Foundation subsidence due to trenching of diaphragm walls and deep braced excavations in alluvium soils

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ABSTRACT
This paper presents a case history in Giza, Egypt in which the settlements of existing structures founded on shallow and deep foundations surrounding a deep excavation retained by strutted diaphragm walls were monitored. The presented case history comprised inclusive settlement monitoring during the different stages of construction including diaphragm wall trenching and the stages of excavation to the foundation level. The measured settlements were back-analyzed using two and three dimensional numerical models to evaluate the different deformation patterns associated with wall and pit excavation stages. The analyses showed that foundation settlements, during the different stages of deep excavations including the diaphragm wall trenching, are substantially influenced by the foundation type and depth and its relation to the depth of excavation.

1 INTRODUCTION

Many authors studied the deformation troughs associated with deep braced excavation. Peck (1969a) provided the first comprehensive review of the factors that control the deformations induced by deep excavations in alluvial soils including local subsurface conditions, depth of excavation, and workmanship quality. oldberg et al. (1976) reviewed 63 monitored case histories of deep excavations to correlate the maximum settlement to the soil type and the depth of excavation. Clough and O'Rourke (1990) and Bentler (1998) suggested updated settlement profiles in alluvial soils. A comparison between the recommended values of the maximum settlement with respect to the maximum depth of excavation is given in Table (1). Settlement troughs associated with braced excavations is shown in figure (1).

Table (1): The maximum ground settlement / the depth of excavation as applicable to deep braced excavations.

Researcher(s)	sands, gravels and very stiff to hard clays	soft to stiff clays
Goldberg et al. (1976)	0.171%	1.22%
Clough & O'Rourke (1990)	0.30%	
Bentler (1998)	0.22%	0.545%

2 SUBSURFACE CONDITIONS

The case study was located in Dokki District at 1 km away from the river Nile. The project location is geologically characterized by typical young alluvial plain for the lowland portion of the Nile Valley. A geotechnical subsurface investigation program was performed comprising 8 boreholes having a depth of 25m. The subsurface soil profile consists generally from a top fill layer appeared from ground surface to a depth of 2.0 m, followed by a silty sand layer up to a depth of 5.0 m. A layer of medium dense fine to medium sand with some silt followed the silty sand layer to a depth of 11.0 m. A dense to very dense graded sand layer followed the previous layer and extended to the end of the boreholes at 25.0 m. The bottom sand layer occasionally contained a percentage of fine gravel in the range of 5.0% to 15%. The results of the SPT tests with depth are presented in figure (2). The groundwater is located at an average depth of 2.00 m.

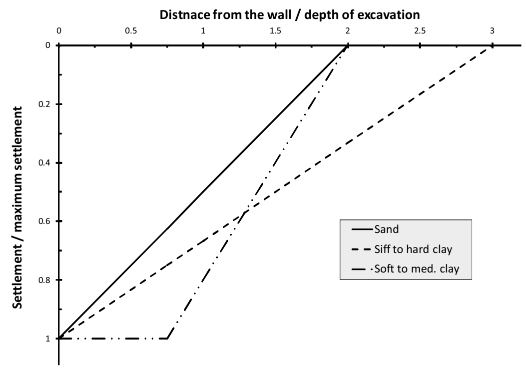

Figure (1). Settlement profiles adjacent to braced excavation (after Clough and O'Rourke, 1990).

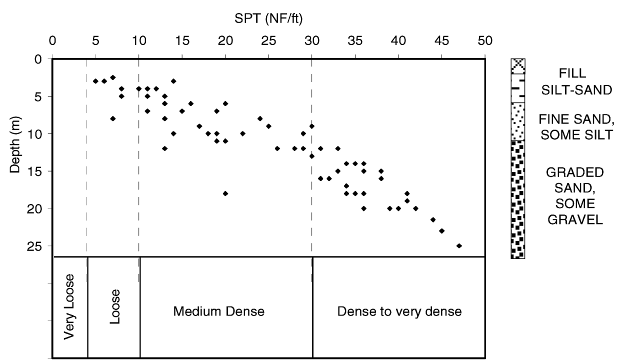

Figure (2). Stratification and SPT data

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3 MONITORING PROGRAM

Five buildings (A), (B), (C), (D) and (E), are located near to the excavation pit as shown in figure (3). Buildings (A), (B), and (C) are 12 to 14 stories, founded on piles of lengths ranging between 14.00 m and 16.00 m. Building (D) is a five story building and building (E) is a two-story building, founded on shallow foundations at a depth of about 2.0 m to 3.0 m, respectively.

