Journal of Zhejiang University-SCIENCE A (Applied Physics & Engineering)
ISSN 1673-565X (Print); ISSN 1862-1775 (Online)
www.zju.edu.cn/jzus; www.springerlink.com
E-mail: jzus@zju.edu.cn

Excavation collapse of Hangzhou subway station in soft clay and numerical investigation based on orthogonal experiment method*

Xiao-nan GONG[1,2], Xue-chan ZHANG[†‡1,2]
([1]*MOE Key Laboratory of Soft Soils and Geoenvironmental Engineering, Zhejiang University, Hangzhou 310058, China*)
([2]*Institute of Geotechnical Engineering, Zhejiang University, Hangzhou 310058, China*)
[†]E-mail: zdtmzhangxc@126.com

Received May 4, 2012; Revision accepted Aug. 30, 2012; Crosschecked Sept. 18, 2012

Abstract: This paper studies the excavation collapse at the Xianghu subway station on Hangzhou metro line 1. The objective is to present an overview of this case study and discuss the cause of the failure. Through field investigation and preliminary analysis, the reasons for the excavation collapse were the misuse of the soil parameters, over excavation, incorrect installation of steel struts, invalid monitoring data, and inadequate ground improvement. Finally, a small strain constitutive model was used for further analysis. In order to estimate damage efficiently, the orthogonal array (OA) was introduced for screening the key factor in the numerical experiments. Six estimated indexes including deformations and internal forces of the excavation were taken, and the effectiveness of four factors which may cause the collapse was evaluated. Through numerical experiments and interaction analysis, it is found that the deformation and internal force can be well controlled by jet grouting of the subsoil under the final cutting surface, but increasing the improvement ratio of the jet grouting cannot help optimize the excavation behavior efficiently, and without jet grouting and the fourth level struts, the deformation and internal force of the excavation in this case will far surpass the allowable value.

Key words: Collapse, Case study, Numerical analysis, Orthogonal experiment method, Interaction
doi:10.1631/jzus.A1200120 **Document code:** A **CLC number:** TU473

1 Introduction

In many big cities, traffic jams bother everyone living in the city. To relieve this situation, extensive metro systems have been rapidly constructed in these cities. Unfortunately, serious construction accidents have happened in recent years (Jebelli *et al.*, 2010; Swanson and Larson, 2010; Kim *et al.*, 2010), including some subway station construction accidents (Li and Wang, 2000; Ferrari, 2007), which resulted in huge loss of lives and property. This paper introduces a case history of the excavation collapse of Hangzhou Xianghu subway station. Twenty-one people died and many automobiles were destroyed in the collapse.

To explore the cause of the collapse and to prevent similar disasters, the collapsed excavation was studied extensively through field investigation and preliminary analysis. The subsequent study uncovered a series of poor engineering decisions which lead to the failure and collapse, including misuse of the soil parameters, over excavation, incorrect installation of steel struts, invalid monitoring data, and inadequate ground improvement.

Numerical analysis was successfully used for inverse analysis and optimization for many projects (Ou *et al.*, 2011; Hashash *et al.*, 2011; Wang and Zhou, 2011; Zhang *et al.*, 2011). The computer program Plaxis V8.5 was used to further investigate the reasons of the collapse. As for advanced analysis, small strain models were desired (Benz, 2007). Sixteen

‡ Corresponding author
* Project (No. 51078377) supported by the National Natural Science Foundation of China

numerical experiments were carried out based on the orthogonal experiment method (Hou and Wang, 1985) to evaluate the effectiveness of the strut stiffness, surcharge on the Fengqing Road, jet grouting improvement ratio of the subsoil under the final cutting surface, and the cutting surface location of the fourth step excavation. The primary cause of the failure was revealed, and the interaction between the jet grouting improvement ratio and location of the fourth step excavation to the deformation and internal force of the excavation was analyzed.

2 Outline of the collapsed excavation

The Xianghu subway station is located in the Xiaoshan district of Hangzhou, China. This station was constructed using the bottom-up construction method in eight excavations from north to south. The collapsed position is the second excavation on the northern side (N2 excavation), which is near the junction of Fengqing Road and Xiangxi Road as shown in Fig. 1. The excavation occupies a planning area of about 2313 m^2, with approximately 21.5 m in width and 107.6 m in length. The maximum digging depth is up to 16.2 m.

