



Lessons and Mitigation Measures Learned from One Deep Excavation Failure Case

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Abstract. Deep excavations, most of those are temporary retaining structures, have not attracted enough attention from all parties involved in projects. Nowadays there have been numerous failures of deep excavations, and mostly have significant impacts on the safety of the excavation and the surrounding environment, especially in congested urban areas with poor hydrogeology and geology conditions. However, many failure cases remained unpublished and poorly explored due to complex reasons. The reported failure cases were too scarce to provide enough lessons to improve corresponding guidelines. Therefore the database needs more supplement cases. This paper briefly presents the failed case of a deep excavation in soft soil area in downtown city of Shanghai, China, which had an excavation area of about 25,000 m² and two-story basements. According to the field investigation, forensic studies and preliminary analysis, the main reasons for the failure was pointed out. Furthermore the re-checking calculations and mitigation measures were described briefly. The direct cost of repairing and rebuilding the failed excavation was more than 20 million RMB (about 2.8 million dollars), much more expensive than savings. Preserving some redundancy in design and construction is essential. This failed case report is expected to help deep excavations attract more attention and avoid similar failures.

Keywords: Deep excavations · Failures · Case histories · Forensic study · Mitigation measures

1 Introduction

With the rapid development of underground-space such as subway, tunnel, underground garage, underground commercial mall and so on, a number of excavations have emerged, and become much larger and bigger than before due to the scarce available ground resources in congested urban areas. However, deep excavation always has the potential to cause unfavorable effects on nearby ground as well as structures and facilities around it, especially in soft soils (Hu et al. 2003; Wang et al. 2010).

Being a temporary structure in most projects, the investment/cost for excavations and corresponding retaining structures is usually extremely restrained, and in consequence the reserved safety factors are much smaller than that of permanent structures. Although design and construction of excavations are stringently regulated by national and local codes, numbers of excavation failures (such as surrounding subsidence, groundwater

inrushing, huge displacement of retaining walls and even fracture, etc.) still occur from time to time (Gong, *et al.* 2012; Jebelli *et al.* 2010). Therefore, it is necessary to summarize the experiences and lessons of the failures and provide reference to help avoid similar tragedies.

As field performance is a collective reflection of various factors involved in a real excavation, experience from the field performance of previous deep excavations provides a useful guidance in practical design. In recent decades, a series of studies helped us improve the understanding of the performance of deep excavations, and provided lots of experience when estimate the magnitude of movements or to check the rationality of the numerical analysis/field data of deep excavations. (Long 2001; Yoo 2001; Moormann 2004; Wang *et al.* 2010). However in the excavation database, failure reports are relatively limited, especially lacking the failure data in China. In spite of the fact that failures could provide a wealth of information to help improve excavation research, design, construction and management, many failure cases still remain unpublished and poorly explored due to problems of disputes and litigation. Unfortunately, the scarce reported failures failed to attract sufficient attentions of all parties involved in projects, failed excavations are still happening.

This paper briefly presents the failed case of a deep excavation in soft soil area in downtown city of Shanghai, China. According to the field investigation, forensic studies and preliminary analysis, the main reasons for the failure was pointed out. Furthermore the re-checking calculations and mitigation measures were described briefly. This failed case study shows how relevant factors “eat up” the safety factor and led to failure, so as to extend failure cases database and help deep excavations attract more attention and avoid similar failure cases.

2 Project Overview

The Failed case of the deep excavation was located in soft soil area in downtown city of Shanghai, China. The project comprised a two-story basements, and the excavation area and perimeter of the project were about 25000 m² and 480 m, respectively. The excavation depth of the site was mostly 8.7 m, where the site elevation of ground surface was leveled to not higher than 3.40 m in Wusong elevation (the elevation above Wusong Sea level). Unless otherwise specified, the elevation below is Wusong elevation.

Figure 1 shows the layout and surrounding environment of the failed project. The environment was not quite complex, surrounded by a small river (about 6 m–10 m in width and 0.4 m–0.9 m in depth) to the north, a 28 m-wide municipal road to the east, a public green land to the west and internal construction site (the construction work of the south 1-story basement was finished, up to ground level) to the south. The main protection objects around the project were the municipal road and underground pipelines in the East.

