizontal movements tend to average about 0.2% of the height H of the excavation. However, as the above researchers have stated, there is an ample scatter in the data and that was also observed by Long (2001). In the diagram of Fig. 8 the data points derived from this project are compared to the average linear relationships recommended by Clough and O'Rourke (1990) for 0.2% and 0.5% lateral wall movement. The two data points are in good agreement with the mean recommended values. Interestingly, the average of the two points falls close to the recommended 0.2% line, with one point being 50% above and the other point 60% below the recommended average. Taking into consideration that the construction method was the same for both data, this variability should be attributed to the variability of soil conditions. Long (2001) has also suggested that deformations greater than 0.3%H should not be taken into consideration when average curves of the type of Fig. 8 are produced since "these cases are likely to involve some particular site-related problem". Interestingly, in the location of inclinometer I-1 the horizontal deformation was 0.3% in spite of the high quality of the construction work. This suggests that at least 0.3% deformations can occur in high quality construction due to the inherent variability of the soils.

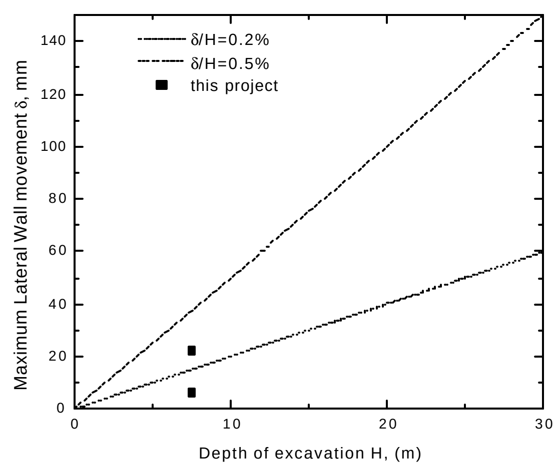

Fig. 8. The data from this project plotted in the linear space recommended by Clough and O' Rourke (1990).

Sites with significant variability in the geometric and mechanical properties of the soil units should be treated carefully. In this project, for the same construction technique, significant differences in the magnitude of deformations have been observed due to the variability of the soil conditions.

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Note

The data presented in this paper, as well as numerous photos from the construction are available by the Geoengineer website, at http:www.geoengineer.org. The authors are especially thankful to the construction site engineers but also to the construction workers who recognized the importance of collecting such data and provided much help. Thanks are also due to Antonis C. Tsitos , graduate student at the Dept. of Civil Engineering, University of Patras, for assisting in the preparation of the manuscript of the present paper.