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Stability of a Deep Foundation Pit with Hard Surrounding Rocks under Different in-Time Transverse Supporting Conditions

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Abstract: This paper focuses on investigating the stability of a deep foundation pit with hard surrounding rocks at different excavation stages with different supporting schemes by means of numerical calculations. The supporting schemes in question were combinations of one fixed vertical support and four varied transverse supports. Drilled grouting piles were used as vertical supports, and the commonly used steel bracings and prestressed anchorages served as transverse supports. The parameters used to evaluate the stability of the foundation pit at different excavation stages included the lateral displacements of the surrounding rocks, the settlement of the surrounding ground, the axial forces of steel bracings, and displacements at the tops of the drilled grouting piles. Simulation results showed that when a transverse supporting scheme consisting of one-layer steel bracings and prestressed anchorages set at 9 m and 22.5 m underground, respectively, was adopted, the lateral displacements of the surrounding rocks and settlement of the surrounding ground at different excavation stages were the largest compared to those under the other three transverse supporting schemes, while the corresponding values were lower compared to those allowed in Chinese standard GB50007-2011, demonstrating that this kind of supporting scheme is effective in terms of ensuring the safety of the foundation pit at different excavation stages. Moreover, the setting techniques for this kind of supporting scheme were relatively simple, and the corresponding influences of supporting element arrangements on excavation techniques were the lowest. Therefore, one-layer steel bracings and one-layer prestressed anchorages constituted the most suitable transverse supporting scheme for excavating a deep foundation pit with hard surrounding rocks.

Keywords: stability; deep foundation pit; hard surrounding rocks; transverse supporting; deformations

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1. Introduction

The development of urbanization is bringing more and more tests of traffic in cities. In order to ease traffic pressure, governments have taken various measurements, such as widening roads, building more flyovers, and developing underground traffic systems [1–3]. Among these measurements, underground traffic systems are the most effective since land resources are very scarce in cities. Subways are one of the most important parts of underground traffic systems, and subway stations are indispensable components of subways [4]. The construction of subway stations usually involves the excavation of a foundation pit. As subway stations are usually located in downtown areas, the construction environment may be complicated when considering the surrounding existing buildings, pipelines, main traffic roads, and subways, if any, as well as urban planning in the future [5–7]. Therefore, during the excavation of a foundation pit, lateral displacement of the surrounding rocks and settlement of the surrounding ground should be strictly controlled to ensure the safety of the surrounding buildings, structures, and so on.

Usually, the surrounding rocks of a foundation pit in downtown areas are soft soils or soft rocks, and it is more challenging for engineers and researchers to come up with a suitable supporting scheme to ensure the safety of foundation pits, especially deep foundation pits, because during excavation, excessive displacement of the surrounding soft soils or soft rocks may occur, threatening the stability of these deep foundation pits [8,9]. Usually, the supporting structures for excavating a deep foundation pit in soft soils or rocks include a retaining structure to restrain the displacement of the surrounding soils and inner supporting structures to control deformations of the retaining structure and the surrounding ground. However, it is not easy to determine supporting schemes for a deep foundation pit, since costs, manpower, construction sequences, and techniques, as well as the safety grade of deep foundation pits as specified in the Chinese standard of Code for Design of Building Foundations (GB50007-2011) [10], should also be taken into consideration [11].

During the past few decades, investigations at home and abroad have always concentrated on safety evaluations for deep foundation pits with soft surrounding rocks [12,13]. Wu et al. [14] adopted a support scheme consisting of bored piles and prestressed anchor cable supports combined with concrete corner bracings to ensure the safety of an ultra-deep foundation pit in soft soils influenced by groundwater. Niu et al. [15] adopted a combined support system consisting of prestressed composite pile-anchors and miniature steel pipe piles to guarantee the safety and stability of a deep foundation pit in fill soil areas. Li et al. [16] adopted a support scheme consisting of a suspended diaphragm wall with two rows of struts set in the upper part and five prestressed tiebacks set in the lower part. Ma et al. [17] adopted a support scheme consisting of underground diaphragm walls with four inner supports composed of concrete bracings and pre-stressed steel bracings. The above investigation results indicated that proper in-time supporting measures consisting of adopting retaining structures and internal supports are able to guarantee the safety and stability of a deep foundation pit with soft surrounding rocks during excavation processes.

However, geological conditions are distinct in different areas [18,19]. Subway station construction in urban areas may also involve dealing with geological conditions such as hard rock strata. For foundation pits with hard surrounding rocks, conventional in-time supporting schemes and sequences for deep foundation pits with soft surrounding rocks are effective enough to control the corresponding displacements during excavation process but may lead to over-support of the foundation pits considering that the surrounding hard rocks have certain strengths and thus the displacement of the surrounding rocks is relatively lower than that of soft surrounding rocks [20,21]. As a result, conventional in-time supports for foundation pits with hard surrounding rocks could lead to unnecessary costs, increased manpower, and longer construction durations. Therefore, selecting appropriate supporting measurements for foundation pits with hard surrounding rocks is significant in terms of lowering both costs and the length of the construction period.