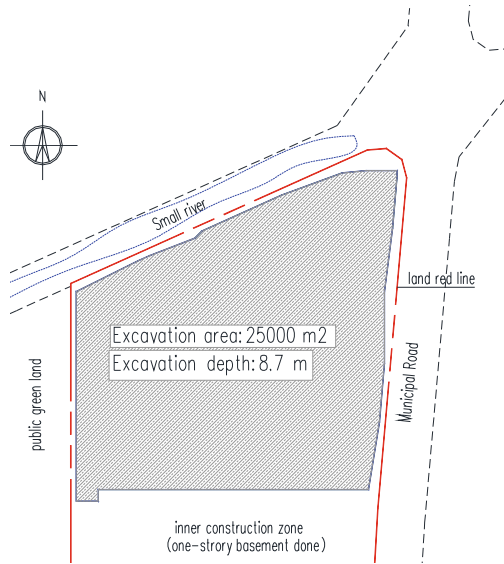


Fig. 1. Sketch of the surroundings around excavations

2.1 Geology and Ground Condition

The project site was situated at Yangtze River Delta alluvial plain. According to the geotechnical investigation report and the division work of Gao et al. (1986) the subsurface soils at the construction site were mainly thick soft soils comprising Quaternary alluvial and marine deposits. As shown in Table 1, from ground surface to a depth of about 30 m, the soil profile could be divided into 4 layers, among which Layer ② and Layer ⑤ could be subdivided into 2 sub-horizontal layers, respectively. The top layer is loose fill mixed with lots of construction waste, having relatively poor engineering mechanical properties. The Layer ②1 and Layer ②3 are over-consolidated and commonly referred to as stiff surface crust. In this site, Layer ③ was lacked. Layer ④, where the excavation was located on, was soft clay layer of thickness about 8.5 m–11.0 m. The soft clay has relatively higher natural water content, void ratio and compressibility but lower shear strength, which is one of the main unfavorable factors for deformation control for excavations in soft areas. Underlying Layer ④ was Layer ⑤1-1 and Layer ⑤1-2. The fifth layer was mainly gray silty clay with medium plastic and medium to high compressibility. The physical and mechanical properties of Layer ⑤ were much better than that of Layer ④.

2.2 Original Excavation Design Scheme

In the original excavation design, the excavation was constructed using the bottom-up construction method, bored piles and waterproof curtain were designed to be the retaining system. Meanwhile, the excavation was supported by one level of reinforced concrete strut. The section size of main RC struts was 1000 mm in width and 800 mm in height.

Table 1. Physico-mechanical parameters of the soils

soil sequence and description	h (m)	γ (kN/m ³)	φ (°)	C (kPa)	Kv (cm/s)	Kh (cm/s)	k* (cm/s)	q _c (MPa)
①Miscellaneous Fill	1.8	<u>18.0</u>	<u>10.0</u>	<u>10.0</u>	/	/	5.0E-4	0.65
②1 Silty clay	1.8	18.4	19.5	21.0	2.73E-7	8.63E-7	9.0E-6	0.74
②3 Sandy silt	3.3	18.6	31.0	5.0	6.94E-5	1.39E-4	4.1E-4	1.81
④ Soft clay	10.4	16.8	11.5	11.0	8.03E-8	8.95E-8	6.5E-7	0.52
⑤1-1 Silty clay	4.7	18.1	18.5	15.0	3.10E-7	1.94E-6	6.0E-6	0.71
⑤1-2 Silty clay	4.3	18.2	19.0	16.0	4.83E-7	2.10E-6	8.5E-6	0.98

Note: Underlined data mean empirical values, while the others mean standard values. h = thickness of typical section; γ = unit weight; φ = angle of internal friction obtained from direct shear test; c = cohesion obtained from direct shear test; kv = vertical coefficient of permeability; kh = horizontal coefficient of permeability; k* = recommended coefficient of permeability. q_c = measured cone penetration test end resistance.

And the horizontal spacing of the struts was approximately 10 m. Figure 2 presents a typical section of retaining structures.

The bored piles was designed embedding into the fifth layer to help retaining soil, with 900 mm in diameter and 18.3 m in length. Moreover the bored piles were filled with concrete strength grade of 30 (the underwater casting need to be improved by one level). Herein the concrete strength ($f_{cu,k}$) refers to the compressive strength of molded 150 mm cubes after 28 days' curing should be more than 30 MPa, according to Chinese code MCPRC (GB50010-2010, 2015). This concrete grade C30 is somewhat equivalent to $f_{ck,cub} = C25/30$ according to European Standard CEN (BS EN 1992-1-1, 2000).

