1. INTRODUCTION

Economic growth in Egypt during recent decades has forced the construction industry to improve the quality of our urban environment, including moving services below the ground surface. Several large projects involving underground structures have been executed during the past 20 years. Most of these projects represent elements of an overall plan to implement the first two lines of the Greater Cairo Metro or replace the existing wastewater networks for the largest urban areas of Greater Cairo and Greater Alexandria (El-Nahhas, 1999). More active utilization of the underground spaces has started recently in order to solve traffic problems through the construction of road tunnels along with several underground car parking projects within the congested urban areas of Greater Cairo (El-Nahhas, 2003).

This paper is concerned with the case history of an underground multi-story garage, referred to as El-Tahreer Garage No. 2. This garage was constructed in El Tahreer Square near Omar Makram mosque, Cairo, over an area of about 5000 m² with four underground floors. The northern boundary of the garage faces the El-Sadat station of the First Line of the Cairo Metro. The nearest point of the El-Sadat station is about 6.2 m as depicted on the El-Sadat station drawings. This project represents one of two underground garages to be constructed at the El-Tahreer Square. Figure 1 shows the general layout of El-Tahreer Garage No. 2, while Fig. 2 shows the cross section of the El-Sadat Metro station.

The analysis and design of the supported deep excavations is considered one of the most difficult tasks facing geotechnical engineers. The effects of the construction procedure and the characteristics of the soil layers and groundwater should all be taken into consideration. A very important task is to address the structural effects of the garage construction on the El-Sadat Station. The first phase of the study was based on the results of a two-dimensional
finite element analysis conducted to model the interaction between the underground garage and the metro station. Implementation of in-situ monitoring programs during the construction of the supported deep excavations was necessary to verify the design assumptions and assure the safety requirements (Burland & Hancock, 1977; Kovari, 1984; El-Nahhas, 1987; Sakurai, 1987; El-Nahhas, et al., 1989; Finno, et al., 1989; Hansmire, et al., 1989; Urlich, 1989; Di Biango, 1991; El-Nahhas, 1992; Boone, et al., 1992; Sharma, et al., 2001; El-Nahhas & Morsy, 2002). Without in-situ monitoring and feedback analysis, it is not qualitatively possible to assess how conservative the design of the supported deep excavation is.

In this paper, the in-situ geotechnical performance of a 13.2 m deep multi-propped excavation in medium dense to very dense sand layers is investigated. Field measurements of the lateral wall deformations and groundwater levels were compiled using inclinometers, deep settlement points, and piezometers. The results of the in-situ monitoring are compared with those obtained from the finite element analysis. The case history presented herein focuses on the behavior of the soil interaction with the diaphragm wall and the general ground deformation regimes in the vicinity of the site.

2. THE SITE’S SUBSURFACE CONDITIONS

The geotechnical condition at the project site was determined from subsurface investigations prepared by (Ardaman Ace, 1999). The investigation comprised of 12 boreholes of a depth ranging between 48 and 50 m below the existing ground surface with field testing, including Standard Penetration Tests. Four boreholes were drilled at the site selected for this paper, and eight boreholes were located at the site of Garage No. 1, which is currently under construction adjacent to the Nile Hilton Hotel.

The soil formation at the site consists of surface man-made clayey fill (3.0 to 6.0 m thick) followed by a sand layer which extends to the end of all the boreholes. The particle size of the sand is fine to medium, with a relative density ranging between medium to very dense. A clay layer appeared at depths ranging between 30.0 to 39.0 m in all the boreholes sunk at the garage. The thickness of this clay layer ranged between 1.0 to 4.0 m. The water table within the site is located at about 3.6 m below ground level.

Fig. 3 illustrates the average subsurface condition at the site and the results of the Standard Penetration Tests.
The garage was constructed following the cut-and-cover technique, utilizing the laterally supported diaphragm wall to retain earth and water pressures. The excavation depth within the garage reaches a maximum of 13.2 m from the ground surface. The diaphragm walls designed for the outer perimeter of the garage are 27.0 m in depth and 0.8 m in thickness and are fully reinforced from top to bottom.