In this investigation, in order to find the most suitable supporting measurements for excavating a deep foundation pit with hard surrounding rocks, a conventional support consisting of a retaining structure and inner supports, which is also usually used for supporting deep foundation pits in soft soils, was adopted. Four different inner supporting schemes were taken into consideration to investigate whether it is necessary to set inner supports, as well as how to set inner supports, for ensuring the safety of deep foundation pits during excavation process. The retaining structure was made of drilled grouting piles. The inner supports included steel bracings and prestressed anchorages. The stability of the foundation pit under the four different supporting conditions was evaluated according to the lateral displacements of the surrounding rocks, settlement of the surrounding ground, the axial forces of the steel bracings, and displacements at the tops of drilled grouting plies, which were determined by means of numerical finite element calculations.

2. Engineering Profile

A deep foundation pit was excavated for the purpose of constructing a subway transfer station, which consisted of a standard section with an excavation depth of 24 m and a transfer section with an excavation depth of 31 m along the length direction. The corresponding excavation length and width were 217.5 m and 21.5 m, respectively. The surrounding rocks of the foundation pit along the excavation depths were miscellaneous fill from the ground to two meters underground, strong and medium weathered limestones between two meters and eight meters underground, marl located at eight to fifteen meters underground, limestone in the range of fifteen to eighteen meters underground, and weak weathered limestones at the bottom, from which it could be obtained that the surrounding rocks of the foundation pit were relatively hard.

3. Establishment of Numerical Model for the Foundation Pit

The detailed procedures of establishing the numerical model and the calculation process are illustrated in Figure 1. The establishment of the numerical model for the foundation pit included building a geometrical model, determining boundary conditions, assigning necessary parameter values of the geologies and support elements for numerical calculation, and choosing a suitable constitutive model and calculation units for geologies and support elements, respectively. The construction of the geometrical model was based on the excavation depth, length, and width of the practical deep foundation pit. Boundary condition determination was based on the deformation characteristics of the foundation pit during the excavation process. The necessary parameter values of the geologies and support elements were derived from indoor tests of the samples obtained from the construction sites. And a constitutive model for the surrounding rocks and calculation units for support elements were created and determined, respectively, based on the corresponding deformation characteristics. Then, the established numerical model was meshed by using radioactive grids to densify the periphery of the foundation pit. The meshed model consisted of 492,310 units in total.

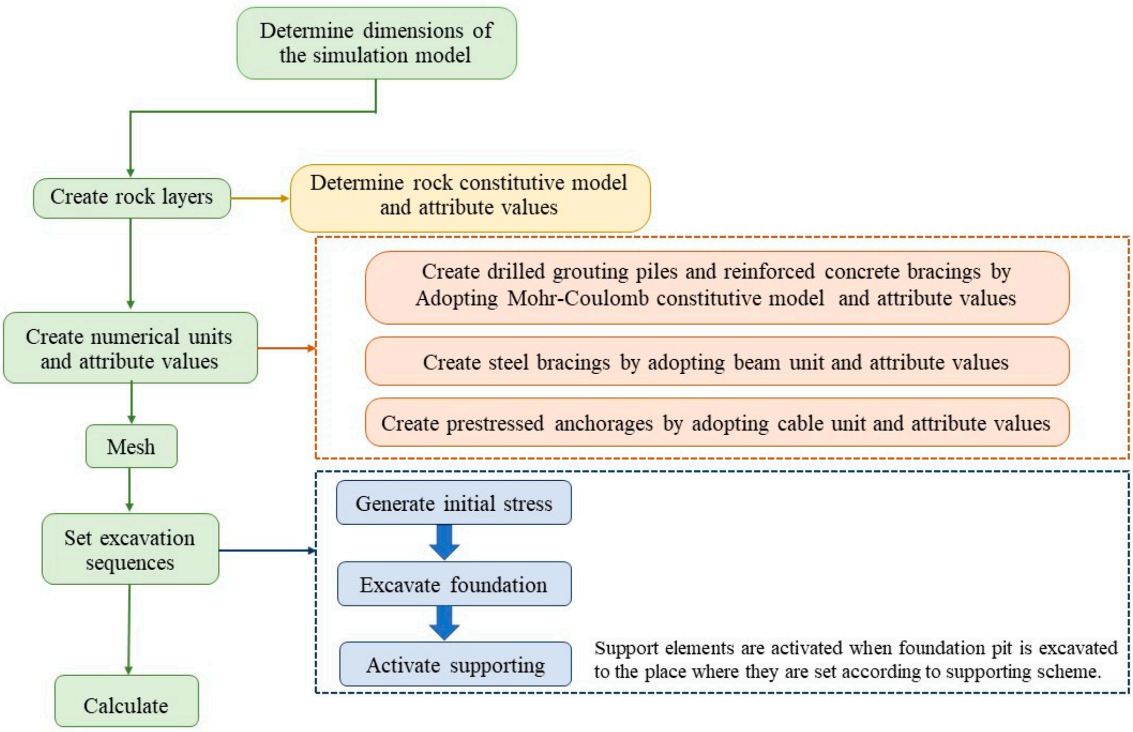


Figure 1. Flowchart depicting the detailed procedure of establishing the numerical model and the calculation process.