An optical surveying monitoring program for settlement of 31 columns in the adjacent buildings was developed from July 5, 2001 to March 24, 2002 to assess buildings settlement and intervene in case any damage is anticipated. The locations of the monitored points, shown in figure (3), were determined based on predicted behavior of the site to elude any geotechnical and structural concerns. The layout of the instrumentation plan tend to provide more monitoring data about building "A" as the wall is just 1.80 m from this building which means that its piles are closer to the diaphragm wall boundary.

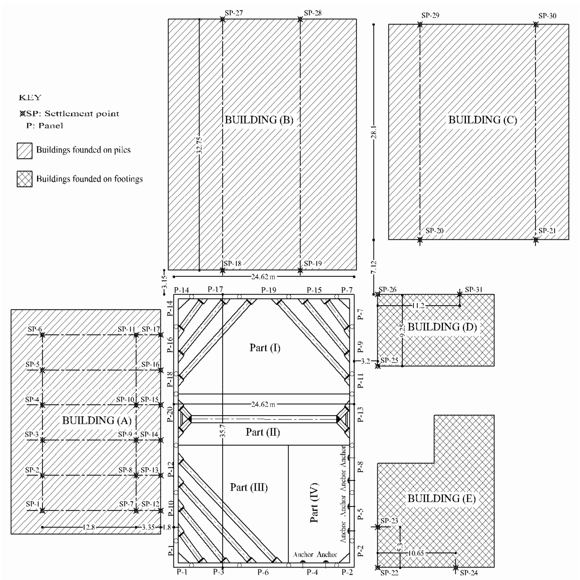

Figure (3). Monitoring system for surrounding buildings

4 CONSTRUCTION SEQUENCE AND SETTLEMENTS

A bottom-up construction method with two levels of temporary bracing was adopted during excavation to construct the foundation and basement floors of the multistory building. A cast-in-situ diaphragm wall, comprising 20-panel having thickness of 0.6m and a depth of 21m, was utilized for shoring the site boundaries. The lengths of the panels are ranged between 2.7 and 6.72m. The paneling scheme is shown in figure (3). The subsequent phases of execution included the excavation of 9100 cubic meters of soil and the installation of lateral support for each panel to exclude the use of walings.

Most of the panels were supported using structural steel pipe struts with outside diameters of 900 mm and a thickness of 9.0 mm. Some panels were supported using tie-back anchors to provide space to move equipment and soil from/to the pit. Figure (4) demonstrates the different stages of construction. Upon completion of the excavation, at a depth of 10.8m below ground surface, the concrete base mat was constructed. Results of the monitored settlement observed after trenching and pit excavation are tabulated in Table (2).

