Finite element analysis of failure of deep excavations in soft clay

Chang-Yu Ou¹ and Tuan-Nghia Do²

1) Professor, Department of Construction Engineering, National Taiwan University of Science and Technology, 43 Keelung, Taipei 106, Taiwan.

2) Ph.D Student, Department of Construction Engineering, National Taiwan University of Science and Technology, 43 Keelung, Taipei 106, Taiwan.

ABSTRACT

Two failure deep excavations in soft clay were investigated in this study using the finite element method (FEM) with reduced shear strength. A comprehensive model of support system was built as regard the existence of wall, struts, and center posts. In order to determine the causes of collapse of the excavations, both elastic and elastoplastic behaviors of support system were analyzed. Result shows that failure of the excavations starts from large soil heave at the final excavation grade as considering an elastic behavior of support system and from yielding of support system if its behavior is elastoplastic. A failure mechanism is proposed for deep excavations in soft clay. Due to an upward movement of center posts, secondary bending moment on struts will reduce their capacity, originally designed for bearing axial load. Yielding of struts is followed by yielding of wall and then failure of excavation. Factors of safety estimated by FEM with modeling elastoplastic support system are also in good agreement with stability of the excavations observed in the field.

Keywords: excavation, stability analysis, finite element method

1 INTRODUCTION

Although conventional methods, including Terzaghi's method, Bjerrum & Eide's method, the slip circle method, and the push-in gross pressure method are able to give reasonable estimates of stability of excavations, use of these methods could not help practical engineers to understand sufficiently a failure mechanism of excavations because plastic behaviors of structural elements are not considered and a rough assumption of the failure surface is made in these methods. The FEM with reduced shear strength, on the other hand, is capable of modeling structural elements and automatically seeking for the failure surface of soil. This method, therefore, has been widely applied to stability analysis of deep excavations by many researchers. However, for sake of simplicity, so far, the elastic behavior of structural elements is often simulated. Results are acceptable but the failure mechanism of excavations modeled is not really close to the actual one as using such structural elements.

In this study, the FEM with **reduced** shear strength using elastoplastic structural elements is employed to analyze the stability of two failure excavations in Taipei. A failure mechanism is then proposed for braced excavations in soft clay.

2 CASE STUDIES

2.1 Taipei Rebar Broadway Case

The profile of excavation sequence and subsoil layers of Taipei Rebar Broadway case are plotted in Fig. 1.

The analysis incorporates the Mohr-Coulomb model implemented in the computer program, which describes the stress-strain behavior of soil as a linear elastic

perfectly plastic relationship. Clay and sand were simulated using undrained and drained materials, respectively. The input parameters of soil were referred from Do et al. (2013).

Struts, center posts, and walls were modeled with elastoplastic plate elements. For the reinforced concrete wall, the input stiffness parameters were determined using the Young's modulus $E = 2.1 \times 10^7$ kN/m² meanwhile the strength parameters (M_p and N_p) were calculated using *XTRACT*, a commercial program. For the steel struts and center posts, the input parameters were calculated using $E = 2.04 \times 10^8$ kN/m² and yield stress $\sigma_y = 250$ MPa. Connections among plate elements were modeled as hinges. A 140 × 44.7-m two-dimensional FE model was used for analysis with the aid of *PLAXIS 2DTM*.

The strength reduction procedure was performed for the excavation at the final stage, at which soil strength is reduced through increasing the strength reduction ratio (SR) until numerical solutions diverge. The maximum SR value was obtained at 1.00 and this value was taken as the factor of safety of the excavation. To investigate behaviors of soil and structural elements just before divergence of numerical solutions, two additional calculations corresponding to the ΣM_{stage} values of 0.98 and 0.99 were performed, at which ΣM_{stage} is the ratio of the load applied successfully in calculation to the required one.

The ΣM_{stage} value of 1.00 indicates that the load required at the final stage is calculated fully. On the other hand, the SR values lower than 1.00 could not be applied in this investigation. In principle, application of SR causes the soil strength to change and results in the

deformations of soil and structural elements due to soil weight. As a matter of fact, if SR is smaller than 1.00, soil strength increases, no additional deformation occurs; hence, the deformations of soil and structural elements are the same as those when SR = 1.00.

