A COMPARATIVE STUDY OF PERFORMANCE DURING EXCAVATION USING VARIOUS GROUND IMPROVEMENT METHODS AT LOT 259C

Yung-Kang Yang and Ming-Feng Song

Moh and Associates, Inc.

Abstract: A comparative study of performance using various ground improvement methods was carried out at a number of typical sections during underground structure excavation at lot 259C of the Taipei Mass Rapid Transient System. Back analyses of the diaphragm wall deformations during excavation were performed for assessing the sensitivity of subsoil parameters and effects of ground improvement, as well as obtaining reliable subsoil parameters. With the soil parameters obtained through back analyses, a reasonable agreement exists between back-analyzed and monitored diaphragm wall deformations. Relationship of monitored diaphragm wall deformations, and adjacent ground surface settlement was then studied and compared using a number of predictive models. The adopted ground improvement measures for lot 259C was concluded to be effective in protecting adjacent buildings based on monitoring data.

Keywords: ground improvement, building protection, diaphragm wall, settlement.

1. INTRODUCTION

In a highly populated metropolitan area like Taipei city, engineering challenges such as deep excavation adjacent to existing buildings are frequently faced. In addition to ensuring construction safety, it is also of significant importance to protect adjacent buildings during the excavation to reduce potential risk.

Ground improvement within the excavation area to reduce diaphragm wall deformations, and consequently to reduce adjacent ground surface settlements as well as building settlements, has been a favorable approach for local engineering practice. This paper presents a comparative study of ground improvement methods, diaphragm wall deformations, and adjacent ground surface settlements at a number of representative sections during underground excavation for lot 259C of Taipei MRT (Mass Rapid Transit System), which lasted from November 1997 through September 1999.

The experience of ground improvement within the excavation site for building protection in 259C is a valuable reference for construction work near existing buildings with similar ground condition.

2. SITE LOCATION

The 259C site is located in the eastern part of the Taipei Basin. The south bound of the site is adjacent to Chung-Hsiao East Road; the east bound of the site is adjacent to Hsiang-Yang Road; the Spring Mall is near the west bound; to the north of the site are a number of adjacent buildings. Figure 1 presents the plan and location of 259C site. The ground surface of 259C site is generally flat, with reference elevations range from EL.+107.3m to EL.+108.9m.

The 259C site included several construction sections. The geotechnical issues of the cut and cover sections noted as B, C, D, E in Figure 1, as well as the adjacent buildings to the north of these sections, are the objectives of this study.

3. SUBSOIL AND GROUNDWATER CONDITIONS

Based on the results of geotechnical investigations carried out at the 259C site prior to and during design stage, the subsoils within 60m deep can be divided into 5 major layers:

Layer I: Silty clay layer existing from ground surface to about 14m deep; the average total unit weight ranges from 1.75 t/m^3 ~ 1.85 t/m^3; the SPT-N values range from 2~4.