The groundwater level in Shanghai is generally located at 0.5-1.0 m below the ground surface and the most unfavorable groundwater level (i.e. 0.5 m) was used in design. Since the confined aquifer is very deep in the subsoil compared with the excavation depth, the confined water pressure is not a risk in this project. Consequently two rows of deep cement-soil mixing wall mixed by dual-anger mixer (Ø700 mm @500 mm) were designed as waterproof curtain, to cutoff the phreatic water during excavation construction. The waterproof curtain was designed 15.6m in length and 13% in cement slurry by mass.

The lateral supporting RC strut was designed 2.1 m below ground surface, 6.6 m above the bottom. The horizontal spacing of the temporary struts was mostly 6 m-10 m, with concrete grade of C30.

Since the top 15 m soil deposits in this site are quite soft, the soils under the final cutting surface need some ground improvement to increase stiffness and help control deformations, especially in the middle of each side and those having deep water-collecting wells inside. The weak soils were designed mixing with cement mixed by dual-anger mixer (Ø700 mm @500 mm, 13% in cement slurry by mass).



Fig. 3. Picture of the failed deep excavation

In order to avoid further expansion of the settlement and damage, the emergency plan was adopted in-time. The northeastern and northwestern part was temporary backfilled until the deformation was stable. Specially, the areas closed to retaining piles were backfilled with gravel soil (adding some cement slurry as soil improvement) to help control deformation. The distribution range of backfill treatment could be seen in Fig. 4.

3.2 Field Investigation and Testing

The failed excavation had drawn lots of attentions from different parties involved in this excavation construction, not limited to the construction investor, designer, monitor, and constructor and so on. With a series of investigation and testing conducted by three different independent third parties (ranging from non-professional operations, soil parameters, design reviews, retaining system tests, monitoring data reviews), the reasons of the failed excavation were carefully studied and analyzed.

3.3 Forensic Studies and Discussions for the Excavation Failure

Here the “forensic study” is similar to reverse engineering, namely the end result is known and one then looks for plausible reasons for the failure. After preliminary analyses, the contributing reasons of the failed excavation were conclude as follows.

1) Insufficient length of retaining bored piles

Core drilling tests were carried out in 8 retaining bored piles, and the investigated length were compared with original design length, as shown in Table 2. The first 3 piles, mostly