Fig. 1. Profile of excavation sequence and subsoil layers of Taipei Rebar Broadway case.

Fig. 2. Wall deflection and soil heave at stage 5 (final stage) of Taipei Rebar Broadway case during loading procedure as using elastoplastic support system.

Fig. 2 shows the wall deflection and the soil heave at the final excavation stage for the ΣM_{stage} values of 0.98, 0.99, and 1.00. It is observed that the wall deflection and the soil heave increase with ΣM_{stage} and exhibit a rapid development corresponding to the ΣM_{stage} value of 1.00. The large maximum deformations of soil and wall herein do not necessarily imply the actual values of deformation but well indicate the divergence of numerical solutions.

Fig. 3 shows the interaction diagrams of internal forces (bending moment, M and axial force, N) of structural elements (struts and walls) for different excavation stages. A boundary line is a combination of the M_p and N_p values. Within the area created by the boundary lines, a structural element will exhibit an elastic behavior meanwhile on the boundary lines it shows a plastic behavior. There are some points which appear slightly outside the area created by the boundary lines. It is due to the fact that the M and N values are taken at the nodes by extrapolating the values at the stress points, which may cause a small error. As shown in Fig. 3, when excavation proceeds, the strut system starts to yield from the 4th stage at the 1st and 2nd layers whereas the wall remains an elastic behavior. At the final

excavation stage (SR = 1 and ΣM_{stage} = 1), all the strut layers together with the wall yield and a rapid increase in the wall deformation and soil heave is observed, as shown in Fig. 2. Therefore, the yielding of the structural elements is followed by the large movement of soil and wall towards the excavation zone and then the failure of the excavation.

Fig. 3. Interaction diagram of internal forces of structural components of Taipei Rebar Broadway case as using elastoplastic support system: (a) strut; (b) wall.

Fig. 4. Wall deflection and soil heave at stage 5 (final stage) of Taipei Rebar Broadway case during strength reduction procedure as using elastic support system.

As shown in Fig. 4, when elastic structural elements are employed, although the wall deformation and the soil heave exhibit very large deformations up to 3 m and 7 m, respectively, a rapid increase is only observed in the soil heave at the SR_{max} value of 1.84937. Note that the

SR value with five digits after the decimal point was employed to investigate accurately the time that numerical solutions diverge. Also, since both the wall and strut remain stable or still behave elastically meanwhile the soil heave is enormous, the failure of the excavation occurs due to basal heave.

Fig. 5. Profile of excavation sequence and subsoil layers of Taipei Shi-Pai case.

Fig. 6. Wall deflection and soil heave at stage 4 (final stage) of Taipei Shi-Pai case during loading procedure as using elastoplastic support system.

2.2 Taipei Shi-Pai Case

The profile of excavation sequence and subsoil layers are plotted in Fig. 5.

The simulation procedure was performed similarly to that at the previous case. As using elastoplastic structural elements, the computation failed at the final excavation stage. The maximum multiplier ΣM_{stage} at the final excavation stage is 0.6734. The wall deformation and soil heave at different ΣM_{stage} values are plotted in Fig. 6. Both the deformations increase with ΣM_{stage} . It is noted that at the maximum ΣM_{stage} of 0.6734, a rapid increase occurs at both the wall deformation and soil heave with about 2.4 m in magnitude. The excavation is hence close to failure at this maximum ΣM_{stage} value.

Fig. 7 shows the interaction diagrams of struts and walls for different excavation stages. When excavation reaches the third stage, the entire strut layers yield already but the wall remains an elastic behavior. At the maximum ΣM_{stage} of 0.6734, all the wall and strut

layers are in plastic states. The excavation collapses then with a slightly increase in ΣM_{stage} (e.g., $\Sigma M_{stage} = 0.6735$). The yielding of the support system, therefore, is the cause of the failure of the excavation.

Fig. 7. Interaction diagram of internal forces of structural components of Taipei Shi-Pai case as using elastoplastic support system: (a) strut; (b) wall.