3.1. Geometrical Dimensions of the Simulation Model

For the purpose of accurately analyzing the stability of the deep foundation pit at different excavation stages, a simulation model with a length of 379 m, a width of 189 m, and a depth of 75 m was established using the commercial finite element software product FLAC3D 6.0 version, as shown in Figure 2. The simulation model not only consisted of the foundation pit but also included potential influential and uninfluential areas. In order to investigate the settlement characteristics of the surrounding ground at different excavation stages, the potential influential areas of surrounding ground and the potential uninfluential areas were set to three-to-four times and one-to-two times the excavation depth, respectively.

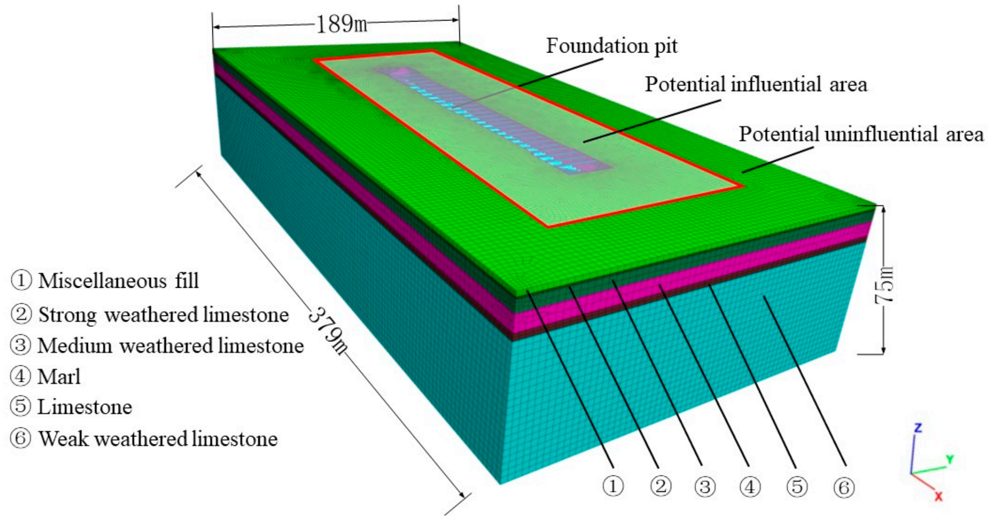


Figure 2. Numerical model for analyzing the stability of the deep foundation pit.

3.2. Boundary Conditions

The boundaries for the numerical model obtained from FLCA3D are illustrated in Figure 3. During the excavation process, basically no deformations took place at the bottom of the foundation pit; therefore, fixed boundaries were set at the bottom of the numerical model. In terms of the sides of the foundation pit, lateral displacements were the main deformation forms, and slip boundaries were applied to sides of the simulation model. The excavation of the foundation pit was from top to bottom, and thus free boundary was set at top of the simulation model.

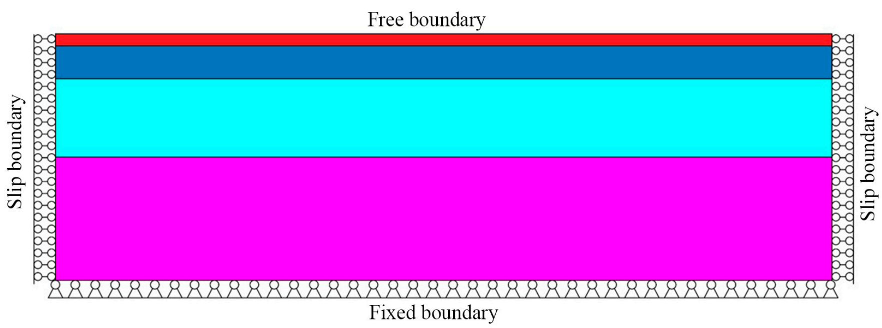


Figure 3. Boundary conditions.

During the excavation process, temporary traffic roads were applied to ensure normal traffic. As the construction site of the subway transfer station was located near the main traffic roads, additional vertical loads were applied by the vehicles around the foundation pit. Therefore, vertical loads needed to be added to the top of the simulation model.

According to traffic statistics, the vertical loads were simplified as a uniform load with a value of 20 kPa.

