Optimum Design of Excavation Deepening for Deep Foundation Pits in Complex Sites

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Abstract: Many historical buildings in the old urban area of Zhongshan Road in Guangzhou have high commercial value and requirements for large underground space. However, their safe development is restricted by many factors, such as limited surrounding space, dense underground pipe networks, poor engineering properties of shallow soft soil, and excessive amounts of groundwater. Hence, this work proposes an optimized approach in which the current supporting structure of foundation pits is used for basement expansion in complex environments.

A case study of a basement expansion in Honghui Plaza II, Guangzhou, P.R. China, was exemplified in this work. The original foundation pit needed to be deepened to expand the current three-story basement into a four-story one. Although the original diaphragm wall was excavated nearly 10 years ago, it was proved to be suitable for further usage after quality inspection. Thus, together with the partial roof of the basement and steel pipe columns, the diaphragm wall was utilized as a part of the support system. Several measures were taken to avoid deformation risk resulting from short embedded sections. They included structure optimization of the current support system and effective control of the earthwork excavation process. The deformation of the supporting structure and adjacent subway tunnel was simulated using the finite element method during excavation. Results showed that the simulation was basically consistent with the field monitoring value. The excavation deepening of the foundation pit in this case study was proved in practice to have little impact on the surrounding environment. Therefore, the results of this work can serve as a reference for similar urban renewal projects in the future.

Key words: Deep foundation pit Excavation deepening Optimum design Support system
1. Introduction

With the development of old city reconstruction and quality improvement, the demand for basement expansion with minimal impact on surrounding areas\(^1\)-\(^3\), especially in old urban districts or commercial centers with complex environments, has increased considerably. To meet this demand, scholars have suggested to utilize soil-retaining and water-stopping systems of current foundation pits for basement expansion to revive and parlay stock resources and greatly reduce construction risks\(^4\).

On the basis of the story-adding and deepening reconstruction of the basement of Honghui Plaza Phase II, a systematic design method involving the use of the original deep foundation pit was adopted in the current study\(^5\),\(^6\). The method included the analysis and selection of geotechnical parameters, quality inspection of the diaphragm wall, structure optimization of the support system\(^7\)-\(^9\), and three-dimensional numerical simulation and monitoring\(^10\),\(^11\). The results can serve as a reference for similar projects in the future.

2. Engineering characteristics and geological environment

2.1 Project profile

Honghui Plaza Phase II is located in the south of Zhongshan 4th Road, Guangzhou, Guangdong. According to the original design, it has a three-story basement with a story height of 4.20–4.80 m. The foundation pit is rectangular with an excavation depth of 12.65 m and perimeter of 250 m. The surrounding environment of the foundation pit is complex (Figure 1). The east and north sides of the foundation pit are close to the municipal road, where high-pressure water pipes and gas pipes are buried approximately 1.50–3.70 m apart. The outer line of the shield tunnel structure of Metro Line 1 is located 7.40 m away at the nearest point. Several old two- and three-story houses stand on the south side, and the nearest one is 4.10 m and is characterized as having a wooden pile foundation and brick concrete structure. The west side is connected to the four-story basement of Honghui Plaza Phase I (sharing a continuous wall). The safety of the foundation pit is at Level I.

![Fig 1. General layout of foundation pit](image-url)
and slab of the lower ground floor was not constructed. The project was later suspended because of fund shortage.

Fig 2. Section of original top-down support

In 2017, the planning scheme of the project was adjusted, and the basement was set to be expanded to four floors. Limited by the embedded depth of the original continuous wall (4.5 m), the basement structure type was changed (cap-type beamless floor), the height of the standard floor of the basement was reduced (3.30–4.35 m), the bottom of the basement was increased (about 0.30 m), and a raft foundation was deployed (plate thickness of 0.90–1.30 m). The excavation depth of the foundation pit was optimized to 14.60–15.50 m and was thus 1.65–2.55 m deeper than the original foundation pit. The embedded depth of the diaphragm wall was 2.90 m.