The hydro-phrase machines used for drilling panels of the diaphragm walls utilized bentonite slurry. Excavation of panels using these machines induces fewer ground movements compared with the grab bucket technique of excavation.

After installing the diaphragm walls, the sequence of excavation and construction within the garage structure was carried out in a top-down sequence of work, i.e., casting slabs and then excavating underneath them to reach the level of the following slabs. With this technique, the slabs act as internal lateral supports to the main diaphragm walls during excavation to the finish level (Elevation +8.0 m). Fig. 4 shows a schematic presentation of the construction stages that was followed at the El- Tahreer garage (2).

The construction of the garage passed through 3 main stages, namely:

1. Diaphragm wall installation. The vertical sides of the 13.2 m deep excavations were supported using fully reinforced cast-in-place concrete diaphragm walls (27.0 m deep and 0.8 m thick).
2. A grouted plug was injected between the diaphragm walls from the elevation (-5.80 m) to (-1.80 m) to control the groundwater level outside the deep excavation.
3. Excavation and strutting sequences to reach the finish level of the garage with simultaneous lowering of the groundwater levels within the deep excavation.

Fig. 5 shows a schematic cross section of the constructed garage (2).
4. FIELD MONITORING

In order to monitor the performance of the supported deep excavation of the El-Tahreer (2) multistory underground garage project, an in-situ instrumentation program was implemented as shown in Fig. 6. It consisted of the following:

1. Six inclinometers (21.0 to 24.0 m long) for measuring lateral displacement. Five of them were installed adjacent to the diaphragm wall near the El-Sadat Metro station (21.0 to 23.0 m long). The sixth was installed inside the diaphragm wall (24.0 m long).
2. Six deep settlement points were also installed surrounding the garage to measure the settlement at different levels. Also, surface settlement measurements were conducted using precise level surveying equipment.
3. Ten piezometers, five of them located within the excavation and five installed outside the diaphragm wall near the El Sadat Metro Station. These instruments were used to monitor the groundwater levels inside and outside the excavation. There was no change in the elevations of the groundwater table around the station itself.

5. NUMERICAL MODELING

Finite element analysis was utilized to model the excavation sequences and calculate the lateral movement of the diaphragm walls as well as the vertical displacement of the soil surface. PLAXIS®™, a commercially available finite element program, was used to conduct this C-type prediction. The two-dimensional mesh of the El-Tahreer garage (2) and the adjacent Metro station is shown in Fig. 7.

Six-node triangular isoparametric elements were used with a total of 2331 nodes and 1019 elements to model the soil strata and the grouting. Three-node beam elements were used to model the diaphragm wall, basement floors, and Metro station. Six node interface elements were utilized to model the interface between the soil and the diaphragm walls.

5.1 Soil Modeling

The hardening soil model developed by Schanz & Vermeer (1997) was implemented in the analysis to model the soil strata. This model combines the merits of plasticity theory with the logic of the Duncan-Chang model (El-Nahhas & Morsy, 2002). Some basic characteristics of the model are:
Stress dependent stiffness according to the power law for the primary loading:

\[ E_i = E_i^{\text{ref}} \left(\frac{C \cot \phi - \sigma_3'}{P_\text{ref}}\right)^m \]  

where
- \( C \) = cohesion;
- \( E_i \) = initial stress dependent on Young’s modulus;
- \( E_i^{\text{ref}} \) = reference to Young’s modulus corresponding to reference pressure \( P_\text{ref} = 100 \text{kPa}; \)
- \( m \) = power presents the stress depending (= 0.50 for sand);
- \( \sigma_3' \) = effective preliminary stresses;
- \( \phi \) = angle of internal friction.

The hyperbolic relationship between the strain and deviator stress:

\[ \varepsilon_3 = \frac{1}{E_i} \left( \frac{q}{q_s} - \frac{q}{q_f} \right) \]  

where
- \( q \) = deviatoric stress in primary triaxial loading;
- \( q_s \) = asymptote deviatoric stress (= \( q_f/R_f \));
- \( q_f \) = failure deviatoric stress;
- \( R_f \) = failure ratio;
- \( \varepsilon_3 \) = vertical strain.