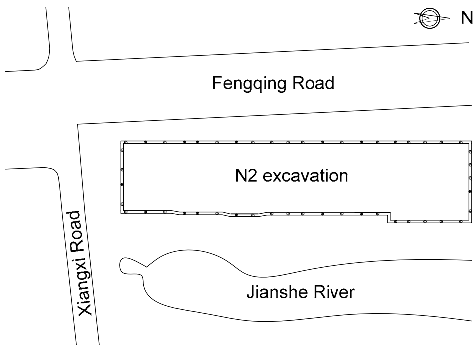

Fig. 1 Plan view of the N2 excavation

The N2 excavation is located on 50 m thick soft soil, overlying a stiff sand/gravel formation. The groundwater table is 0.5 m. As the confined aquifer is very deep in the subsoil compared with the excavation depth, the confined water pressure is not considered. Fig. 2 displays the typical soil profile of the N2 excavation.

A reinforced concrete diaphragm wall with four levels of steel struts (Φ609 mm pipe strut with a thickness of 16 mm) is used as the retaining system. The horizontal spacing of the struts is 3 m. The diaphragm wall is 0.8 m in thickness and extends from the ground surface down to a depth of 33.0 m. Following the construction of the diaphragm walls, bored piles, steel lattice columns, and ground improvement, the N2 excavation should be subsequently excavated and completed in nine stages, as shown in Fig. 2.

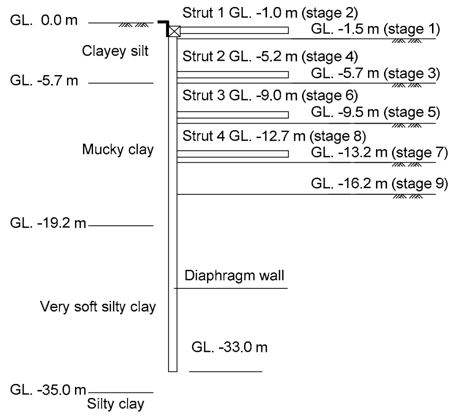

Fig. 2 Excavation and subsurface soil profiles of the N2 excavation

The N2 excavation collapsed at 15:15 on Nov. 15, 2008. Struts were destroyed and the diaphragm wall broke, as shown in Fig. 3. Lots of silt beneath the Fengqing Road rushed into the N2 excavation, resulting in a 7.5 m base heave in the excavation and a 6.5 m deep hole on the Fengqing Road. The water from Jianshe River and damaged pipelines nearby caused flooding at the collapsed site.

Fig. 3 View of the collapsed site for the N2 excavation

3 Preliminary analyses for the failure

After field investigation and preliminary analysis, the reasons of the excavation collapse were concluded as follows.

3.1 Misuse of the soil parameters

Borings were drilled more than 5 m away from the collapsed west diaphragm wall. This area was not disturbed after the failure. Samples were obtained using thin-walled tube sampling by the Hangzhou Exploration & Surveying Design Institute (HESDI). Geotechnical tests were conducted by the Shanghai Geotechnical Investigations & Design Institute Co., Ltd (SGIDI) and the Civil Engineering Testing Center of Zhejiang University (CETCZU). The soil parameters from the CETCZU and SGIDI were compared with the original design parameters from the Institute of Geology & Mineral Resources Exploration of Zhejiang (IGMREZ) in Table 1.

Standard values of consolidated undrained triaxial test (CU) are needed for design purposes according to GB 50021-2001. The soil samples obtained by the IGMREZ are not enough to determine the strength parameters of the subsoil. In the original design, the average value of consolidated quick direct shear test (CQ) was taken, which may also have caused a mistake. As the average value of the strength parameter is greater than the standard value, the use of the average value is unsafe. Table 1 indicates that the strength parameters of the three institutes are much different to each other. The data of IGMREZ give unreasonable strength parameters that obviously disobey the general principles of soil mechanics, such as $c<c'$ (CU), φ' of mucky clay is only 10.8°, etc.