3.3. Supporting Schemes and Parameters for the Simulation Model

3.3.1. Supporting Schemes

In this investigation, both vertical and transverse supports were used to guarantee the stability of the foundation pit at different excavation stages. For vertical support, drilled grouting piles with a diameter of 1000 mm were utilized to control lateral displacements of the surrounding rocks. The pile intervals at standard and transfer sections were 1000 mm and 500 mm, respectively, and the pile lengths at standard and transfer sections were 24 m and 31 m, respectively. For transverse supports, commonly used support elements such as reinforced concrete bracings and steel bracings were adopted for further restraining the lateral displacement of the surrounding rocks and the settlement of the foundation pit at different excavation stages. Considering that the surrounding rocks at eight to fifteen meters underground were marl, prestressed anchorages were also adopted to further control displacements of the surrounding rocks at different excavation stages. The layout of the support elements is illustrated in Figure 4.

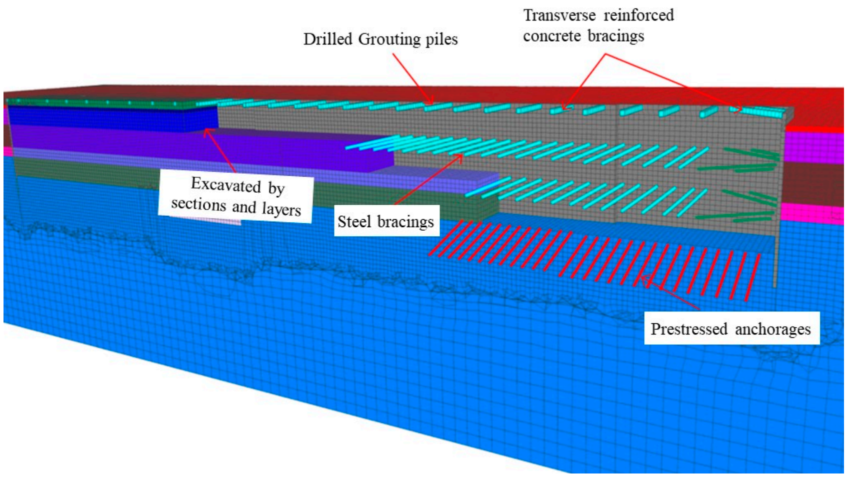
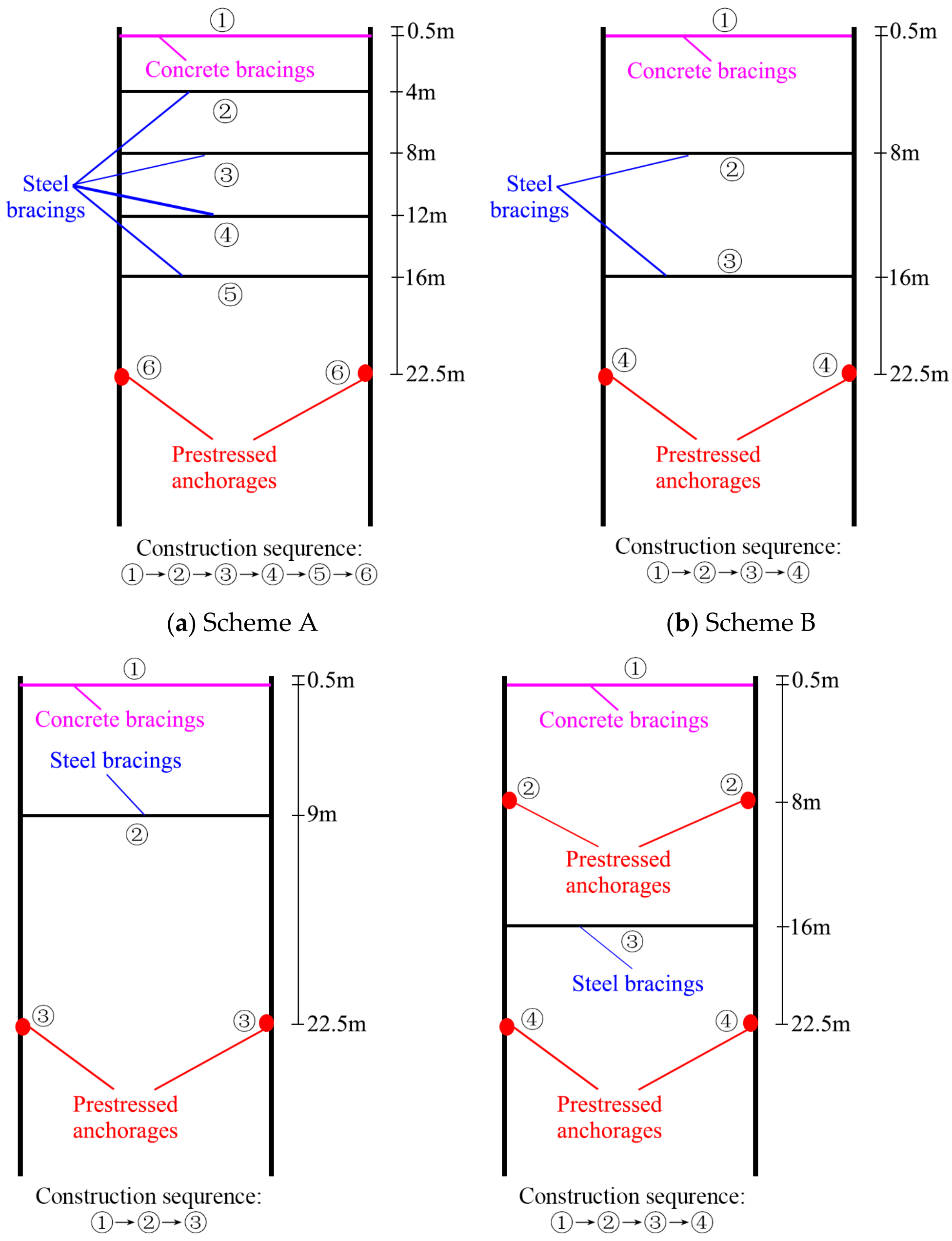