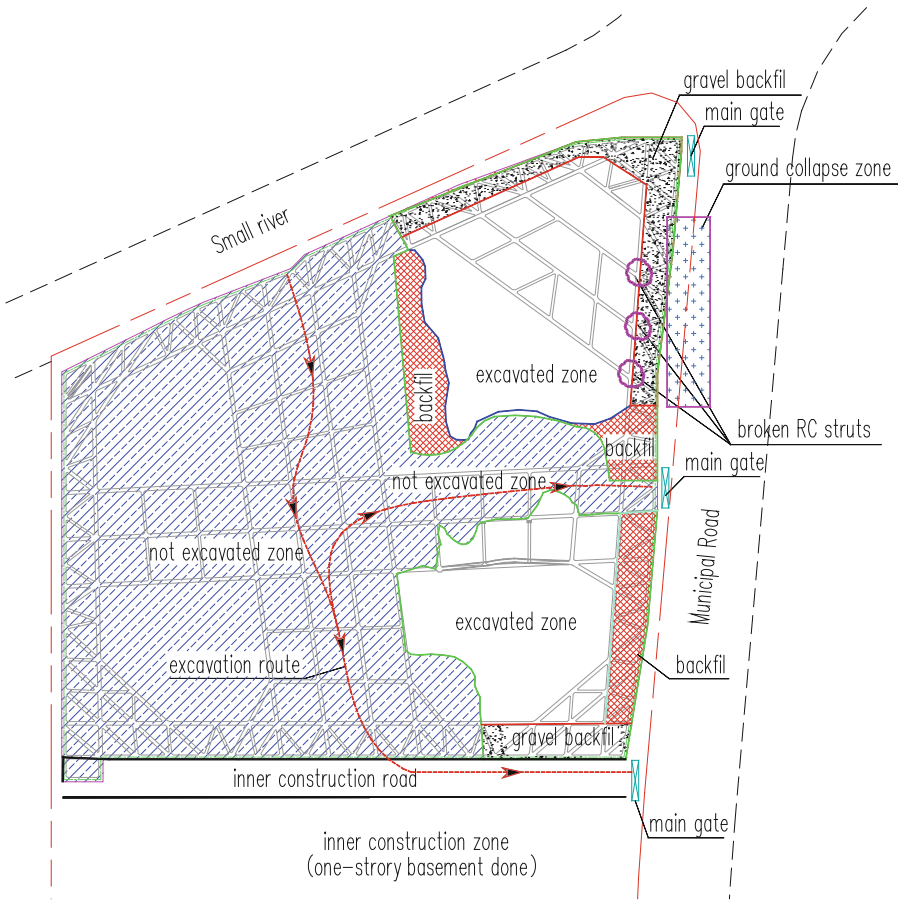


Fig. 4. Schematic diagram of the failed excavation (broken RC struts was marked by circles) and temporary backfill treatment

located in eastern part, were tested by Company A and the 4th-8th pile (located in north, west and south) were tested by Company B. Note that the A-1# and A-2# piles were located in the collapse area where the displacement of retaining piles were quite large, the coring operation was incomplete and discontinuous. Hence, the length of this two piles were estimated from the length of steel reinforcement cage.

Table 2 indicates that the length of tested retaining bored piles were all insufficient, 0.46 m–1.60 m shorter than original design. According to the geotechnical investigation report, the elevation of bottom of Layer ④ was -12.05 m--14.19 m. The shortest tested piles probably embedded only 0.31 m underneath Layer ⑤1-1, where the insufficient embedded depth was too small to provide anchoring force. Large deformation thus generated quickly from the bottom and finally led to “kicking” failure of the whole excavation.

Table 2. Comparison of tested and designed length for retaining bored piles

Test No	Elevation of pile-bottom (m)		Length of retaining pile (m)		Difference between tested and designed length (m)
	Designed	Tested	Designed	Tested	
A-1#	-16.1	<u>-14.9</u>	18.3	<u>17.1</u>	-1.2
A-2#	-16.1	<u>-14.6</u>	18.3	<u>16.8</u>	-1.5
A-3#	-16.1	-14.5	18.3	16.7	-1.6
B-4#	-16.1	-14.59	18.3	16.79	-1.51
B-5#	-16.1	-15.21	18.3	17.41	-0.89
B-6#	-16.1	-15.13	18.3	17.33	-0.97
B-7#	-16.1	-15.64	18.3	17.84	-0.46
B-8#	-15.5	-14.92	16.4	15.82	-0.58

Note: (1) The top elevation of retaining piles was + 2.20;

(2) Underlined data were inferred from the length of steel reinforcement cage;

(3) Difference between tested and designed pile length=length of tested pile – length of designed pile.

2) Inadequate strength of retaining bored piles

Table 3 lists the tested concrete compressive strength ($f_{cu,k}$) of the core sample from core drilling tests conducted by two different independent third party. The cylindrical strength (f_{ck}) is about 24% lower than cubic strength ($f_{cu,k}$) in Shanghai according to Chinese national code MCPRC (GB50010–2010, 2015). Therefore the tested compressive strength ($f_{cu,k}$) listed in Table 3 was calculated by tested cylindrical strength (f_{ck}) multiplied 0.76, i.e., $f_{cu,k} = 0.76f_{ck}$.

Note that the tested compressive strength values of the 1st and 2nd piles (locations near the collapse zone) were much lower than others, it is possible that the piles got damaged more or less, thus these two results were not accurate to some extent and recommended not to be included in the statistics.

As shown in Table 3, about 75% of the tested retaining bored piles had inadequate compressive strength, and the difference with original design C30 was about 1.3 MPa–6.3 MPa. Inadequate strength of piles will lead to lower E_c and further lower bearing capacity against horizontal earth pressure, which is another factor contributing to the failure.