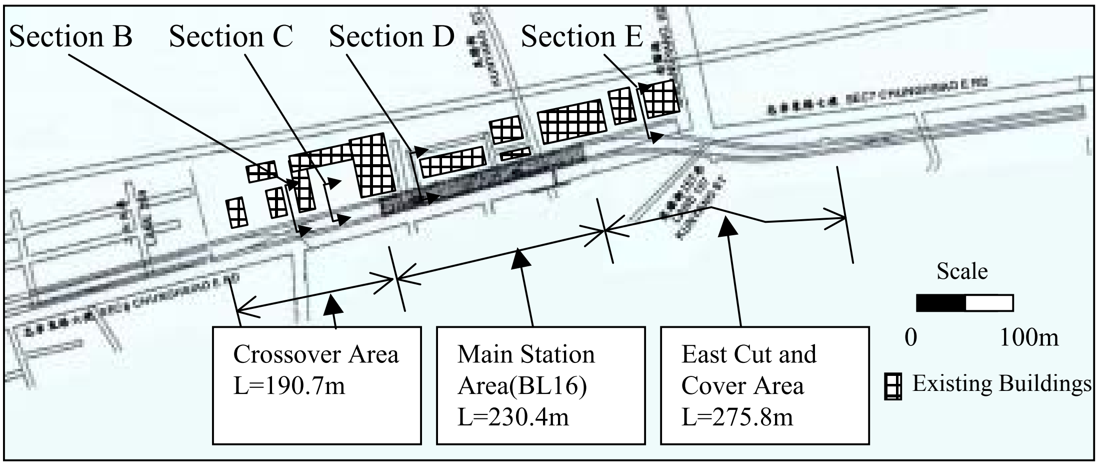


Figure 1 Site plan of 259C

Layer II: Silty clay layer existing from about 14m deep to 26m deep; the average total unit weight ranges from 1.75 t/m^3 ~ 1.85 t/m^3; the SPT-N values range from 5~10.
Layer III: Silty sand layer with occasional silty clay thin layers existing from about 26m deep to 36m deep; the average total unit weight is about 1.90 t/m^3; the SPT-N values range from 22~30.
Layer IV: Gravel layer existing from about 36m deep to 39m deep; the average total unit weight is about 2.00 t/m^3; the SPT-N values are greater than 100.
Layer V: Sandstone layer existing below about 39m deep; the average total unit weight is about 2.20 t/m^3; the SPT-N values are greater than 100.

Table 1 summarizes the subsoil conditions at the project site.

Table 1 Subsoil Conditions at 259C

Soil Type	Depth, m	Avg. SPT-N	Wn, %	γ_t, t/m^3	c', t/m^2	ϕ, deg
CL	14	2~4	30~43	1.75~1.85	-	-
CL	26	5~10	20~33	1.75~1.85	-	-
SM/CL	36	22~30	17~28	1.9	0	30
GM	39	>100	10~20	2.0	0	38
SS	-	>100	-	2.2	0	40

The groundwater distribution at 259C is presented in Figure 2, which indicates a groundwater pressure lower than the hydrostatic level below about 10m ~ 15m deep. The under hydrostatic condition was due to excessive pumping of groundwater before the 1970's, and has been recovering since the Taipei city government banned further pumping actions in mid 1970's.

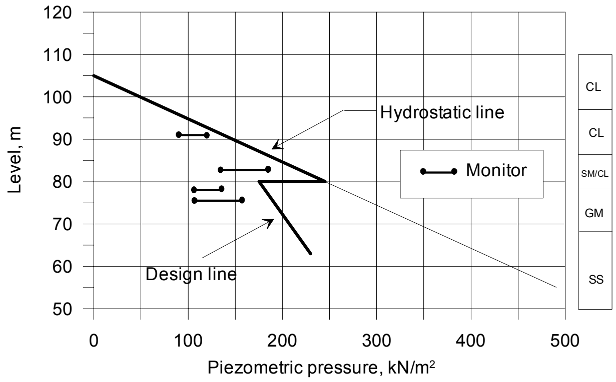


Figure 2 Variation of Groundwater Pressure with Level

4. SUMMARY OF THE EXCAVATION SECTIONS

As mentioned in Section 2, the excavation sections B, C, D, and E are the objectives of this study. The general descriptions, including chainage, excavation depths, areas, diaphragm wall depths and thicknesses, as well as ground improvement measures, of the studied sections are described as follows:

Area I: Crossover Area

Sections B and C are both located in the crossover area. Section B, which had an excavation depth of 15.0m, was internally braced with 4 levels of struts, and the subsoils were improved with ground beams within 3m beneath the final excavation depth to reduce diaphragm wall deformation. The ground beams in section B were 1m in width with 5m intervals. Section C, which had an excavation depth of 13.5m, was internally braced with 4 levels of struts, and no ground improvement was performed at this section since there was no immediately adjacent building.

Area II: Main Station Area

Sections D, which is located in the main station area and had an excavation depth of 15.0m, was internally braced with 4 levels of struts, and the subsoils were improved with ground beams within 3m beneath the excavation depth to reduce diaphragm deformation. The ground beams were 1m in width with 5m intervals.

