Geotechnical Aspects of Underground Construction in Soft Ground – Yoo, Park, Kim & Ban (Eds)
© 2014 Korean Geotechnical Society, Seoul, Korea, ISBN 978-1-138-02700-8

Use of 3D finite element method for back study of a failed basement excavation in soft clay

S.S. Lin
Professor, Department of Harbor/River Engineering, National Taiwan Ocean University, Keelung, Taiwan

J.C. Liao
Associate Researcher, Taiwan Construction Research Institute, New Taipei City, Taiwan

S.D. Yang
General Manager, Artech Engineering Service Inc., Taipei, Taiwan

ABSTRACT: In this paper, the three-dimensional finite element software, 3-D PLAXIS, is used to back study possible failure reason of a deep excavation project in soft clay. The building project is located in Taipei covering an area of 200 m by 200 m. General ground condition consists approximately 29 m of soft with SPT-N of 1 to 5 only. The average elevation of the construction site is plus 4.4 m. It's planned to excavate down to minus 9.5 m. It is planned to leave a berm into 1(V) by 2.5 (H) slope along with installation of shotcrete and 70 cm of drilled shafts along the east side of the site. Two horizontal layers of H pile will then be installed when reaching the second basement level. Slope failure occurs on the east side during excavation. Comparison between the 3D FEM analytical results and instrumentation measurement data showed asymmetric excavation and misjudgment of the elevation of bedrock are the possible reasons to cause the failure of the excavation.

1 INTRODUCTION

The building project in this study is located at north-east side of Taipei city covering an area of 200 m by 200 m. Average elevation of the surroundings of the site is +4.4 m. Planned excavation depth is at elevation −9.5 m. The general subsurface condition of the building site consists approximately 29 m of low plasticity clay, with SPT-N value between 1 and 5, underlain by shale/sandstone bedrock. However, at the depth of 4 m and 10 m below ground surface, the soil condition is much like low plasticity silt.

Along the east side, excavation is planned to be bermed excavation into 1 (V) by 2.5 (H) slope along with shotcrete and 70 cm of drilled shafts installed for slope stability. When reaching the B2F (basement second floor) level of the building structure, two horizontal layers of H-type pile are then installed further excavation (Figure 1). Slope failure occurs on this east side during excavation. A preliminary study using the method proposed by the first author (Lin and Liao 2006; Lin et al. 2005) has been focusing on the drilled shaft performance (Lin et al. 2013). In this paper, the three-dimensional finite element software, 3-D PLAXIS, is used to back study possible failure reason of the aforementioned deep excavation project in soft clay.

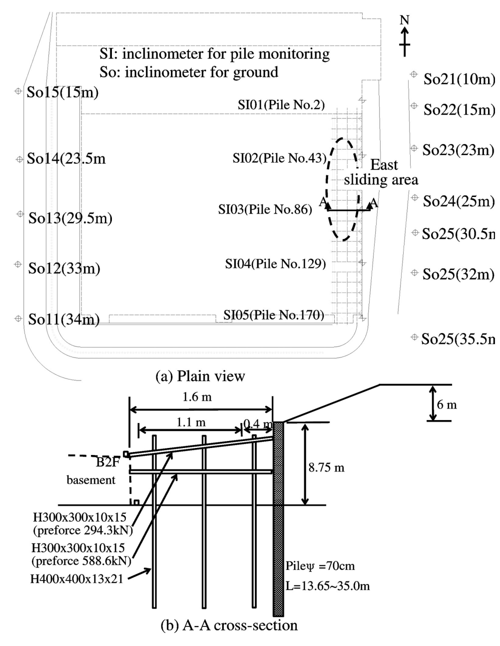

Figure 1. Site arrangement (redraw from Lin et al. 2013).

2 OBSERVATION

188 drilled shafts with diameter of 70 cm, 13.6 m to 35 m long spaced at 70 cm center-to-center, socket on an average of 3 m into the rock, were installed along the east side of the project site before excavation.