3.2 Over excavation

The N2 excavation is carried out in six zones. The plan view and profile of the collapsed excavation is shown in Fig. 4. It can be seen from Fig. 4 that the fifth step excavation is set out just after the fourth step excavation, without installing the fourth level struts in zones 4 and 5 (as shown in profile A-A′). This performance is not consistent with the construction sequence as shown in Fig. 2. The maximum deflection of the west wall is much too big (as shown in profile B-B′), which leads to an extreme axial force rise in the struts, and the bending moment of the wall increases rapidly. In zones 2 and 3, excavation is finished with

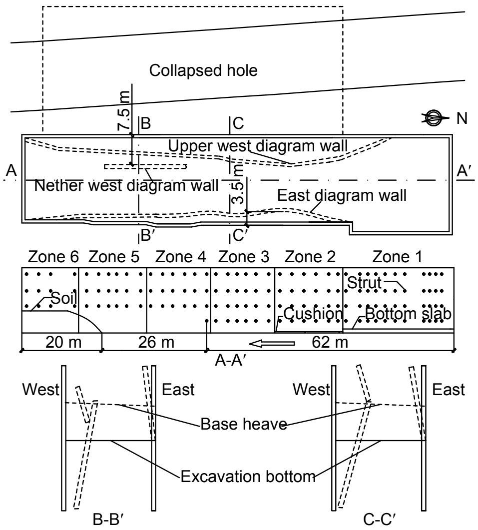

Fig. 4 Plan view and profile of the collapsed excavation

Table 1 Comparison of the soil parameters from IGMREZ, CETCZU, and SGIDI

Soil type	Institute	w (%)	e	CQ		CU			
				c (kPa)	φ (°)	c (kPa)	φ (°)	c' (kPa)	φ' (°)
Clayey silt	IGMREZ	30.6	0.95	3.1 (6.8*)	32.5 (36.1*)	–	–	–	–
	CETCZU	41.7	1.17	9.8	18.7	15.7	8.3	5.2	28.5
	SGIDI	33.2	0.96	4.1	27.4	8.8	13.4	1.8	30.4
Mucky clay	IGMREZ	50.5	1.41	12.5 (14.4*)	8.4 (9.8*)	9.0 (20.3*)	8.1 (12.1*)	10.3 (22.0*)	10.8 (16.4*)
	CETCZU	50.2	1.43	13.5	11.1	22.3	8.3	4.8	23.4
	SGIDI	50.1	1.42	13.3	10.6	12.5	13.1	2.3	24.5
Very soft silty clay	IGMREZ	44.5	1.32	12.6 (14.4*)	12.1 (14.4*)	–	–	–	–
	CETCZU	45.4	1.29	16.2	13.1	19.7	11.3	2.6	26.1
	SGIDI	47.9	1.38	12.5	13.7	12.4	12.8	3.2	24.4
Silty clay	IGMREZ	31.6	0.99	11.0 (11.0*)	26.0 (26.0*)	–	–	–	–
	CETCZU	32.2	0.92	12.9	18.5	13.1	20.3	1.3	35.0
	SGIDI	37.6	1.09	12.5	16.5	15.5	19.3	3.4	29.7

Data with ‘*’ mean average values, while the others mean standard values. w is the water content, e is the void ratio, CQ is the consolidated quick direct shear test, CU is the consolidated undrained triaxial test, c is the cohesion, φ is the angle of internal friction, c' is the effective cohesion, and φ' is the effective angle of internal friction all four levels of struts. The west wall deflection here is not that big (as shown in profile C-C′). By comparing the profiles B-B′ and C-C′, it follows that the fourth level struts are necessary for this case.

3.3 Incorrect installation of steel struts

The designed connection of the pipe strut and coupling beam is shown in Fig. 5a. Pipe struts should be fixed on the coupling beam with I beam (I14). In fact, some pipe struts were simply fixed on the coupling beam with rebar, as shown in Fig. 5b, while others were just located on the coupling beam without any fixtures. As the connection of pipe strut and coupling beam was so weak, intermediate supports did not work, and the effective length of each strut increased. Bearing capacity of the pipe strut declined 32.5% from 5265 to 3555 kN according to GB 50017-2003. In addition, struts were just located on the bracket without effective welding, which resulted in a poor connection between the steel strut and the diaphragm wall, so when there is an impact load or the wall deflection is too big, the struts will be ineffective. All of these effects will lead to a decrease in the stability of the retaining system.

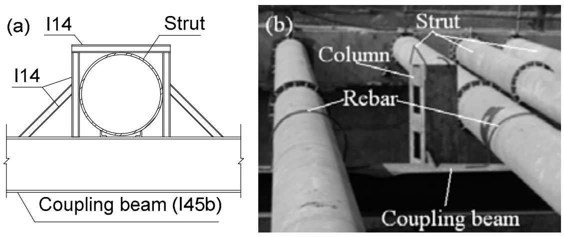

Fig. 5 Designed connection (a) and actual connection (b) of pipe strut and coupling beam

3.4 Invalid monitoring data

The number of instruments for the designed and actual monitoring layouts is compared in Table 2. Since some of the instruments were destroyed during the excavation, the number of instruments in the actual monitoring layout was much less than designed.