Fig. 8. Wall deflection and soil heave at stage 4 (final stage) of Taipei Shi-Pai case during strength reduction procedure as using elastic support system.

As using elastic structural elements, the program succeeded in calculating all the excavation stages. The maximum SR value at the final excavation stage is 1.54807. The wall deformation and the soil heave during the strength reduction procedure are plotted in Fig. 8. At

Table 1. Factors of safety by various methods.

Case history	Finite element method		Conventional method			
	Elastoplastic structural elements	Elastic structural elements	Terzaghi's method	Bjerrum & Eide's method	Slip circle method	Push-in gross pressure method
Taipei Rebar Broadway case	1.00	1.84937	1.088	0.952	0.809	0.755
Taipei Shi-Pai case	$\Sigma M_{stage} = 0.6734$	1.54807	0.901	0.792	0.732	0.721

the SR_{max} value, there is only a rapid increase in the soil heave with the maximum magnitude of 5 m. Also, since the yielding of the support system cannot occur, it is concluded that the failure of the excavation is due to basal heave.

Fig. 9. Failure mechanism of braced excavations in soft clay.

3 DISCUSSION

Based on results of the stability analyses of the two case histories, it can be observed that when elastoplastic structural elements are employed, the strut layers start to yield earlier than the wall and the failure of the excavations occurs right after both struts and walls yield. As using elastic structural elements, on the other hand, the support system never yields and the large soil heave causes the failure of the excavations. Since structural elements often exhibit an elastoplastic behavior rather than an elastic one, the failure of the excavations modeled with elastoplastic structural elements is more reasonable. A failure mechanism of a braced excavation in soft clay is proposed as shown in Fig. 9. The failure of an excavation is mainly due to the fact that center posts are pushed upward to a certain distance, which would cause the bending of the struts, originally designed only for bearing the axial load. The bending of, or the secondary moment on, the struts would reduce their axial load capacity. Once the struts yield, the walls will yield then, followed by a large movement of surrounding soil towards the excavation zone, and then the failure of the excavation. **The proposed mechanism is applicable as long as the soft clay is thick enough and strut section and wall thickness are designed under normal conditions.**

Table 1 summarizes factors of safety estimated by both the FEMs and the conventional methods, including Terzaghi's method, Bjerrum & Eide's method, slip circle method, and push-in gross pressure method. For the two

case histories, the results of the FEM using elastoplastic structural components are generally close to unity, hence, they are in good agreement with the field observations because the cases ever experienced failure. The FEM using elastic structural components gives the FS values of 1.84937 and 1.54807 for the Taipei Rebar Broadway case and the Taipei Shi-Pai case, respectively, which overestimate the stability of the cases. Since the results of the conventional methods are generally less than or equal to unity, these methods give conservative estimates of stability of the excavations. The FS values calculated by Terzaghi's method, Bjerrum & Eide's method, slip circle method, and push-in gross pressure method reduce gradually in this order.

4 CONCLUSION

Based on the study presented herein, the following conclusions may be drawn:

- With modeling elastoplastic structural elements (struts, center posts, and walls), results of the stability analysis indicate that the failure of the excavations at the case histories is due to yielding of the bracing systems. As using elastic structural components, the excavations collapse because of the large soil heave at the bottom of the excavation.
- A failure mechanism of a braced excavation in soft clay is proposed. An upward movement of the center posts causes the occurrence of the secondary bending moment on the struts, which are initially subject to the axial force. Once struts yield, walls will yield, followed by the large movement of surrounding soil towards the excavation, and then the failure of the excavation.
- The FS values calculated by the FEM using elastoplastic structural components are in good agreement with the field observations of the case histories. Therefore, the method can be used to estimate stability of the excavations in soft clay. The FEM using elastic structural components, on the other hand, overestimate stability of the cases.

REFERENCES

- Do, T. N., Ou, C. Y., and Lim, A. (2013). "Evaluation of factors of safety against basal heave for deep excavations in soft clay using the finite element method." *J. Geotech. Geoenviron. Eng.*, 139(12), 2125-213.