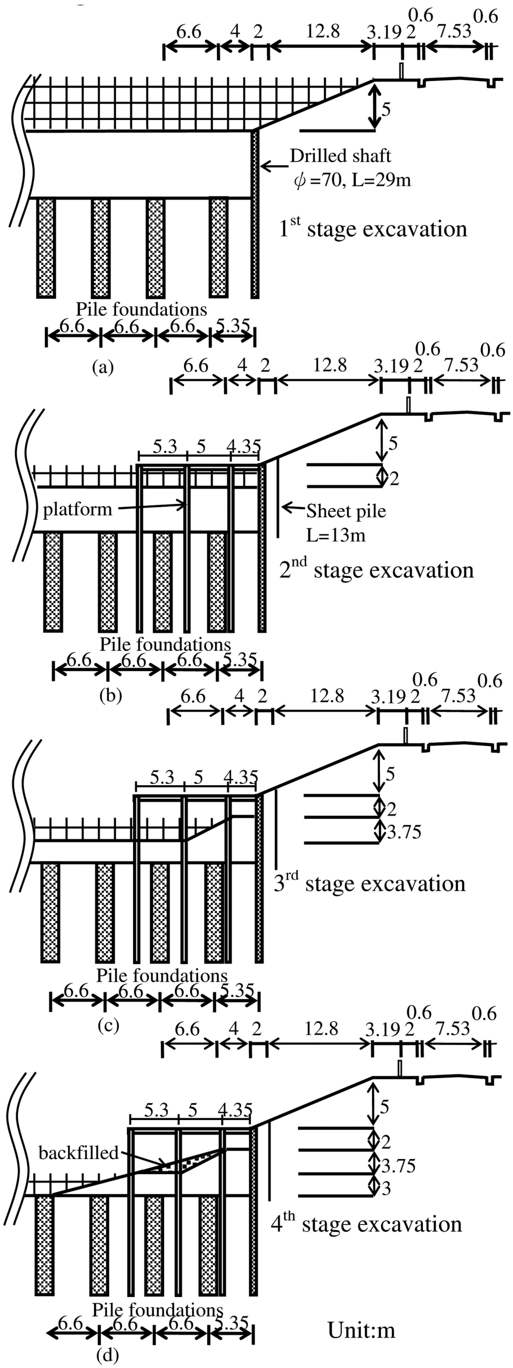

Figure 2. Four stages excavation on the east side (redraw from Lin et al. 2013).

Excavation was planned at four stages via bermed excavation method down to a depth of 13.9 meters (Figure 2). The top 5 m of the ground was removed for the first stage of excavation. Local seepage condition was observed at the silt layer, hence a row of 13 m long sheet-pile wall (YSP type III) was installed

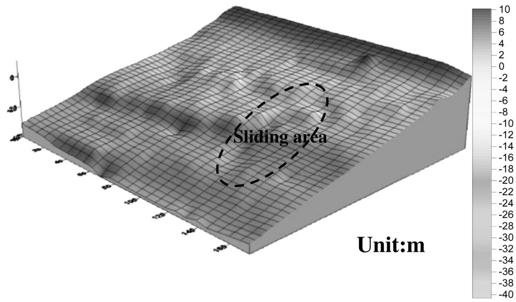

Figure 3. Bedrock elevation at sliding area.

behind the drilled shaft to seal the water. In addition, chemical churning pile (CCP) was used to improve the ground down to 29 m behind the drilled shaft before the construction platform was constructed. The construction platform was installed after the first stage of excavation was reached the planned elevation. Another 2.15 m of ground was removed for the second stage of excavation.

For the third stage, excavation was initially planned to remove another 6.75 m of soil but large ground movement occurred after only 3.75 m thick of soil was removed. Since an open crack of 1.5 cm to 2.5 cm wide was observed on the ground surface behind the sheet-pile wall 3 m thick of soil-cement mixture was immediately backfilled into the excavated area. In addition, the excavated slope surface angle was made smaller before the next stage of excavation. However, when excavation continued and an additional 3 m of soil was removed, another severe ground movement occurred that resulted in large settlement of the ground surface outside the project area. Backfilling in the excavation side was conducted again to stop the damage. Maximum lateral displacement at the head of drilled shafts from inclinometer was measured at 39.9 cm at the beginning of the incident the head displacement stabilized at 42.61 cm.

Based on the availability of investigation information after failure occurred, a 3D view of the variation of bedrock depth is interpreted and is shown in Figure 3. A local trough appearing on the east side in the figure, which was not observed in designing, matches the location of the incident. In addition, more detail ground investigation is also conducted. Comparison of the ground investigation results between the design stage and the post design stage is shown in Table 1.