Table 2 Comparison of designed and actual monitoring layouts

Instrument	Designed	Actual
Settlement point	12	8
Inclinometer	10	8
Load cell	22	4
Heave gauge	5	–

According to the actual monitoring layout, the farthest settlement point was set at 7.5 m away from the excavation in this case. The settlement of Fengqing Road could not be presented well. Through data recovery, the following message was obtained: the maximum deflection of the east wall was 94.5 mm on Oct. 27, 2008, which far surpassed the allowable value according to GB 50497-2009. However, in the monitoring report data were tampered to be normal. Insufficient monitoring instruments to layout and ignoring the unusual performance of the excavation also contributed to the failure.

3.5 Inadequate ground improvement

The mucky clay is 13.5 m thick as shown in Fig. 2, which primarily affects the excavation behavior in this case. In the original design, the soil under the final cutting surface was suggested grouting using 1 m diameter by 3 m deep boreholes distributed around the excavation area, and dewatering should be taken four weeks before excavation.

Finally, artesian wells were adopted for dewatering four weeks before excavation, and only zone 1 was reinforced. As the permeability (k) of clay is just 10^{-6}–10^{-7} cm/s, and the spacing time between dewatering and excavation was just 28 d, the subsoil could not consolidate well, and was still too soft to stabilize the walls and foundation of the excavation during construction.

4 Numerical analysis

4.1 Constitutive model and parameters

To further investigate the reason of the collapse, the N2 excavation was analyzed numerically in this section with the program PLAXIS V8.5. This analysis was simplified with a plane strain problem since the length of the excavation was quite considerable when comparing with the width of the excavation.

Fig. 6 shows the 2D finite-element mesh for the analysis of the center section of the N2 excavation. The bottom of the mesh, 50 m from the ground surface, is set as a fixed boundary. The vertical boundaries for the two sides, which are set 80 m behind the wall (almost four times the maximum excavation depth), are restricted in the horizontal movement. A surcharge of 20 kPa is imposed on the ground surface at a distance of 6 m to 36 m from the excavation. After each level excavation, the water pressure of the exposed cutting surface is set as zero. The water pressure is generated by groundwater calculation. Structural elements are modeled with linear-elastic materials, while the soil is simulated with hardening-soil small model (Benz, 2007). The parameters adopted in this model and its notations are listed in Table 3.

For the initial numerical model, the stiffness of the diaphragm wall and bored pile is assumed with a reduction of 20%, and a reduction of 40% for the steel struts, as a result of wall cracking, bending, and incorrect installation of struts (Ou, 2002). For the plane strain analysis, the structural parameters of the bored pile (Φ800 mm with a pile spacing of 6 m) should be transformed to the value per unit width. The input parameters of the structures for the initial numerical model are shown in Table 4.

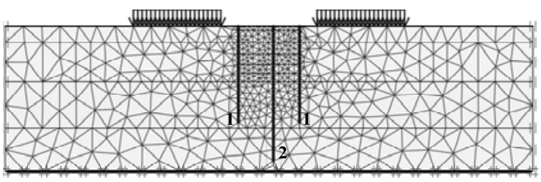

Fig. 6 Meshing for the numerical analysis
1: diaphragm wall; 2: bored pile and steel lattice column

Table 3 Parameters for hardening-soil small model

Parameter	Notation
c'	Effective cohesion
φ'	Effective angle of internal friction
ψ	Angle of dilatancy
E_{50}^{ref}	Reference secant stiffness
$E_{\text{oed}}^{\text{ref}}$	Reference tangent stiffness
$E_{\text{ur}}^{\text{ref}}$	Reference unloading/reloading stiffness
v_{ur}	Poisson's ratio of unloading/reloading stiffness
R_{f}	Failure ratio
m	Power for stress-level dependency of stiffness
G_0^{ref}	Reference shear modulus at very small strains
$\gamma_{0.7}$	Shear strain at which G=0.722G_0*

* G is the shear modulus, and G_0 is the shear modulus at very small strains

Ground improvement is required for the original design. As the unconfined compressive strength (q_u) of the jet grouting column is 1 MPa in this case, E_{50}^{ref}=126q_u=126 MPa (Huang and Gao, 2005), and the compression modulus of the column (E_p) is approximately equal to 63 MPa. The improved soil here is treated as the composite foundation, and the composite modulus can be calculated as

$$E_{\text{sp}} = mE_{\text{p}} + (1-m)E_{\text{s}}, \quad (1)$$

where E_s denotes the compression modulus of the soil, and m is the improvement ratio (0.6 for the original design). The other parameters of the improved soil are valued according to Hou *et al.* (2010).