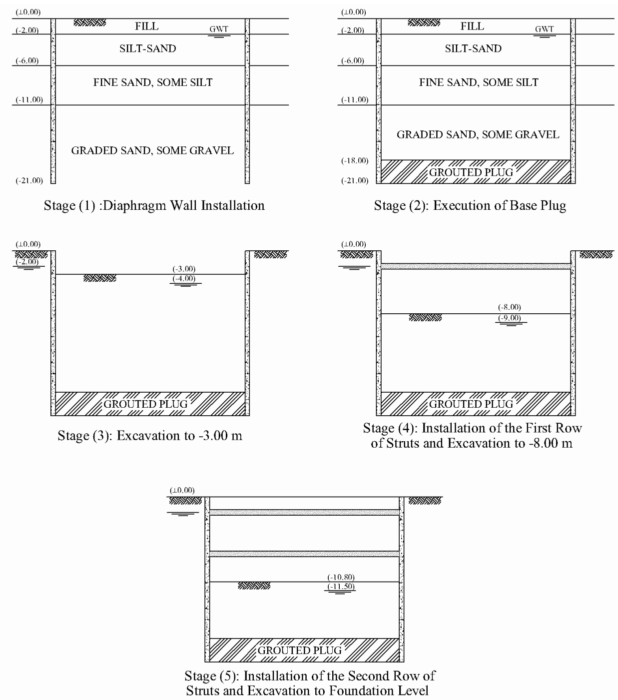

Figure (4). Stages of construction

Table (2): Observed settlement through the different construction stages

Point	Sett. due to trenching (mm)	Sett. due to pit excavation (mm)	Total sett. (mm)
1	1.1	-0.4	0.7
2	0.7	-0.5	0.2
3	0.9	0.3	1.2
4	1.3	1.2	2.5
5	1.1	1.2	2.3
6	2	1.3	3.3
7	4	-0.2	3.8
8	5.3	0.5	5.8
9	5.3	0.7	6
10	6.5	2.2	8.7
11	3.4	1.1	4.5
12	7.5	0	7.5
13	8.5	0.7	9.2
14	8.6	1.4	10
15	8.6	1.4	10
16	8.4	1.9	10.3
17	8.6	2.4	11
18	8	4.3	12.3
19	7.8	4.7	12.5
20	2.5	1.3	3.8
21	2	2.5	4.5
22	6	9.1	15.1
23	6.8	11	17.8
24	0.5	0.7	1.2
25	6.5	7.4	13.9
26	5.8	6	11.8
27	1.1	0	1.1
28	0.3	0	0.3
29	0.4	0	0.4
30	0	0	0
31	0.4	2.4	2.8

5 ANALYSIS OF THE OBSERVED SETTLEMENTS

5.1 *Settlement due to Diaphragm Wall Installation*

As shown in Table (2) and figure (5), a maximum settlement of 8.6 mm was recorded during trenching at the location of point 15 (Building “A” on pile foundations), while a null settlement was recorded at point 30 (Building “C” on pile foundations also). That could be related to the fact that point 15 is located very close to the diaphragm wall, while point 30 is located more than 40 m from the corner of the construction site. 2D and 3D finite element back-analyses of building settlement during the installation of the diaphragm wall of this project were carried out and discussed in details by Abdel-Rahman and El-Sayed in two separate articles (Abdel-Rahman & El-Sayed 2002a; El-Sayed & Abdel-Rahman, 2002); the maximum lateral displacement was estimated as almost two-thirds of the maximum settlement; moreover, trenching settlement for secondary panels is generally less than primary panels in case of having equal panel dimensions. It was also concluded that the settlements of all buildings can be expressed with one envelope with a maximum settlement equivalent to 0.045% of the diaphragm wall trench depth, while its extent away from the wall reaches to twice the trench depth, regardless of the foundation type for the case study under consideration.

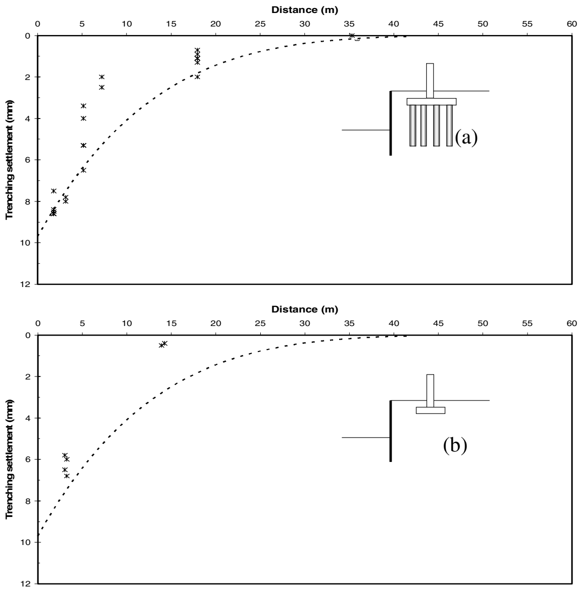

Figure (5). Settlement due to trenching (a) deep foundations; (b) shallow foundations

5.2 *Settlement due to Pit Excavation:*

As indicated in Table (2) and figure (6), settlement of points on deep foundations showed much less settlement increments (max. 4.7 mm) than those installed on shallow foundation (max. 11mm). It is also worth noting that only deep-foundation points (18) and (19) of building "B" experienced a typical settlement values (4.3 & 4.7 mm) that generally did not follow the pattern of behavior of the rest of the other deep-foundation settlement points, as shown in figure (6-b), although building "B" is located a bit away from the excavation boundary than building "A". This could be related to the relatively short distance between the depth of its pile tip (14.0 m from ground surface, as reported or could be less when executed) and the maximum depth of excavation inside the pit (10.80 m).