3 THREE DIMENSIONAL NUMERICAL ANALYSIS

To further understand the performance of the drilled shaft stabilized slope, the available computer code PLAXIS was also used to simulate excavation step effects on drilled shafts and on ground movement. Layout of the drilled shafts along the east side for numerical analysis is shown in Figure 4. Failure in the excavated area after the numerical simulation is

Table 1. Soil properties (Lin et al. 2013).

(a) Properties used for design

Depth (m)	Soil type	SPT-N	γ_t (kN/m^3)	ω (%)	LL/PI (%)	Cc/Cs	C′/C (kPa)	ϕ'/ϕ (°)	S_u (kPa)
0∼3.2	SF/CL	3.9	18.54	25.9	37.3/12.3		0.0/14.72	24/16	44.15
3.2∼19.5	CL/ML	2.7	17.56	42.2	40.0/15.8	0.42/0.04	0.0/9.81	22/15	34.33
19.5∼25.0	CL/ML	5.0	18.44	34.8	41.4/17.6	0.42/0.04	0.0/24.53	27/16	58.86
25 below	Sandstone		20.7				−/58.86	−/28	392.4

(b) Properties used for PLAXIS analysis

Depth (m)	Soil type	γ_t (kN/m^3)	C′ (kPa)	ϕ' (°)	C (kPa)	ϕ (°)	S_u (kPa)
0∼5	SF/CL	18.15	–	–	–	–	17.76
5∼10.9	SM/ML/CL	17.36	0	28	–	–	–
10.9∼14.6	CL	17.07	–	–	16.67	12	22.76
14.6∼19.1	CL	17.07	–	–	9.81	11	16.97
19.1∼22.1	CL	17.07	–	–	9.81	11	25.41
22.1∼25.2	CL	17.07	–	–	9.81	11	27.47
25.2∼29	SM/CL	18.84	0	32	–	–	–
29 below	Sandstone	20.7	58.86	28	–	–	–

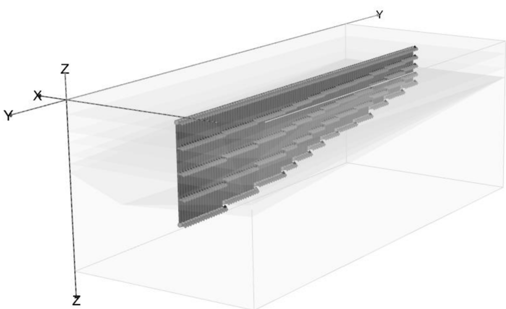

Figure 4. Numerical analysis model of PLAXIS 3D.

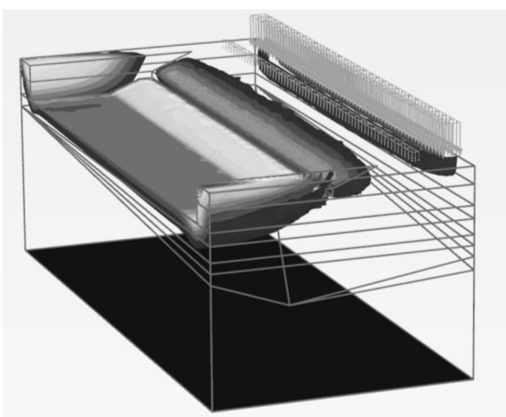

Figure 5. Numerical results of ground displacement.

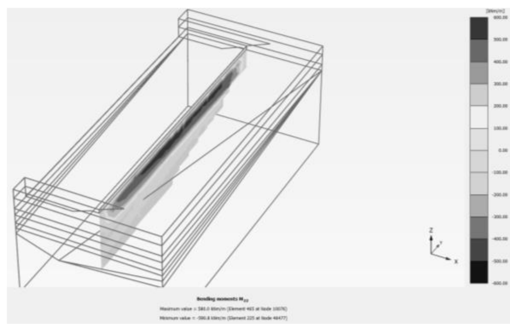

Figure 6. Numerical results of bending moments.