3) Poor quality of reinforced concrete struts

The original design required the RC struts to be straight so as to decrease the external forces. However, the in-situ investigation found out that at least 21% of the struts in excavated zone had deflection like fish belly beyond the requirements of design and standards. In the northeast side of the excavation where struts had visible ruptures or even broken, a series of investigations detecting the reinforcement and concrete qualities were carried out in 15 different RC struts by Company C. The investigation indicated that the reinforcement arrangement and quantity of the tested struts basically meet the original

Table 3. Comparison of tested and designed strength ($f_{cu,k}$) for retaining bored piles

Test No	Concrete compressive strength $f_{cu,k}$ of piles (MPa)		Difference between tested and designed concrete compressive strength of piles (MPa)
	Designed	Tested	
A-1#	30.0	<u>16.68</u>	-13.32
A-2#	30.0	<u>15.89</u>	-14.11
A-3#	30.0	23.70	-6.30
B-4#	30.0	23.90	-6.10
B-5#	30.0	28.70	-1.30
B-6#	30.0	27.80	-2.20
B-7#	30.0	32.00	2.00
B-8#	30.0	31.00	1.00

Note: (1) Underlined data were probably disturbed by collapse, not included in statistics and forensic studies;

(2) Difference between tested and designed concrete compressive strength of piles = $f_{cu,k}$ of tested pile - $f_{cu,k}$ of designed pile.

design requirements. However, there were 5 RC struts (over 30% of the tested sample) having insufficient concrete strength, the difference with original design (concrete grade C30) was about 0.3 MPa–7.3 MPa. For the strut having $f_{cu,k}$ of 22.7 MPa, the maximum axial compressive force it could bear was only 75.7% of the original design.

In addition, quite a lot of RC struts (over 20%) had visible missing edges and corners. Smaller section area of RC struts will inevitably lead to less bearing capacity, even fail to meet the design requirements. Therefore, when there is a dynamic/impact load or the retaining bored piles had larger deformation, RC struts having low straightness would have larger eccentric loads and bending moment. Compounding the problems are the defects of struts (missing edges and corners, eccentric error, inadequate strength). Gradually the struts will be less effective, leading to a decrease in the stability of the retaining system, and even developed to failure.

4) Invalid monitoring data

Overall, the environment of this project is not complex and even kind of loose, only the municipal road and adjacent pipelines needed more protection. Whereas, good environment was directly related to the failure. Through recalculation and back analysis on data got from in-situ investigations, the maximum deflection of the retaining wall was over 50 mm, the central posts were uplifted over 30 mm, and the deformation rate exceeded 3 mm/d, which already surpassed the allowable value according to original design and the Shanghai code (SUCTC, DG/TJ08–61–2018, 2018). However, in the monitoring report, data were tampered to be normal. It was said that some measuring points were not actually monitored for several days, which means the monitoring data were somewhat invalid, and did not truly reflect the real performance of the excavation. Lack of vigilance and invalid monitoring data resulted in ignoring the unusual performance of

the excavation, the rate and amount of deformation increased more and more, finally the accident occurred.

4 Re-checking Calculation and Mitigation Design

Before further measures were taken to repairing or even rebuilding the excavations, a series of re-checking calculations were conducted with the program, FRWS 8.2 (2018). For this project, the limit equilibrium method was used for overall stability of retaining system calculation, moreover, the horizontal coefficient of subgrade based on elastic foundation beam was chose to predict displacement based on the Chinese code and Shanghai local standard (MCPRC, JGJ120–2012,2012 and SUCTC, DG/TJ08–61-2018, 2018).

For this excavation, the required factors of stability are as follows:

- $F_{so} = 1.3$, $F_{sb} = 1.6$, Safety Level II, according to Chinese standard (MCPRC, JGJ120–2012,2012);
- $F_{so} = 1.25$, $F_{sb} = 1.9$, Safety Level II, according to Shanghai local standard (SUCTC, DG/TJ08–61-2018, 2018).

Typical results of re-checking calculations were shown in Table 4 and 5.