Figure 4. Schematic diagram of the support scheme of the foundation pit.

According to the excavation depth of the foundation pit and the space required for the construction equipment, the minimum supporting depth for steel bracings was set to 4 m. Meanwhile, to further control the displacements of the surrounding rocks at different excavation stages, prestressed anchorages were set at 8 m and/or 22.5 m underground. In order to investigate the influences of steel bracings and prestressed anchorages on the stability of the foundation pit at different excavation stages, four different in-time transverse supporting schemes were considered in this simulation. The arrangements of the transverse supporting elements for the four in-time supporting schemes are illustrated in Table 1 and Figure 5.

Table 1. Supporting schemes.

Scheme Type	Steel Bracing		Prestressed Anchorage		Row Distance/m
	Layer	Depth/m	Layer	Depth/m	
A	4	4, 8, 12, 16	1	22.5	4
B	2	8, 16	1	22.5	8
C	1	9	1	22.5	9
D	1	16	2	8, 22.5	8



(**a**) Scheme A (**b**) Scheme B (**c**) Scheme C (**d**) Scheme D

Figure 5. Support element arrangements and construction sequences of the four supporting schemes.

For the vertical supports, drilled grouting piles were set ahead of foundation pit excavation. For the transverse supports, reinforced concrete bracings were set at the top of the foundation pit, and steel bracings and prestressed anchorages were set when the excavation depth was 0.5 m deeper than the corresponding locations provided in Table 1.

(1) Scheme A

As shown in Figure 5a, supporting scheme A included four-layer steel bracings at 4 m, 8 m, 12 m, and 16 m underground, respectively, and one-layer prestressed anchorages at 22.5 m underground. The raw distance of these transverse supporting elements was set to 4 m. This kind of supporting combination was advantageous for ensuring safety of the foundation pit at different excavation stages, but the corresponding costs were higher among the four supporting schemes, considering material cost, manpower costs, and construction equipment cost. Meanwhile, the construction sequence of scheme A is rather complicated, and setting too many supporting elements along the excavation depth is disadvantageous for construction spaces.

(2) Scheme B

As shown in Figure 5b, supporting scheme B consisted of two-layer steel bracings located at 8 m and 16 m underground and one-layer prestressed anchorages 22.5 m beneath the ground. The raw distance of these transverse supporting elements was set to 8 m. This kind of supporting combination might be sufficient for controlling the lateral displacements of the surrounding rocks, thus ensuring the safety of the foundation pit at different excavation stages. The corresponding supporting techniques are relatively simple, and the construction costs are lower than those of Scheme A.

(3) Scheme C

As shown in Figure 5c, supporting scheme C included one-layer steel bracings 9 m beneath ground and prestressed anchorages located at 22.5 m underground. The raw distance of these transverse supporting elements was set to 9 m. Though this kind of supporting combination had the lowest cost and involved the simplest construction techniques, as well as the largest construction spaces, the lateral displacements of the surrounding rocks might not be sufficiently restrained, and thus the stability of the foundation pit at different excavation stages might be influenced.

(4) Scheme D

As shown in Figure 5d, supporting scheme D consisted of one-layer steel bracings 16 m beneath ground and two-layer prestressed anchorages located at 8 m and 22.5 m underground. The raw distance of these transverse supporting elements was set to 8 m. This kind of supporting combination was advantageous for controlling lateral displacements of the surrounding rocks and leaving enough construction space during the excavation process. However, the setting techniques for prestressed anchorages were complicated, and the corresponding costs for this kind of supporting combination might be the highest among all the supporting schemes.