Area III: East Cut and Cover Tunnel Area

Sections E is located in the east cut and cover tunnel areas. Section E, which had an excavation depth of 13.1m, was internally braced with 4 levels of struts, and the subsoils were improved with a ground slab within 3m beneath the final excavation depth to reduce diaphragm deformation.

The reduction of diaphragm wall deformations in sections B, D, and E due to ground improvement would lead to a reduction of adjacent ground surface settlements, and consequently a reduction of building settlements and inclinations immediately to the north of these sections.

Table 2 presents a summary of ground improvement works in these excavation sections.

5. BACK ANALYSIS OF DIAPHRAGM WALL DEFORMATION DURING EXCAVATION

To assess the sensitivity of subsoil parameters and effectiveness of ground improvement, as well as to obtain reliable subsoil parameters, a representative cross-section in each of sections B~E was selected for back analyses. Figure 1 presents the approximate locations of these representative cross-sections.

For performing back analyses, the computer software FREW was adopted as the analysis tool.

The soil parameters as listed in Table 1 and groundwater conditions presented in Figure 2 were used as geotechnical input data. Other input data adopted for analysis included diaphragm wall information, excavation procedures, strut system, and properties of improved ground.

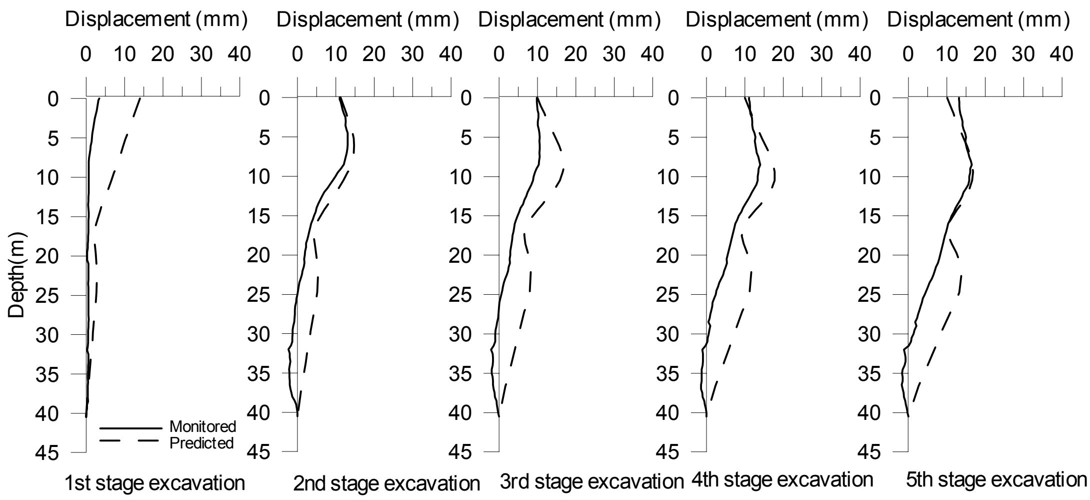
The results of back analysis showed that by assuming $E=1000*s_u$ for clayey soils and $E=300*N$ for sandy soils, the predicted diaphragm wall deformations best fit the observations. Figure 3 presents the results of back analysis against monitoring data of diaphragm wall deformations at section B, where ground improvement with ground beams was adopted, and the result of back analysis against monitoring data of diaphragm wall deformations at section C, where no ground improvement was performed. Figure 3, which demonstrates typical results of back analyses, indicates that the diaphragm wall deformations obtained through back analyses had a good agreement with monitoring data. Similar results were obtained at other sections.