2.2 Geotechnical conditions

The geological conditions of the site are poor, and the excavation depth of the foundation pit mainly includes miscellaneous fill, flowing plastic silt and muddy soil, plastic to hard plastic residual silty clay (mainly plastic), and locally completely weathered and strongly weathered rocks. The silt has high moisture content (61%–72%), large thickness, and poor engineering properties. The fully weathered and strongly weathered rocks easily soften and disintegrate upon contact with water. The depth of the perched water level stored in the miscellaneous fill soil is 1.00–1.20 m. In situ cross-plate shear tests were carried out on silt, and the parameters of shear strength, residual strength, and sensitivity were 28–43 kPa, 18–25 kPa, and 1.7–2.4, respectively. The results of the hit number after rod length modification fell in the range of 21–34 in the heavy dynamic penetration tests conducted on fully weathered and strongly weathered rocks. We performed in situ cross-plate shear tests on silt (shear strength of 28–43 kPa, residual strength of 18–25 kPa, and sensitivity of 1.7–2.4) and heavy-duty power touch tests on fully weathered and strongly weathered rocks (21–34 blows after rod length correction). The geotechnical design parameters were determined, and the results are shown in Table 1 according to the survey data and regional experience.
Table 1 Proposed values of main rock and soil design parameters

| Rock-soil layer          | state              | unit weight of soil $\gamma$(kN/m$^3$) | Thickness $(m)$ | cohesion C(kPa) | Internal friction angle $\phi$(°) | Blow count of SPT $N$ (count) | $m$ value (MN/m$^4$) |
|--------------------------|--------------------|----------------------------------------|----------------|-----------------|----------------------------------|-------------------------------|---------------------|
| 1 miscellaneous fill     | loose-slightly     | 17.3                                   | 2.20~5.60      | 8~10            | 22~25                            | 4~12                          | 5~8                 |
| 2 muck, mucky soil       | Flowing plastic    | 15.0                                   | 3.20~7.80      | 8~10            | 6~7                              | 1.5~2.0                       |                     |
| 3-1 silty clay, clay     | plastic            | 19.0                                   | 0.80~4.10      | 18~20           | 16~18                            | 6~10                          | 6~8                 |
| 3-2 silty clay, clay     | Hard plastic       | 19.5                                   | 0.90~3.90      | 22~25           | 20~22                            | 12~21                         | 10~12               |
| 3-3 silty clay, silt     | hard               | 20.0                                   | 0.60~4.70      | 30~32           | 24~26                            | 22~36                         | 15~18               |
| 4-C siltstone, gristone  | Completely weathered | 21.0                               | 1.20~6.60      | 45~50           | 25~27                            | 35~46                         | 2~25                |
| 4-I siltstone, gristone  | Intense weathered  | 22.0                                   | 4.00~27.80     | 55~65           | 26~28                            | 56~72                         | 30~35               |

2.3 Main technical difficulties
The loose–soft soil onsite showed poor engineering properties while the deformation of the subway tunnel adjacent to the site should be less than 10 mm. The diaphragm wall in the excavated area had been exposed for more than 10 years with no protection, thereby leading to the possible damage and corrosion of the wall. Therefore, a structural quality test should be performed. In addition, the reinforcement rate of the diaphragm wall was low. The reinforcement models of the excavation and soil side were HRB335ф25@100 and HRB335ф25@150, respectively, and the embedded depth was only 2.9 m. Such low reinforcement rate and embedded depth may cause skirt instability.

After comparative research, the five-story frame structure above the ground was proposed to be dismantled. Other recommendations included adopting the original diaphragm wall and internal support and then applying the open-cut construction method instead of the original reverse way. Through this method, the cost can be reduced by 30%, and the project duration and risk can be controlled.

3. Quality inspection and utilization of original supporting structure
During the excavation of the original foundation pit and the construction of Honghui Plaza I, deformation monitoring showed that the maximum horizontal displacement of the diaphragm wall was only 8 mm. Abnormalities such as water leakage or wall cracking were not observed, and the subway tunnel subsidence and lateral displacement were ±2–3 mm.