The basic parameters of the soil are shown in Table 5. The other parameters are determined based on Zhou (2010): (1) $\gamma_{0.7}$=5×10^{-5}; (2) m=0.5 for sandy soil, and 0.8 for clayey soil; (3) v_{ur}=0.2; (4) R_f=0.9; (5) $\psi=\varphi'-30°$.

The relationships $E_{50}^{\text{ref}} = 2E_{\text{oed}}^{\text{ref}}$ for clayey soil, $E_{50}^{\text{ref}} = E_{\text{oed}}^{\text{ref}}$ for sandy soil, and $E_{\text{oed}}^{\text{ref}} = E_{\text{s}}$, $E_{\text{ur}}^{\text{ref}} = 5E_{50}^{\text{ref}}$, $G_0^{\text{ref}} = 2E_{\text{ur}}^{\text{ref}}$ are taken from Zhou (2010).

4.2 Orthogonal experiments

The orthogonal experiment method is an effective and scientific method in the multi-level factorial experimental design. It offers an excellent way to investigate the effect of each factor by selecting a finite number of typical trials from Hou and Wang (1985).

Table 4 Input parameters of structures for initial numerical model

Structure	Material	EA (kN/m)	EI (kN·m^2/m)
Diaphragm wall	C30	1.920×10^7	1.024×10^6
Bored pile	C30	2.010×10^6	8.040×10^4
Steel strut	Q235-B	3.682×10^6	1.620×10^5

Table 5 Soil parameters of the N2 excavation case study

Soil type	Drainage condition	γ (kN/m^3)	k_x (m/d)	k_y (m/d)	E_s (MPa)	c' (kPa)	φ' (°)
Clayey silt	Drained	18.7	0.15	0.08	8.17	5	29
Mucky clay	Undrained	17.3	3.28×10^{-4}	1.64×10^{-4}	2.47	4	24
Very soft silty clay	Undrained	17.4	5.70×10^{-4}	3.80×10^{-4}	3.40	3	26
Silty clay	Undrained	18.9	1.21×10^{-3}	1.21×10^{-3}	5.15	2	32
Improved soil	Undrained	20.0	3.28×10^{-4}	1.64×10^{-4}	38.79	50	40

γ is the natural unit weight, k_x is the horizontal permeability coefficient, and k_y is the vertical permeability coefficient

The essentials of the orthogonal experiment method are factors, levels, and estimated indexes. An orthogonal array L_{16} (4^5) is used in this analysis. Based on the preliminary analysis on the excavation failure, the effectiveness of the following factors is evaluated: strut stiffness (A), surcharge on the Fengqing Road (B), jet grouting improvement ratio of the subsoil under the final cutting surface (C), and cutting surface location of the fourth step excavation (D). Besides, the maximum wall deflection (δ_{hm}), the maximum ground settlement (δ_{vm}), the maximum base heave (δ_{bm}), the maximum wall bending moment (M) and the axial force of the first (F_1) and third (F_3) level struts are selected as estimated indexes. Factors and levels are shown in Table 6, and orthogonal trials are shown in Table 7.

Table 6 Factors and levels of orthogonal experiments

Level	Factor A	Factor B	Factor C	Factor D
1	3.682×10^6 kN	20 kPa	0.6	GL. −13.2 m
2	2.946×10^6 kN	25 kPa	0.4	GL. −14.2 m
3	2.209×10^6 kN	30 kPa	0.2	GL. −15.2 m
4	1.473×10^6 kN	35 kPa	–	GL. −16.2 m