The strut forces obtained through back analyses were also investigated and compared with the monitoring data. Table 3 lists the results of comparisons for sections B, C, D, and E. Generally, the monitoring data have a reasonable agreement with the back-analyzed data, except for the bottom strut level of sections B and D, where the differences were 80% and 160% respectively. The significantly greater monitored strut force at the bottom level of sections B and D, where ground improvement were performed, might be attributed to the following two reasons which were difficult to be considered in computer modeling:

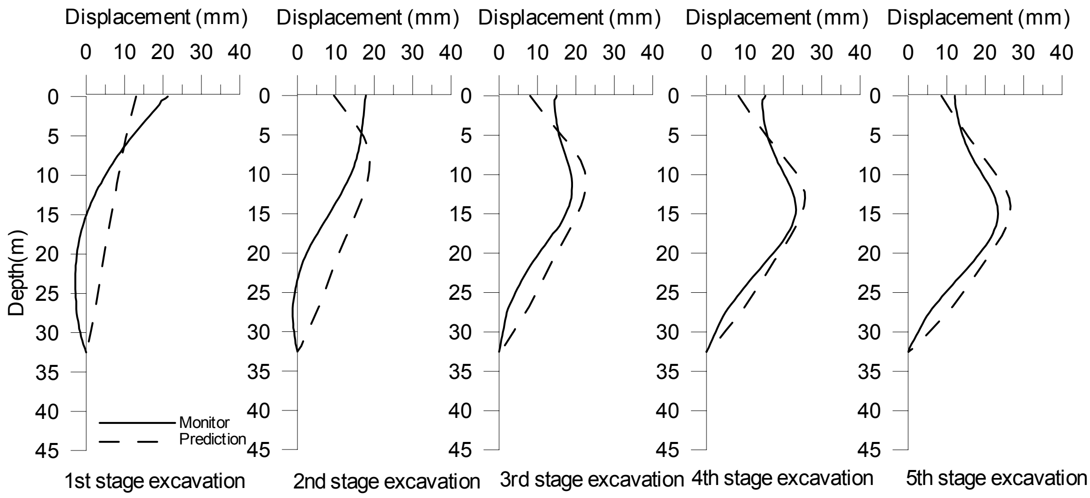
Table 2 Descriptions of Each Excavation Section

Area	Section	Plan Area, m×m	Ground Improvement Method	Diaphragm Wall	
				Depth,m	Thickness,m
Crossover Area	B	14×21	Ground beam improved within 3m below final excavation depth	(N)32 (S)31	1.0
	C	70×21	No ground impreovement	(N)33 (S)32	1.0
Main Station Area	D	230×21	Ground beam improved within 3m below final excavation depth	(N)32 (S)30	1.0
East Cut and Cover Area	E	113×21~40	Ground slab improvement within 3m below final excavation depth	(N)30.5 (S)29.5	1.0

Note: (N) represents north side; (S) represents south side



Section B (with Ground Improvement)



Section C (without Ground Improvement)

Figure 3 Back-Analyzed and Monitored Diaphragm Wall Deformations at Sections B and C

Table 3 Comparisons of Maximum Strut Forces

Section	Strut Level	Depth, m	Back Analyzed Maximum Strut Force, ton/m	Measured Maximum Strut Force, ton/m
B	1	2.0	21.4	26.4
	2	5.2	54.3	54.5
	3	8.2	36.7	46.1
	4	11.7	30.8	56.0
C	1	2.0	38.6	35.2
	2	5.2	52.9	72.3
	3	8.2	49.5	63.9
	4	11.0	44.0	61.2
D	1	2.0	22.7	28.8
	2	4.8	40.0	50.1
	3	8.0	38.2	80.3
	4	11.6	27.7	71.8
E	1	2.0	19.9	26.6
	2	4.7	48.7	41.0
	3	7.2	36.9	42.1
	4	9.7	43.6	47.2

(i) Certain gaps existed between the improved soils and the diaphragm wall, allowing certain lateral movement of diaphragm wall before the passive resistance of the improved ground became effective.
(ii) Some local defects occurred within the area of improved soils facing the diaphragm wall.

Both reasons could also explain the larger back-analyzed diaphragm wall deformation near the final excavation depth. Nonetheless, the real cause of the greater monitored strut force at the bottom level of strut should be further studied.