The scan of the steel bar spacing and distribution of 18 diaphragm walls with a PS200S steel detector and the excavation of the protective layer of the diaphragm wall steel bars revealed that the surface concrete was slightly carbonized and that the steel bars showed no corrosion. The core drilling method was used to conduct core pulling tests on nine diaphragm walls. The results showed that the coarse and fine concrete aggregates in a long strip shape were evenly distributed and well cemented. The integrity of the wall body belonged to Class I. The minimum compressive strength of
the core sample was 38.6 MPa, reaching the original design strength value of C35. The thickness of
the sediments at the bottom of the wall ranged from 25 mm to 48 mm, and the length of the wall and
the thickness of the bearing layer at the bottom of the wall were in line with the original design
requirements. After a comprehensive analysis, the diaphragm wall was certified to be of good quality.

Given the surrounding environment and the current state of excavation, the two-span beam slab
(16 m wide) on the first floor of the north side of the building was kept as a platform to help reduce
the impact of the lateral deformation of the diaphragm wall on high-pressure pipelines and subway
tunnels. After the underground construction of the first-floor slab, the beam slab and the first support
were removed simultaneously, and the original five steel pipe columns were used as supporting
columns. The remaining frames were removed by cutting.

4. Optimized layout of support system

According to the shape of the foundation pit, the angle support and double-row opposite support
layouts were adopted for the interior support (Figure 3). The northeast corner was equipped with
beams, slabs, and trusses to enhance the overall stiffness. The distribution range of the corner braces
at the northeast and southeast corners was enlarged to some extent to reduce the transfer of the earth
pressure from east to west. Meanwhile, plate braces were deployed at the west end of the double-row
pair braces to convert the point-type concentrated force into uniform linear load and reduce the
impact of the concentrated force on the west diaphragm wall. A large space was reserved at the north
and south ends of the foundation pit to facilitate earth excavation and the construction of the south
tower.

Fig 3. Horizon layout of foundation pit

Taking reinforcement ratio, deformation control, construction convenience and economics of the
diaphragm wall as constraints, this study checked the strength, deformation, and stability of the
diaphragm wall by using the Lizheng deep foundation pit software. The results of the calculation
suggested the deployment of two internal supports at −2.70 and −9.70 m (Figures 4 and 5,
respectively). In addition, the first-floor beams and slabs (16 m width) on the north side should be
temporarily maintained to limit the deformation of the diaphragm wall in the deep excavation area (−7.3 m).

The bending moment, support axial force, displacement, shear force, and earth pressure of the diaphragm wall in the north support section (Figure 4) were calculated, as shown in Figure 6. The maximum deformation was 12.97 mm, which was 7–8 m below the ground surface. The buried depth of the subway tunnel top was 9.4 m while that of the horizontal center line was 12.4 m. The lateral deformation of the diaphragm wall was calculated to be 8–9 mm, which was less than the deformation threshold of 10 mm. The lateral deformation of the east and south diaphragm walls was calculated to be 19.4–26.8 mm. The strength and deformation of the west diaphragm wall were
checked with the load that was distributed evenly on the multislip continuous beam, which meets the requirements of strength, stability and viability.

The seventh working condition ——Excavation

![Calculation results of excavation of north support section](image)

Fig 6. Calculation results of excavation of north support section

According to the two-dimensional calculation, the bottom displacement of the diaphragm wall was 2–3 mm, which was less than the upper limit of the soil shear failure deformation by 4–6 mm. Given the spatial effect of the adjacent load-bearing walls situated 12 m apart and the structure with two and three fulcrums in the north, the instability risk of the diaphragm wall foot was found to be quite limited. To effectively control the deformation of the wall bottom during the earthwork excavation, this study maintained the 1 m thickness of the reserved soil layer at the bottom of the pit. The remaining soil was removed, and the bottom was sealed in three sections from the south and north to the middle. The basement floor was also constructed in sections.

For a convenient and quick construction, the supporting column was made of seamless steel pipe with a diameter of 426 mm and wall thickness of 12 mm. The hole was drilled using a 300-type drilling machine with a diameter of 500 mm. The bottom of the hole was grouted and filled with pisolite. The water–cement ratio of the cement slurry was 0.4–0.6, and the strength grade was C20.

5. Numerical simulation and result analysis

5.1 Calculation model

An integrated three-dimensional model combining the deep foundation pit supporting structure, subway tunnel, and rock and soil was established in Midas GTS NX commercial software. The foundation pit soil was simulated with hexahedral solid elements and a Mohr–Coulomb model, underground continuous wall and shield tunnel with beam elements and elastic models, and internal
support with beam elements.