Table 7 Orthogonal trails in the experiments

Trial	Factor A	Factor B	Factor C	Factor D
1	3.682×10^6 kN	20 kPa	0.6	GL. −13.2 m
2	3.682×10^6 kN	25 kPa	0.4	GL. −14.2 m
3	3.682×10^6 kN	30 kPa	0.2	GL. −15.2 m
4	3.682×10^6 kN	35 kPa	–	GL. −16.2 m
5	2.946×10^6 kN	20 kPa	0.4	GL. −15.2 m
6	2.946×10^6 kN	25 kPa	0.6	GL. −16.2 m
7	2.946×10^6 kN	30 kPa	–	GL. −13.2 m
8	2.946×10^6 kN	35 kPa	0.2	GL. −14.2 m
9	2.209×10^6 kN	20 kPa	0.2	GL. −16.2 m
10	2.209×10^6 kN	25 kPa	–	GL. −15.2 m
11	2.209×10^6 kN	30 kPa	0.6	GL. −14.2 m
12	2.209×10^6 kN	35 kPa	0.4	GL. −13.2 m
13	1.473×10^6 kN	20 kPa	–	GL. −14.2 m
14	1.473×10^6 kN	25 kPa	0.2	GL. −13.2 m
15	1.473×10^6 kN	30 kPa	0.4	GL. −16.2 m
16	1.473×10^6 kN	35 kPa	0.6	GL. −15.2 m

The numerical experiments are conducted following the construction procedure (as displayed in Fig. 2), excavation geometry and retaining system as the N2 excavation case but with the factors A–D changing according to Table 7 for each experiment. The results are shown in Table 8, and the negative values in F_1 and F_3 represent a compressive force.

It is shown in Table 8 that there is usually a tensile force in the first level struts. Due to the poor connection of the steel strut and diaphragm wall, when the tensile force becomes large, the strut will break away from the diaphragm wall. The integrity of the strut system is important in excavation construction. The use of concrete struts as the first level struts is a good way to avoid this failure.

Table 8 Calculated results of the orthogonal experiments

Trial	δ_{hm} (mm)	δ_{vm} (mm)	δ_{bm} (mm)	M (kN·m/m)	F_1 (kN)	F_3 (kN)
1	54.5	46.9	89.4	810	3	−2055
2	66.5	58.0	100.0	939	81	−2772
3	92.2	79.1	126.8	1320	198	−3729
4	213.6	175.9	260.4	3020	627	−6813
5	69.9	59.2	105.7	1050	174	−3351
6	88.5	73.1	122.4	1600	270	−4257
7	132.4	111.6	176.5	1960	171	−2706
8	92.2	81.1	124.8	1210	96	−2931
9	102.3	82.5	143.5	1840	378	−4440
10	146.1	122.0	195.0	2230	450	−4377
11	72.6	64.8	103.5	933	57	−2637
12	83.5	74.5	116.3	1090	−3	−2238
13	122.1	101.6	173.3	1910	261	−3276
14	79.5	68.8	115.6	1140	15	−2223
15	118.0	98.2	152.1	1910	279	−4020
16	95.8	84.9	123.7	1210	99	−3084

Fig. 7 shows the relationship between each estimated index and different factors in corresponding levels. The factor with different levels that has the greatest effect on the estimated index is the key factor to this index (Hou and Wang, 1985). As shown in Fig. 7, the effectiveness of the four factors on δ_{hm}, δ_{vm}, δ_{bm}, and M is ranked as C>D>B>A, and D>C>A>B on F_1 and F_3. It is also shown in Fig. 7 that the variation of the six estimated indexes are not significant when the jet grouting improvement ratio of the subsoil under the final cutting surface changes from 0.6 to 0.2; however, when no jet grouting is taken, these estimated indexes increase obviously. From this phenomenon, it is found that ground improvement with jet grouting or some other methods is essential when the final cutting surface is located on the soft soil, but the difference between each improvement ratio for optimizing the excavation behavior is not significant.

Table 9 shows the variance ratio (F) of each factor corresponding to each estimated index.

According to Hou and Wang (1985), when considering the reliability of 95%, the critical variance ratio (F_c) is 9.28. When $F>F_c$, it means the level change of the factor is significant to the index.