6. RELATIONSHIP BETWEEN DIAPHRAGM WALL DEFORMATION AND GROUND SETTLEMENT

For the geological conditions in the Taipei Basin, a number of relationships have been built to correlate diaphragm wall deformations with ground surface settlement. For this study, the settlement troughs obtained by using the predictive models proposed by Ou [1] and by Moh and associates, Inc. [2] [3], respectively, were adopted to compare the results with the monitoring data. The model proposed by Ou is based on result of finite element modeling; the model proposed by Moh and Associates, Inc. is an empirical approach considering factors such as diaphragm wall construction, diaphragm wall deformation during excavation, and dewatering, etc. Both models were developed based on project experiences in the Taipei Basin.

Sections B, D, and E were treated with ground improvement and were well instrumented by using inclinometers in the diaphragm walls and ground surface settlement points. Theses sections were adopted for comparing the diaphragm wall deformations vs. ground surface settlement relationships, as presented in Figure 4. The results from the two predictive models are both in a reasonable agreement with the observations.

As can be noted, during the early stage of retaining system design, the allowed diaphragm wall deformation may be controlled, by means of suitable ground improvement methods in addition to strut system, to such a level that the impact upon adjacent buildings would fall within acceptable level. However, the possible upper bound and lower bound of the settlement trough must be taken into account to cover uncertain factors such as variations in subsoil conditions and the quality of construction and ground improvement.

For the 259C project, the ultimate objective of controlling diaphragm wall deformations is to protect adjacent buildings to the north of sections B, D, and E against potential structural damages. Until the completion of the excavation work, only four neighborhood buildings subjected to various levels of damages, all of them faced only non-structural minor problems. It is important to emphasize that the impact of excavation upon adjacent buildings lies on a wide range of factors, such as workmanship, quality of ground improvement, and age, foundation and structural type of the buildings. However, ground improvement does play a key role in further protecting the overall neighborhood buildings, of which the safety usually cannot be fulfilled with typical bracing system alone.