The maximum settlement due to pit excavation can be expressed as 0.03% H for deep foundations (Excluding points 18 & 19). For shallow foundations, maximum settlement is 0.11% H. The widths of the settlement trough, in case of pit excavation, are about 41 m (3.8 H), and 24 m (2.2 H) for the cases of pile foundations and shallow foundations, respectively.

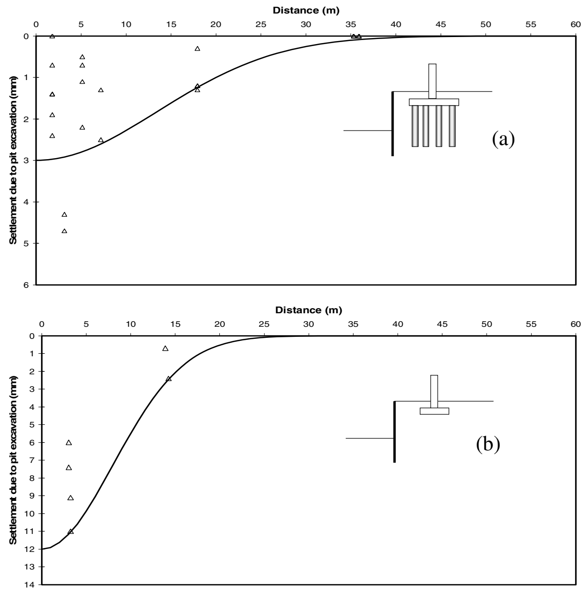

Figure (6). Settlement due to pit excavation (a) deep foundations; (b) shallow foundations

5.3 *Settlements Envelope for the Different Components*

Settlements can be divided into two components: trenching settlement ($S^{trenching}$) and pit excavation ($S^{pit\ excav}$). Settlement envelope due to trenching can be expressed by the following equation (Abdel-Rahman & El-Sayed, 2002a & 2002b):

$$S^{trenching} = S_{max}^{trenching}\left(\frac{2d-x}{2d}\right)^6 \qquad \ldots\ldots(1)$$

where, "$S^{trenching}$" is the settlement at a distance "x" from the trench boundary, "$S_{max}^{trenching}$" is the maximum settlement at the trench location (0.045% of the trench depth), and "d" is the trench depth.

Settlement envelope due to pit excavation can be described by the following equation (Abdel-Rahman & El-Sayed, 2002c):

$$S^{pit\ excav} = S_{max}^{pit\ excav}\exp\left(\frac{-x^2}{2(K\cdot H)^2}\right) \qquad \ldots\ldots(2)$$

where, "$S^{pit\ excav}$" is the settlement at a distance "x" from the trench, "$S_{max}^{pit\ excav}$" is the maximum settlement at the wall location, "K" is a dimensionless factor and "H" is the final depth of pit excavation. The proposed distribution is similar to that proposed by Peck (1969b) for tunneling; theses parameters were determined using best fitting analysis and the results are summarized in Table (3).

Table (3) Parameters of the Pit excavation component

Buildings	$S_{max}^{pit\ exc.}$	K	Trough width
"A", "B" & "C" (on pile foundations)	0.03% H	1.25	3.8 H
"D" & "E" (on shallow foundations)	0.11% H	0.75	2.2 H

It should be noted that the above settlement envelope for buildings on pile foundation represents the common case of having an excavation of a depth less than the pile tip.

5.4 *Total/combined settlement trough*

Figure (7) shows the distribution of the total settlement field for building on deep foundations and on shallow foundations during the different stages of construction. The maximum total settlement is about 0.20% and 0.12% of the excavation depth for the cases of shallow, and pile foundations, respectively.