The distinction between the primary deviatoric loading and unloading/reloading stress path:

\[ E_{\text{un}} = E_{\text{un}}^{\text{ref}} \left(\frac{C \cot \phi - \sigma_3'}{P_\text{ref}}\right)^m \]  

where
- \( C \) = cohesion;
- \( E_{\text{un}} \) = unloading and reloading of Young’s modulus;
- \( E_{\text{un}}^{\text{ref}} = E_i^{\text{ref}} \).

Failure behavior according to the Mohr-Coloumb failure criteria:

The yield function of the hard soil model,

\[ F = \frac{2}{E_i^{\text{ref}}} \left( \frac{q}{q_s} - \frac{2q}{q_f} \right) - \gamma' \]  

where
- \( \gamma' \) = strain hardening parameter;
- \( \varepsilon_1' \) = plastic axial strain;
- \( \varepsilon_1'' \) = plastic volume strain.

In the case of sand, the plastic volumetric strain tends to be relatively small, and this leads to the approximation \( \varepsilon_1'' = 0.0 \). The total strain variant,

\[ \varepsilon_i = \varepsilon_1' + \varepsilon_1'' \]  

where

\[ \varepsilon_1' = \frac{q}{E_i^{\text{ref}}} - \frac{q}{q_s} \]  

\[ \varepsilon_1'' = \frac{1}{E_i^{\text{ref}}} \left( \frac{q}{q_s} - \frac{q}{q_f} \right) \]  

\[ \varepsilon_i = \frac{1}{E_i} \left( \frac{q}{q_s} - \frac{q}{q_f} \right) \]  

Fig. 7. Finite element idealization of the diaphragm wall and the El-Sadat Metro station.
The stress dilatancy theory:
The essential feature of the stress dilatancy theory is that the material compacts for small stress ratios ($\phi_m < \phi_{cv}$), while the dilatancy occurs for high stress ratios ($\phi_m > \phi_{cv}$), where $\phi_m$ is the mobilized angle of the internal friction and $\phi_{cv}$ is the critical state of the internal friction angle. The corresponding volumetric strain ratio to the mobilized dilatancy ($\psi_m$) is described by:

$$\psi_m = \sin \phi_m - \sin \phi_{cv}$$

$$\sin \phi_m = \frac{c'_1 - c'_2}{c'_1 + c'_2 - 2C \cos \phi}$$

where

- $\gamma^P$ = rate of plastic shear strain;
- $\psi_m$ = mobilized dilatancy angle;
- $\psi$ = failure of the dilatancy angle.

Table (2) summarizes the parameters for the fill and the two main soil strata used for the finite element simulation.

### 5.2 Modeling the Supporting Elements

The components of the laterally supported diaphragm walls were simulated using the capabilities of the Plaxis program. The properties of the reinforced diaphragm walls, horizontal reinforced slabs, raft foundation, and Metro station were simulated using beam elements.

### 6. RESULTS AND DISCUSSION

The compiled field measurements of the three inclinometers at the locations I(3), I(4), and I(5) were selected to represent the measured lateral displacement of the soil. Inclinometer I(3) is about 2.0 m from the El-Sadat Metro station. Readings were recorded throughout the excavation and construction of the diaphragm walls and throughout the different construction stages that are shown in Fig. (4). Figs. 8 to 10 show a comparison between the measured and calculated lateral deflections of the diaphragm walls. The negative sign in the lateral deformation indicates displacement towards the excavation.

The maximum lateral displacement during the construction of the diaphragm wall was predicted to be 8.2 mm, while the maximum measured lateral displacement did not exceed 1.85 mm. This can be attributed to the limited depth of the inclinometers used (21.0 to 23.0 m from the ground surface). It should be noted that the data reduction of the inclinometer readings is based on assuming the fixed lower end of the casing. The limit depth of the inclinometers used allowed the toe of the pipe to move laterally. Thus, the measured displacement by the inclinometers is considered to be a relative displacement between the upper point and the lower point of the inclinometer.