Normal displacement constraints were imposed around the deep foundation pit model, and fixed constraints were imposed on the bottom. The surface was a free boundary. A fixed boundary water head was applied around the deep foundation pit model. In the process of simulating the internal force and deformation of the supporting structure and subway tunnel during the earth excavation of the deep foundation pit, the soil in the foundation pit was first dewatered and then excavated. Thereafter, internal support was applied. Taking three times the depth of the deep foundation pit as the extended range of the soil body in the model, this study shows the overall model and grid division of the deep foundation pit in Figure 7.

![Fig 7. 3D calculation model and grid division of deep foundation pit](image)

5.2 Calculation results and analysis

The numerical simulation results of each working condition showed that the maximum lateral deformation on the east and north sides of the underground diaphragm wall were 20.87 and 12.83 mm, respectively. The deformation curve was parabolic, which was basically similar to the calculation results in the Lizheng deep foundation pit software.

The maximum horizontal deformation of the tunnel structure of Metro Line 1 was 2.3 mm, the maximum vertical deformation was 1.2 mm, and the maximum additional load on the outer wall was 6.4 kPa. These values conformed to the safety control index of the tunnel structure. The results also indicated that the foundation pit support system was reasonably arranged and that the deepening of the foundation pit exerted little impact on the subway tunnel and the surrounding environment.

In the simulation, when the groundwater level dropped by 2, 4, 8, and 12 m, the subway tunnel sunk by 3, 8, 18, and 24 mm, respectively. This result indicated that the change of the groundwater level was the key factor for the deformation of the tunnel structure. To prevent the abnormal deformation of the subway tunnel and the unusual operation of the subway caused by the drop of the groundwater level, recharge wells were deployed in the north side and in the north section of the east side of the foundation pit. The well was 18 m deep and could be recharged by a simple series-type water tank.

6. Deformation monitoring

Monitoring points for horizontal deformation of diaphragm wall and groundwater level are set in the east, south and north of foundation pit respectively by using diaphragm wall core-pulling inspection
hole and supplementary survey drilling hole. Furthermore, the wall top settlement, support axial force, column settlement, and subway cross-sectional deformation were monitored. Information-based construction was also utilized for improved management.

The excavation of soil for the foundation pit started in May 2017, and the basement structure was completed by mid-December. The horizontal deformation curve of the diaphragm wall with a waist drum shape is shown in Figure 8. The horizontal displacement of the top of the wall was 1.0–6.5 mm, and the settlement was 2.3–3.7 mm. The lateral horizontal displacements of the wall were 8.5–11.19 mm on the north side (monitoring points S1 and S2), 13.8–21.9 mm on the east side (S3 and S4), and 16.7 mm on the south side (S5). The deformation was relatively large at −7.0 m to −12.0 m. The horizontal displacement at the bottom of the pit was less than 2 mm. The settlement and horizontal displacement of the top of the diaphragm wall on the west side was 0.5–1.2 mm. The support axial force was 2,720–4230 kN. The maximum horizontal displacement of the subway tunnel was 2.0 mm, and the maximum settlement was 2.9 mm. The construction period coincided with the rainy season characterized by ample recharging surface water. Thus, the groundwater level was relatively stable with a variation of 0.5–1.2 m. The monitoring results of the deformation and axial force were basically consistent with the results of the 3D numerical simulation and typical support profile calculation.

![Horizontal Displacement](image)

**Fig 8. Monitoring curve of horizontal displacement of underground diaphragm wall**

### 7. Conclusion

This study make full use of the original part of a structure as a supporting component for the excavation deepening of the foundation pit. The methods not only reduced the deformation of the diaphragm wall in the excavated area but also avoided the waste caused by backfilling and excavation.

Considering the foundation pit layout, nearby environment, and current construction situation, this study designed an optimized support layout. The deformation of the foundation pit and its impact on a subway tunnel during excavation were simulated using Midas GTS/NX software to reveal potential risks. The key indicators for information-based construction, such as the horizontal deformation of the diaphragm wall, support axial force, and groundwater level, were monitored to
ensure safety.

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