Table 4. Parameters for different models in re-checking calculation

Model No	Length of retaining bored pile (L/m)	Strength of retaining bored pile ($f_{cu,k}$ /Mpa)	Stiffness of reinforced concrete struts (EA/kN)	Surcharge on the municipal road (P/kPa)	Remarks
M1	18.3 m	30	2.16×10^7	20	Original design
M2	16.7 m	20	1.12×10^7	30	Worst condition (collapsed zone)
M3	16.7m	20	2.16×10^7	10	Mitigation measure A
M4	20.3 m	30	1.12×10^7	10	Mitigation measure B

Table 5. Calculated results of different re-checking models

Model No	Maximum wall horizontal displacement (δ_{hm}/mm)	Maximum ground settlement (δ_{vm}/mm)	Maximum wall bending moment (M/kN.m/m)	Axial force of the struts (F/kN/m)	Factor of overall stability (F_{so})	Factor of stability against basal heave (F_{sb})
M1	31.9	39.5	1259.7	351.5	1.93	1.59
M2	38.8	51.2	1239.1	387.7	1.57	1.27
M3	23	31.1	745.9	183.5 (1 st)	1.75	1.39
				<u>373.1 (2nd)</u>		
M4	21.1	25.7	794.3	198 (1 st)	2.45	1.96
				<u>356.8 (2nd)</u>		

Note: (1) Underlined data are the axial force of the second struts (added);

(2) In M2~M4, the parameters of miscellaneous fill (mainly φ and c) were discounted to some extent, taking the effect of collapse into consideration.

(3) F_{so} = ratio of anti-sliding moment to sliding moment based on Swedish slice method; F_{sb} = ratio of the resistant shear force to the driving shear force along a certain slip surface, based on Prandtl theory.

After several trials, the mitigation measures for this failed excavations (mainly the typical section as shown in Fig. 2) could be divided into categories: Mitigation measure A and B.

- **Mitigation measure A** (for zones bored piles were still effective):

1) repair the visible cracks, missing edges and corners of the RC struts; cut off and rebuild the broken struts; 2) add another level of RC struts; 3) restrict the surcharge on surrounding road to no more than 10 kPa; 4) add trestles, platforms and corresponding central posts to help organize construction works inside the excavation.

- **Mitigation measure B** (for zones where bored piles were broken or not effective):

Besides the mitigation measures in Mitigation measure A, the following measures are still needed (refer to Fig. 5).

1) clear underground obstacles and increase ground stiffness with deep cement-soil mixing wall; 2) install new bored piles inside the deep cement-soil mixing wall; 3) improve soils between the original failed retaining pile and new retaining piles, and those inside excavation (under the bottom).