3.3.2. Parameters Needed for Numerical Calculations

(1) Geological parameters

The parameters of each kind of surrounding rock used for numerical calculation in the simulation model are shown in Table 2. Parameters such as the elastic modulus, cohesion, and internal friction angle were determined via indoor tests of the rock samples obtained at construction sites.

Table 2. Geological parameters for the surrounding rocks of the foundation pit.

Geotechnical Stratum	Depth (m)	Unit Weight (kN/m^3)	Elastics Modulus (MPa)	Cohesion (kPa)	Internal Friction Angle (°)	Poisson's Ratio
miscellaneous fill	0~2	16.5	40	25	5.5	0.3
strong weathered limestone	2~3	19.5	70	45	8	0.3
medium weathered limestone	3~8	20.3	70	45	8.2	0.2
marl	8~15	25	110	25	9.5	0.2
limestone	15~18	25	110	25	10	0.2
weak weathered limestone	>18	27.5	1150	150	13.5	0.15

(2) Reinforced concrete bracings and drilled grouting piles

The raw material used to fabricate these kinds of supporting elements was concrete, the properties of which are basically similar to those of rocks. Therefore, in order to calculate deformations of these supporting elements, the Mohr–Coulomb constitutive model was applied. The parameters needed for numerical finite element calculations included density, cohesion, and so on, the values of which were the same as those required in the practical support design scheme of the foundation pit and are provided in Table 3.

Table 3. Parameters used for numerical calculations applied to drilled grouting piles and reinforced concrete bracings.

Supporting Type	Density (kg/m^3)	Elastic Modulus (GPa)	Cohesion (MPa)	Internal Friction Angle (°)	Tensile Strength (MPa)	Poisson's Ratio
Drilled grouting piles	2500	31.5	20	18	3	0.25
Transverse reinforced concrete bracings	2500	31.5	20	18	3	0.25

(3) Steel bracings

The elastic modulus, section area, and Possion's ratio of the steel bracings were 200 GPa, 0.0394 m^2, and 0.3, respectively, and these values are also same as those required in the practical support design scheme of the foundation pit. Considering that steel bracings are mainly influenced by axial forces, a beam unit suitable for axially stressed components was applied for the calculation of deformations and stresses of the steel bracings in the simulation model. To calculate the deformations and stresses of the steel bracings using FLAC3D, parameters such as the Z-axial inertial moment should be provided. The values of the Z-axial inertia moment, Y-axial inertial moment, and section areas were determined according to dimensions of the steel bracings in the practical support design scheme and are provided in Table 4.

Table 4. Beam unit parameters of steel bracings.

Elastic Modulus (GPa)	Section Area (m^2)	Z-Axial Inertia Moment (m^3)	Y-Axial Inertial Moment (m^3)	Polar Inertia Moment (m^3)	Poisson's Ratio
200	0.0394	0.003	0.003	0	0.3

(4) Prestressed anchorage

For prestressed anchorages, the corresponding anchorage and unanchored lengths were 8 m and 9 m, respectively. The grouting perimeter of the prestressed anchorages was 0.565 m. To calculate the stresses and strains of prestressed anchorages, cable units were used, and the corresponding parameters needed for calculation included prestressed force, elastic modulus, section area, and so on, as illustrated in Table 5. All the values used for numerical calculation were the same as those required in the practical support design scheme of the deep foundation pit.

Table 5. Cable unit parameters of prestressed anchorages.

Prestressed Force (kN)	Elastic Modulus (GPa)	Section Area (m^2)	Unanchored Length (m)	Anchorage Length (m)	Bond Stiffness (MPa)	Grouting Perimeter (m)
200	200	0.002	9	8	27.5	0.565

3.4. Parameters for Analyzing Stability of the Foundation Pit during Excavation Process

Lateral displacements of the surrounding rocks and settlement of the surrounding ground are the main direct parameters that are clearly required in GB50007-2011. Therefore, the stability of the foundation pit at different excavation stages could be evaluated based on the above two parameters. In GB50007-2011 [10], the values of the above two parameters should be less than 0.3% and 0.15% of the excavation depth, respectively.

Apart from the above direct parameters, the stability of the foundation pit at different excavation stages could also be evaluated according to the stress conditions and deformation characteristics of the supporting elements. Therefore, the parameters used to analyze the stability of the deep foundation pit with hard surrounding rocks at different excavation stages included the lateral displacement of the surrounding rocks, settlement of the surrounding ground, axial forces of steel bracings, and displacement at the tops of the drilled grouting piles.