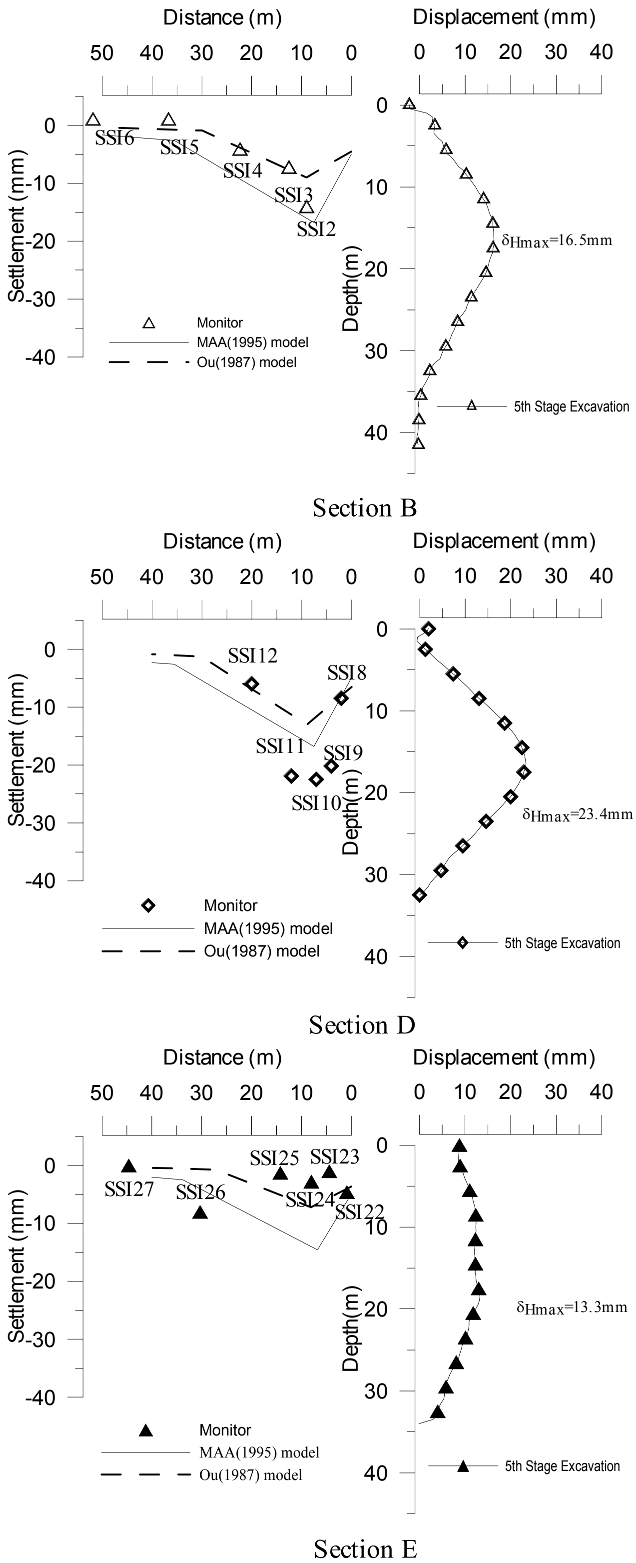


Figure 4 Diaphragm Wall Deformations vs. Ground Surface Settlement

7. SUMMARY AND CONCLUSIONS

1.Due to the existence of existing buildings adjacent to the north bound of the 259C site, ground improvement within the excavation area of section B, section D, and section E was performed as a protective measure. Section B and D were treated with ground beam method; section E was treated with ground slab method. Section C, without ground improvement, was also included in this paper for comparisons.

2.Back analyses of the diaphragm wall deformations during excavation were performed for assessing the sensitivity of subsoil parameters and the effectiveness of ground improvement, as well as obtaining reliable subsoil parameters. The diaphragm wall deformations obtained through back analyses had a good agreement with monitoring data in sections with and without ground improvement.

3.The monitored bottom strut forces in sections B and D were significantly larger than those obtained in back-analyses. Some possible reasons are discussed in this paper.

4.Relationship of monitored diaphragm wall deformation and ground surface settlement was studied and compared with predictive models proposed by Ou and Moh and Associates, Inc. based on experiences of excavation projects within Taipei Basin. The two predictive models are both in a reasonable agreement with the monitoring data.

5.In predicting ground surface settlement trough due to excavations, factors such as variations in subsoil conditions, construction and ground improvement qualities must be taken into account to cover uncertain factors.

6.Until the completion of the excavation work, only four neighborhood buildings subjected to various levels of damages, all of which faced only non-structural minor problems. Ground improvement played a key role in protecting the overall neighborhood building safety.

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REFERENCES

[1] Ou, C. Y and Shieh, P. G., "Predictions of Deep Excavation Induced Ground Surface Settlement"(in Chinese), *Design and Practice of Deep Excavation*, pp. 1-33, 1999.

[2] *Report on the Evaluation of Building Protection of Lot 303 of the Wan-Pan Project* (in Chinese), Moh and Associates, Inc., 1995.

[3] Chin, C. T., "Deep Excavation and Building Protection" (in Chinese), *Proceedings on Theory and Practice of Deep Excavation Design*, H.1~H.23, 1991.

[4] *Geotechnical Consultancy Report of Lots CN255, CN255B/C, CN256, CN256A/B, CN257, CN258, CN258C/D, CN259, CN260 of The Taipei Mass Rapid Transient System – Nan-Kang East Line* (in Chinese), Moh and Associates, Inc., 2001.

[5] *Final Geotechnical Consultancy Report of Lot CN25C of The Taipei Mass Rapid Transient System – Nan-Kang East Line* (in Chinese), Moh and Associates, Inc., 2000.

[6] Woo, S. M. and Moh, Z. C., "Geotechnical Characteristics of Soils in the Taipei Basin", *10th Southeast Asian Geotechnical Conference*, Special Taiwan Session, 1990.