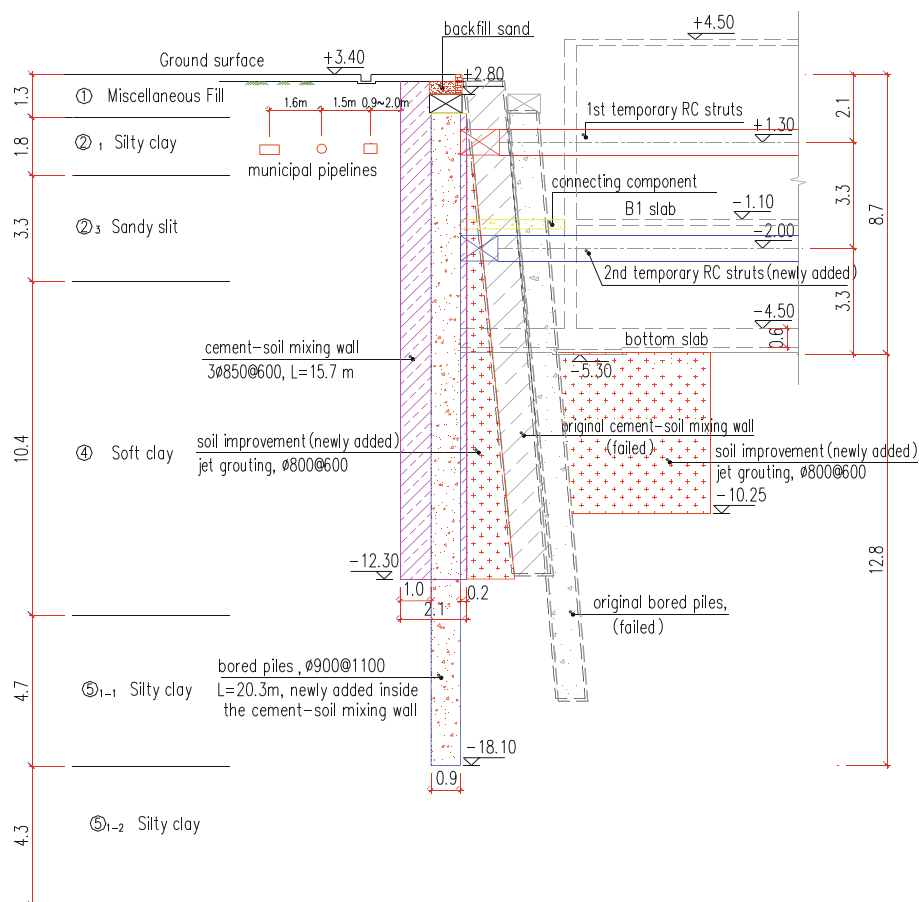


Fig. 5. Typical mitigation measures of retaining structures at the failed zone (Mitigation measure B)

5 Concluding Remarks

This paper presents a failed case of a deep excavation in soft soil area in downtown city of Shanghai, China. After field investigation, forensic studies and preliminary analysis, four main reasons of excavation failure were concluded: (1) insufficient length of retaining bored piles led to poor anchoring and kicking failure; (2) inadequate strength of retaining bored piles; (3) poor quality of reinforced concrete struts in straightness, strength and section area; (4) invalid monitoring data and ignoring the unusual performance of the excavation.

Further consulting and mitigation design were carried out with a series of re-checking calculations based on limit equilibrium approach and elastic foundation beam method. Finally, the direct cost of repairing and rebuilding the retaining system was more than 20 million RMB (about 2.8 million dollars) in total, more expensive than savings of jerry works and original design optimization. Another important lessons could be learn

from this case are summarized as follows. Although retaining systems for excavation are temporary, the optimization design, jerry-build construction and other relevant factors together “eat up” the safety factor, good design and construction should still preserve rational or relevant redundancy. The cost of this redundancy is far less than the cost of failure and increased repair or even rebuild.

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References

- CEN (European Committee for Standardization). Concrete-Part1: Specification, performance, production and conformity. BS EN206-1 (European Standard English version). BSI (2000) FRWS/2018. Standard user's manual-version 8.2[in Chinese].QiMSTAR (Tongji Qimingxing, Inc.), Shanghai, China (2018)
- Gao, D.Z., Wei, D.D., Hu, Z.X.: Geotechnical properties of Shanghai soils and engineering applications. ASTM Special Technical Publication, pp. 161–177 (1986)
- Gong, X.-N., Zhang, X.-C.: Excavation collapse of Hangzhou subway station in soft clay and numerical investigation based on orthogonal experiment method. *13*(10), 760–767 (2012)
- Hu, Z.F., Yue, Z.Q., Zhou, J., et al.: Design and construction of a deep excavation in soft soils adjacent to the Shanghai metro tunnels. *Can. Geotech. J./Rev. Can. Geotech.* **40**(5), 933–948 (2003)
- Jebelli, J., Meguid, M.A., Sedghinejad, M.K.: Excavation failure during micro-tunneling in fine sands: a case study. *Tunnel. Undergr. Space Technol.* **25**(SI), 811–818 (2010)
- MCPRC (Ministry of Construction of the People's Republic of China). Code for design of concrete structures GB 50010-2010 (revised edition, 2015). China Architecture and Building Press, Beijing (2015). [in Chinese]
- MCPRC (Ministry of Construction of the People's Republic of China). Technical specification for retaining and protection of building foundation excavations. JGJ120-2012. China Architecture and Building Press, Beijing (2012). [in Chinese]
- Long, M.: Database for retaining wall and ground movements due to deep excavations. **127**(3), 203–224 (2001)
- Moormann, C.: Analysis of wall and ground movements due to deep excavations in soft soil based on a new worldwide database. *Soils Found.* **44**(1), 87–98 (2004)
- SUCTC (Shanghai Urban-rural Construction and Transportation Commission). Technical code for excavation engineering. DG/TJ08-61-2018. Tongji University Press, Shanghai (2010). [in Chinese]
- Wang, J.H., Xu, Z.H., Wang, W.D.: Wall and ground movements due to deep excavations in Shanghai soft soils. *J. Geotech. Geo-environ. Eng.* **136**(7), 985–994 (2010)
- Yoo, C.: Behavior of braced and anchored walls in soils overlying rock. *J. Geotech. Geoenviron. Eng.* **127**(3), 225–233 (2001)