4. Simulation Results and Discussion

4.1. Lateral Displacements of the Surrounding Rocks

(1) At an excavation depth of 8 m

The lateral displacements of the surrounding rocks when the excavation of the foundation pit reached 8 m beneath the ground are illustrated in Figure 6. According to Figure 6, when the excavation depth was 8 m, the maximum lateral displacements of the surrounding rocks were basically similar under different in-time transverse supporting conditions. Before the excavation of the foundation pit, vertical supports of the drilled grouting piles were set. And when excavation of the foundation pit began, transverse supports of concrete bracings were set at a depth of 0.5 m. Then, with continuous excavation of the foundation pit, steel bracings or prestressed anchorages were set when the excavation depth was 0.5 m deeper than the preset depths of the supporting elements, meaning that when the excavation depth was 8 m, the support conditions of the foundation pit were the same for supporting schemes B, C, and D but not scheme A. Therefore, the deformations of the surrounding rocks of the foundation pit under supporting Schemes B, C, and D were the same, while they were relatively smaller when Scheme A was adopted.

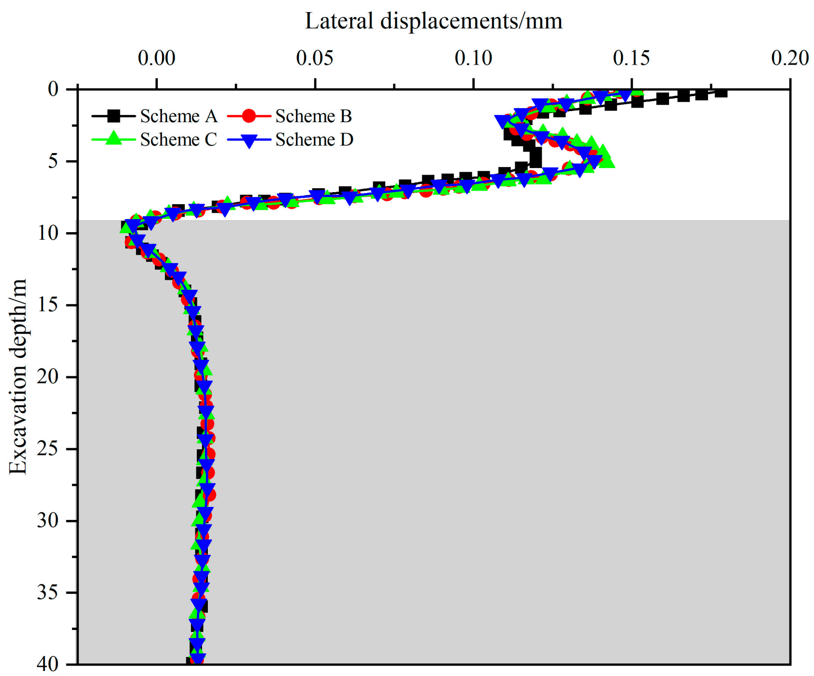


Figure 6. Lateral displacements of the surrounding rocks at an excavation depth of 8 m.

The similarity of these lateral deformations indicated that lateral deformations of the surrounding rocks could be fully restrained by setting the drilled grouting piles and concrete bracings to a depth of 0.5 m and that it might be unnecessary to set steel bracings at a depth of 4 m (Scheme A). Meanwhile, when the excavation depth was 8 m, the maximum lateral displacement of the surrounding rocks was 0.17 mm, which is far lower than the allowed value of 24 mm required in GB50007-2011 [10].

(2) At an excavation depth of 16 m

The lateral displacements of the surrounding rocks when the excavation of the foundation pit reached 16 m beneath the ground are illustrated in Figure 7. When the excavation depth was 16 m, the displacement changes in the surrounding rocks under supporting schemes A, B, and C were basically identical, presenting a P-shape in the excavation direction, while displacements of the surrounding rocks under supporting scheme D were restrained. Meanwhile, the places where maximum lateral displacements occurred were basically located at 7 m underground under the supporting conditions of scheme A, B, and C. Furthermore, the maximum lateral displacement of the surrounding rocks was 2.3 mm when the excavation depth was 16 m, which is far less than the allowed value of 48 mm required in GB50007-2011 [10].

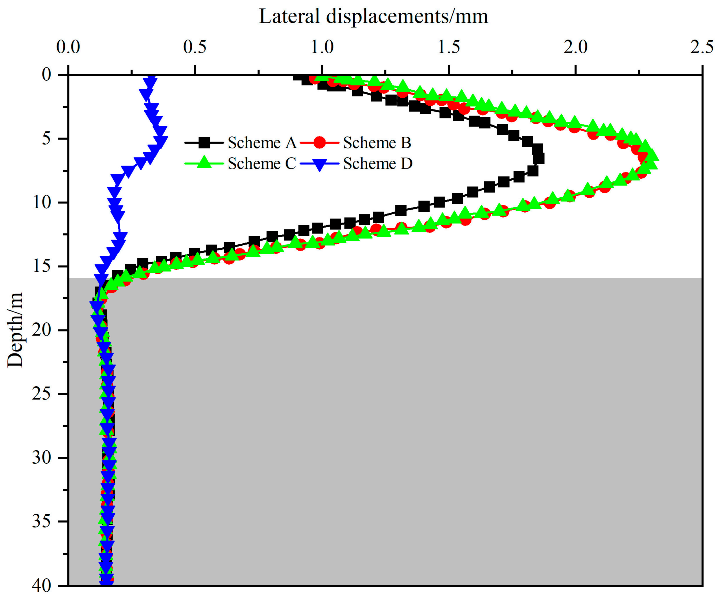


Figure 7. Lateral displacements of the surrounding rocks at an excavation depth of 16 m.