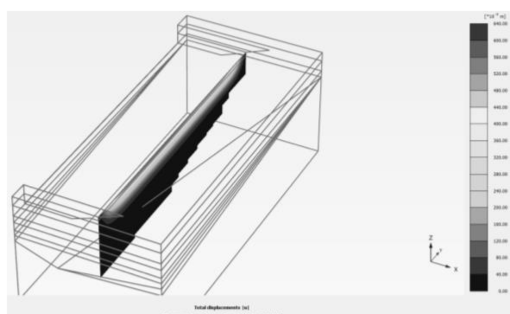

Figure 7. Numerical results of drilled shafts displacement.

shown in Figure 5. In addition, the bending moment and the displacement of the failed drilled shafts are shown in Figure 6 and 7, respectively. Based on the numerical analysis, it shows the sliding occurs along the excavated surface and results in the large displacement of the drilled shafts. Figure 8 shows the displacement of the ground at GL. −14 m and the effect of the trough of the bedrock.

The inclinometer was used to monitor the drilled shaft displacement during different stages

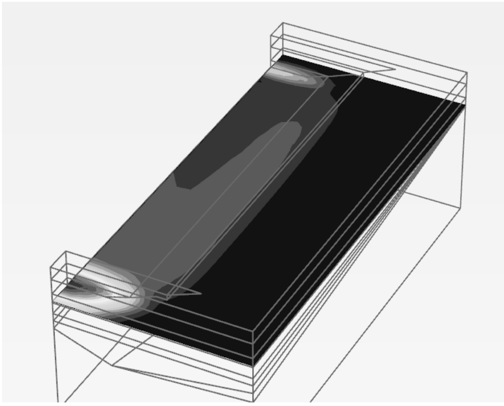

Figure 8. Numerical results of displacement at depth −14.

Table 2. Plaxis analysis versus. inclinometer data.

Excavation stage	Inclinometer data (mm)	2-D Plaxis analysis (mm) (Lin et al. 2013)	3-D Plaxis analysis (mm)
1st stage excavation	23.78	35.65	–
2nd stage excavation	110.67	265.82	–
3rd stage excavation	156.29	265.82	–
4th stage excavation	398.84 (2005/07/15) 670.82 (2005/7/25)	357.63	638.23

of excavation From stage 1 to stage 4 of excavation, the shaft displacement varied from 23.78 mm, 83.40 mm, 156.29 mm, to 398.84 mm (2005/07/15) until 670.82 mm (2005/07/25) (Table 2). Based on monitoring data, the inclinometer took ten days to reach stabilization after sliding at the fourth stage excavation occurred. In addition, comparison between the results of 2D and 3D analysis as given in Table 2, the trough shape of the bedrock can only be captured by 3D analysis. Hence, 3D analysis can simulate better ground condition than that of the 2D analysis.

4 CONCLUSIONS

From the study in this paper, the following observations can be summarized:

1) Soil strength used in the design stage was overestimated. Consequently, the size and the rock socket length of the drilled shaft design were insufficient to stabilize the excavated slope. Also, even when sheep-piles were installed and CCP was adopted for ground improvement after the first stage of excavation, the sheet pile penetrated depth and the ground improved depth were apparently not long enough or deep enough to stop further failure due to subsequent excavations.
2) A trough on the bedrock surface of the incident zone was not observed in the design stage of ground investigation. In addition, the bedrock surface was highly weathered, which could not provide enough strength for the socket part of the drilled shaft. Hence, drilled shafts installed around the incident area were all similar to floating piles and could not function properly in excavation slope stabilization.
3) 2D analysis results cannot capture the variation of the ground bedrock elevation at the sliding area. Hence, comparing to the monitoring data, 3D numerical study results are more reasonable than that of the 2D analysis.

ACKNOWLEDGEMENT

This study was partly supported by the National Science Council, Taiwan, under grant number NSC 98-2211-E-019-043-MY2. Grateful appreciation is expressed for this support.

REFERENCES

Liao, J.C., Lin, S.S., and Yang, S.D. (2013) "Failure Investigation of a Drilled Shaft Stabilized Slope" *Proceedings of ICE: Forensic Engineering*, 166(1), 17–26.

Lin, S.S. and Liao, J.C. 2006. Lateral Response Evaluation of Single Piles Using Inclinometer Data. *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, 132(12): 1566–1573.

Lin, S.S., Liao. J,C., Chen, J.T. and Chen, L. 2005. Lateral Performance of Piles Evaluated via Inclinometer Data. *Computers and Geotechnics*, 32(6): 411–421.

PLAXIS 3D. 1998. *Plaxis Finite Element Code for Soil and Rock Analyses*. PLAXIS BV, Delft, The Netherlands.