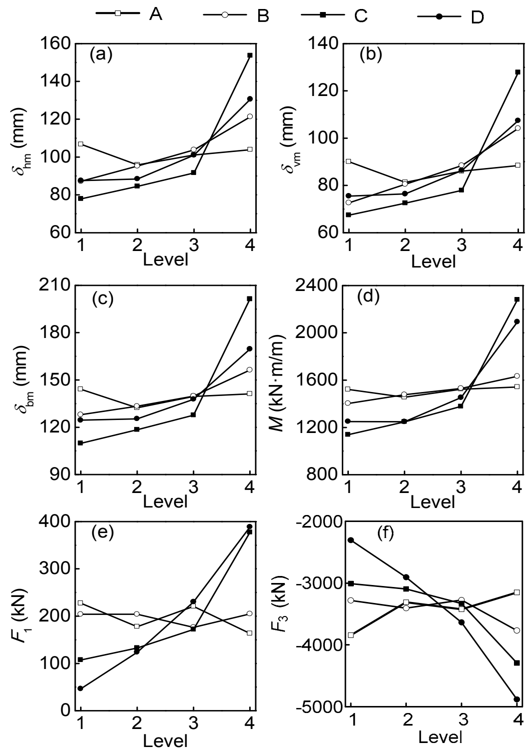

Fig. 7 Factor effects on wall deflection (a), ground settlement (b), base heave (c), wall bending moment (d) and the axial forces of the first (e) and third (f) level struts

Table 9 Variance ratios and the significance

Factor	Estimated index					
	δ_{hm}	δ_{vm}	δ_{bm}	M	F_1	F_3
A	0.8	0.8	0.9	1.5	2.9	1.8
B	7.5	9.5*	5.5	9.4*	0.6	1.1
C	43.0*	41.0*	64.0*	274.5*	44.9*	7.0
D	14.3*	11.6*	16.2*	160.8*	65.2*	24.9*

* The level change of the factor is significant to the index

It is shown in Table 9 that the factors A and B are less significant to the estimated indexes compared to factors C and D. Thus, the strut stiffness is sufficient and the change of the vehicle surcharge on the Fengqing Road has tiny influence on the excavation behavior. Factor C is proved to be the most significant factor to deformations and wall moment, and factor D plays an important role in controlling the axial force of the strut. The interaction between factors C and D on δ_{hm} and F_3 are shown in Fig. 8.

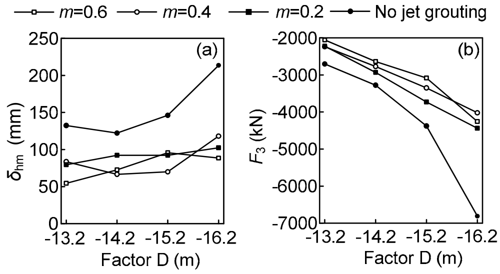

Fig. 8 Interaction between factors C and D on δ_{hm} (a) and F_3 (b)

As shown in Fig. 8, when treating with jet grouting, the cutting surface location of the fourth step excavation changing from GL. −13.2 m to GL. −16.2 m has little influence on δ_{hm}, but provides a linear growth on F_3. However, δ_{hm} and F_3 appear to have non-linear growth according to the cutting surface location of the fourth step excavation when no jet grouting is taken. When the cutting surface location of the fourth level excavation surface is set at GL. −16.2 m, which means the 6.7 m thick soil above the final cutting surface is excavated in one step without setting the fourth level struts, δ_{hm} reaches more than 200 mm, and F_3 is approximately 7000 kN, which far surpass the allowable value (3555 kN) calculated in Section 3.3.

5 Conclusions

After field investigation and preliminary analysis, five reasons of excavation collapse were concluded: (1) using the average value of CQ and not the standard value of CU as the strength parameter; (2) canceling the fourth level struts; (3) poor connection of pipe struts with coupling beams and the diaphragm wall; (4) insufficient monitoring instruments and ignoring the unusual performance of the excavation; and (5) using artesian wells for consolidation instead of jet grouting.

For further investigation, numerical analysis based on the orthogonal experiment method was given with conclusions as follows.

1. When the axial force in the first level struts is tensile force and the struts are not well connected, the strut system may be ineffective. Using concrete struts as the first level struts is a good way to ensure the integrity of the strut system.

2. Compared to the strut stiffness and surcharge on the Fengqing Road, the jet grouting improvement ratio and the cutting surface location of the fourth step excavation have a great effect on the deformation and internal force.

3. Jet grouting can help optimize the excavation behavior, but the difference between each improvement ratio for optimizing the excavation behavior is not significant, so increasing the improvement ratio to reduce the deformation and internal force of the excavation is in vain.

4. When the 6.7 m thick soil above the final cutting surface is excavated in one step without setting the fourth level struts and jet grouting, the deformation and internal force will far surpass the allowable value.