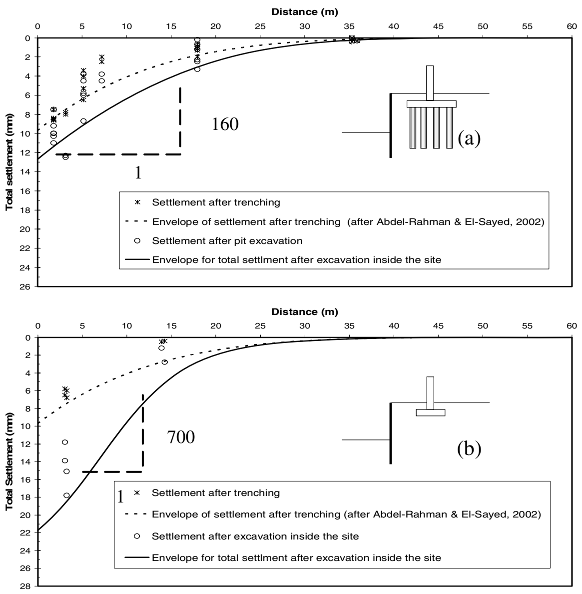

Figure (7). Total settlement (a) deep foundations; (b) shallow foundations

Most of the settlements in buildings on pile foundations occurred with during the trenching stage (about 76% of the total settlement). On the contrary, most of the settlement pile-founded buildings (about 56% of the total settlement) is attributed to pit excavation. The maximum angular inclinations of settlements are 1/1600 and 1/700 for deep foundations and shallow foundations, respectively. A comparison between the maximum settlements and the recommendations of some of the researchers is shown in Table (4). The final trough width, as shown in figure (7) and Table (4), can be practically set to be 3.5 of the pit excavation depth (~38.0 m), despite the different behaviors of the pile and deep foundations under the different stages of construction.

Table (4) Comparison between measurements and different settlement criteria

Buildings	% Max. settlement/ depth of excavation	Trough width/Exec. depth
“A”, “B” & “C” (pile foundations)	0.12	3.5
“D” & “E” (shallow found.)	0.20	3.5
Goldberg et al. (1976)	0.171	-
Clough and O'Rourke (1990)	0.30	2
Bentler (1998)	0.22	-

6 SUMMARY AND CONCLUSION

The data compiled from a case history carried out in Nile alluvial soils was used to proposed envelopes of the different settlement components demonstrating the effect of the foundation types/depths as an additional criteria that should be considered in analysis of building settlements associated with deep braced excavations.

REFERENCES

Abdel-Rahman, A.H., “Construction Risk Management of Deep Braced Excavations in Cairo”, Australian Journal of Basic and Applied Sciences, 1(4): pp. 506-518, 2007.

Abdel-Rahman, A. H. and El-Sayed, S. M., 2002a, "Settlement Trough Associated with Diaphragm Wall Construction in Greater Cairo", the Journal of the Egyptian Geotechnical Society, Cairo, Egypt.

Abdel-Rahman, A. H. and El-Sayed, S. M., 2002b, “Building Subsidence Associated with Cut-and-Cover Excavations in Alluvial Soils”, Faculty of Engineering Scientific Bulletin, Ain Shams University, Vol. 37, No. 4, 2002, Cairo, Egypt.

Bentler, D. J., 1998, “Finite Element Analysis of Deep Excavations”, Ph.D. Thesis, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

Clough, G. and O'Rourke, T., 1990, "Construction Induced Movements of Insitu Walls", Design and Performance of Earth Retaining Structures, ASCE Geotechnical Special Publications 25, pp. 439-470.

El-Sayed, S. M. and Abdel-Rahman, A. H., 2002, “Spatial Stress-Deformation Analysis for Installation of a Diaphragm Wall”, Faculty of Engineering Scientific Bulletin, Ain Shams University, Vol. 37, No. 3, 2002, Cairo, Egypt.

Goldberg, D.T., Jaworski, W.E. and Gordon, M.D., 1976, "Lateral Support Systems and Underpinning", Report FHWA-RD-75-128, Vol. 1, Federal Highway Administration, Washington D.C., p. 312.

Peck, R. B., 1969a, State-of-the-art, "Deep Excavation and Tunneling in Soft Ground", Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, Universidad Nacional Autonoma de Mexico Instituto de Ingenira, Mexico City, Mexico, Vol. 3, pp. 225-290

Peck, R.B, 1969b, "Advantages and Limitations of the Observational Method in Applied Soil Mechanics. Geotechnique, Vol. 19, No. 2, pp. 171-187.