A detailed comparison between the predicted and observed lateral displacement during the actual construction sequences at the inclinometers I(3), I(4), and I(5) are shown in Figs 8a, b, c, and d. Figure 8 reveals that:

- During the first and second stages of construction, the field and the computed lateral movement had similar trends and close values, especially those of inclinometer I(3).
- The measured values of the lateral movement of the inclinometers I(4) and I(5) were less than the computed ones by about 20% to 50% from 5.0 m up to 5.0 m

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Identification Type</th>
<th>$\gamma_{dry}$ (kN/m$^3$)</th>
<th>$\gamma_{wet}$ (kN/m$^3$)</th>
<th>$k_x$ (m/day)</th>
<th>$K_y$ (m/day)</th>
<th>$C_{ref}$ (kN/m$^3$)</th>
<th>$\phi$ (°)</th>
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<tr>
<td>1</td>
<td>Fill</td>
<td>Drained</td>
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<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>Medium dense Sand</td>
<td>Drained</td>
<td>17.0</td>
<td>18.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>Very dense sand</td>
<td>Drained</td>
<td>17.5</td>
<td>19.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
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<table>
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<tr>
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<th>Identification Type</th>
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<th>$\psi_{ref}$ (°)</th>
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<th>$E_{ref}^{sat}$ (kN/m$^2$)</th>
<th>$E_{ref}^{sed}$ (kN/m$^2$)</th>
<th>$E_{ref}^{cr}$ (kN/m$^2$)</th>
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<tbody>
<tr>
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<td>Drained</td>
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<td>0.2</td>
<td>100</td>
<td>45000.0</td>
<td>45000.0</td>
</tr>
<tr>
<td>2</td>
<td>Medium dense Sand</td>
<td>Drained</td>
<td>3</td>
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<td>Drained</td>
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<td>0.3</td>
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</table>
below the surface. The deeper measured values approximately coincide with the predicted values. During the third and fourth stages of construction, the field and the computed lateral movement were completely different. This is due to the limited depth of the inclinometers used. The limited depth of the inclinometers used allowed the toe of the pipe to move laterally. Thus, the measured displacements by the inclinometers are considered to be the relative displacement between the different points and the lower point of the inclinometer. The modified predicted deformation was obtained by subtracting the calculated movement at the end of the inclinometer casing from all the upper calculated values. The measured lateral displacements subsequently better matched the modified calculated deformation of these two stages.

Fig. 9 shows a comparison between the predicted and observed lateral displacements during the first and fourth construction stages as an example of the obtained values. Also an evaluation was carried out for the calculated and field-measured lateral deformation for inclinometer I(1), which represents the lateral deformation of the soil near the garage diaphragm walls, but is relatively far from the Metro tunnel. This evaluation is shown in Fig. 10, which represents the first and fourth stages of construction as an example of the results obtained.
The predicted settlement at point DS(2) and the measured surface and subsurface settlements are shown in Fig. 11. The measured and predicted surface and subsurface settlement profiles do not match, but the field-measured settlements or vertical movements were generally less than the predicted values. This result agrees with Abdel-Rahman, A. & El-Said, S. M. (2002). The predicted settlement was 100%, at some points, more than the field-measured values. The measured surface settlement was also less than that proposed by Clough and O’Rourke (1990), but matched Thompson (1991).

7. CONCLUSIONS

This paper discusses the performance of the diaphragm wall of an underground multi-story garage constructed in downtown Cairo based on the geotechnical instrumentation. The field measurements are compared with the predicted soil movements calculated using a two-dimensional finite element analysis. Although the measured soil movements were less than the predicted values, a reasonable concordance is achieved. This can be attributed to the difference between the physical and analytical modeling.

Inclinometers are good tools to measure and investigate the lateral deformation of soil due to earthwork. However the casing utilized must be deep enough to obtain reliable results. For this reason inclinometers inside the diaphragm walls have to be installed at least to the end of the walls or even deeper.

It is recommended to install the inclinometers in a similarly important project to develop feedback analysis for performing more accurate analytical models of soil parameters for the lateral deformation of diaphragm walls or for better design criteria.

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