Acknowledgements

The authors would like to thank the Hangzhou government for facilitating access to the data. Guo-rong GU of Shanghai Geotechnical Investigations & Design Institute Co., Ltd, Xing-wang LIU of Zhejiang Institute of Architectural Design, and Guo-xin WANG of China Construction Eighth Engineering Division, China are also appreciated.

References

Benz, T., 2007. Small-Strain Stiffness of Soils and Its Numerical Consequences. PhD Thesis, University of Stuttgart, Stuttgart, Germany.

Ferrari, A.A., 2007. A Remedial Approach to Stabilize a Deep Excavation in Singapore. MS Thesis, Massachusetts Institute of Technology, Boston, USA.

GB 50017-2003. Code for Design of Steel Structures. The National Standards Compilation Group of People's Republic of China, Beijing (in Chinese).

GB 50021-2001. Code for Investigation of Geotechnical Engineering. The National Standards Compilation Group of People's Republic of China, Beijing (in Chinese).

GB 50497-2009. Technical Code for Monitoring of Building Excavation Engineering. The National Standards Compilation Group of People's Republic of China, Beijing (in Chinese).

Hashash, Y.M.A., Song, H., Osouli, A., 2011. Three-dimensional inverse analyses of a deep excavation in Chicago clays. *International Journal for Numerical and Analytical Methods in Geomechanics*, **35**(9):1059-1075. [doi:10.1002/nag.949]

Hou, H.G., Wang, Y.M., 1985. Orthogonal Experiment. Jilin People's Press, Jilin, China, p.1-3 (in Chinese).

Hou, X.Y., Liu, S.Y., Tong, L.Y., 2010. Effect of deep mixing pile reinforce on deformation of deep foundation pit in metro station in passive zone. *Journal of Southeast University (Natural Science Edition)*, **40**(1):180-184 (in Chinese).

Huang, S.M., Gao, D.Z., 2005. Soft Soil and Underground Engineering (Second Edition). China Architecture & Building Press, Beijing, China, p.428-448 (in Chinese).

Jebelli, J., Meguid, M.A., Sedghinejad, M.K., 2010. Excavation failure during micro-tunneling in fine sands: A case study. *Tunnelling and Underground Space Technology*, **25**(SI):811-818. [doi:10.1016/j.tust.2010.05.006]

Kim, S.H., Koo, H.B., Bautista, F.E., Choi, J.Y., 2010. A Case Study on the Geotechnical Characteristics of Collapsed Cut-Slope in Yeosu, Korea. Proceedings of the GeoFlorida Conference (ASCE), West Palm Beach, Florida, USA. ASCE, USA, p.862-871. [doi:10.1061/41095(365)330]

Li, H.W., Wang, G.X., 2010. Causes and suggestion on deep foundation excavation accident in some metro station. *Construction Technology*, **39**(3):56-58 (in Chinese).

Ou, C.Y., 2002. Deep Excavation. Science & Technology Press, Taiwan, p.278-281 (in Chinese).

Ou, C.Y., Hsieh, P.G., Lin, Y.L., 2011. Performance of excavations with cross walls. *Journal of Geotechnical and Geoenvironmental Engineering*, **137**(1):94-104. [doi:10.1061/(ASCE)GT.1943-5606.0000402]

Swanson, P.G., Larson, T.W., 2000. Sewer Tunnel Collapse: A Case History. Proceedings of the Second Forensic Congress (ASCE), San Juan, Puerto Rico. ASCE, USA, p.112-122. [doi:10.1061/40482(280)13]

Wang, Z.H., Zhou, J., 2011. Three-dimensional numerical simulation and earth pressure analysis on double-row piles with consideration of spatial effects. *Journal of Zhejiang University-SCIENCE A (Applied Physics and Engineering)*, **12**(10):758-770. [doi:10.1631/jzus.A1100067]

Zhang, M., Wang, X.H., Yang, G.C., Wang, Y., 2011. Numerical investigation of the convex effect on the behavior of crossing excavations. *Journal of Zhejiang University-SCIENCE A (Applied Physics and Engineering)*, **12**(10):747-757. [doi:10.1631/jzus.A1100028]

Zhou, E.P., 2010. Application of Hardening Soil Model with Small-Strain in Deformation Analysis for Foundation Pit. MS Thesis, Harbin Institute of Technology, Harbin, China (in Chinese).