As shown in Table 2, at an excavation depth of 2 m to 8 m, mainly strongly weathered limestone and medium-weathered limestone were found, indicating that surrounding rocks at an excavation depth of 2 m to 8 m might be fissured. Therefore, the deformation controlling the performance of these steel bracings located at 4 m, 8 m, and 9 m was not as intense as that of the prestressed anchorages. However, steel bracings located at 4 m, 8 m, and 9 m somehow served to control the lateral deformations of the fissured surrounding rocks. The lateral deformations of the foundation pit with prestressed anchorages set to 8 m were significantly restrained, and they were lower when steel bracings were set at 4 m, 8 m, and 12 m (Scheme A). Additionally, the lateral deformations of the surrounding rocks when steel bracings were set to 8 m were relatively smaller than those when the steel bracings set at 9 m.

(3) At excavation depths of 24 m and 31 m

The lateral displacements of the surrounding rocks when the excavation of the foundation pit reached 24 m and 31 m beneath ground are illustrated in Figure 8. When the excavation depths were 24 m and 31 m, the lateral displacements of the surrounding rocks under supporting schemes A, B, and C still presented a P-shape, while those under supporting scheme D presented a B-shape, indicating that setting two-layer prestressed anchorages to 8 m and 22.5 m underground was effective in restraining lateral displacements due to improvements in the integrity of the surrounding rocks.

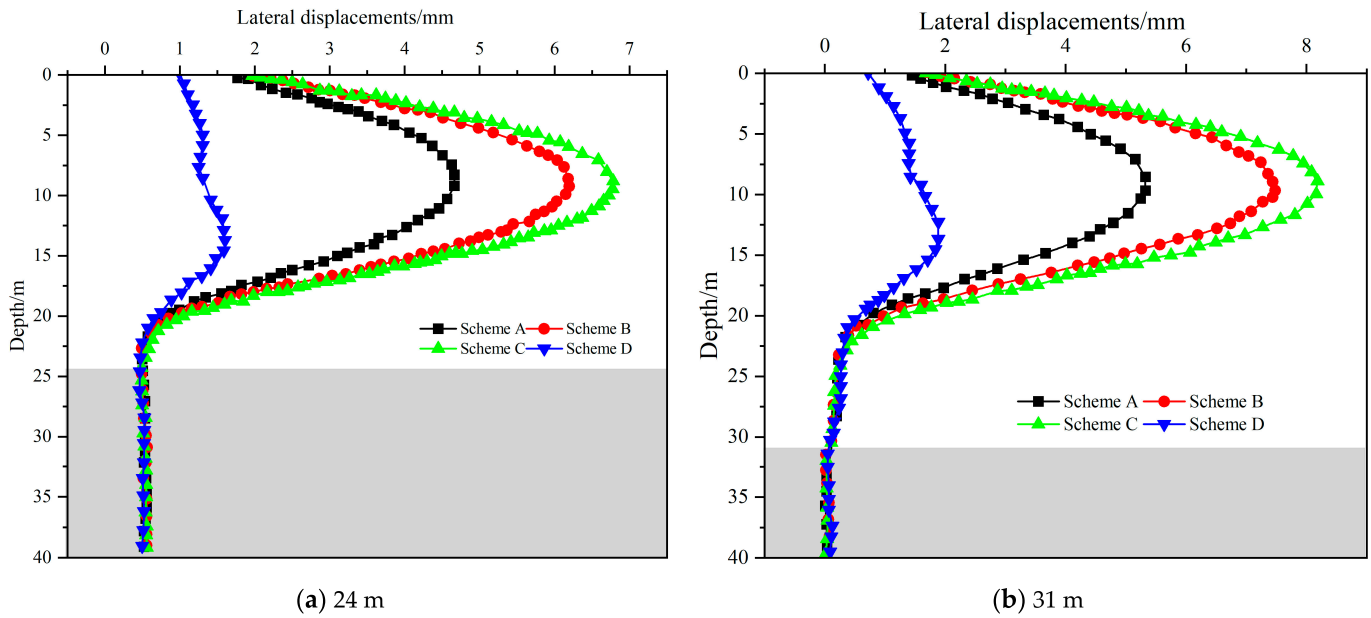


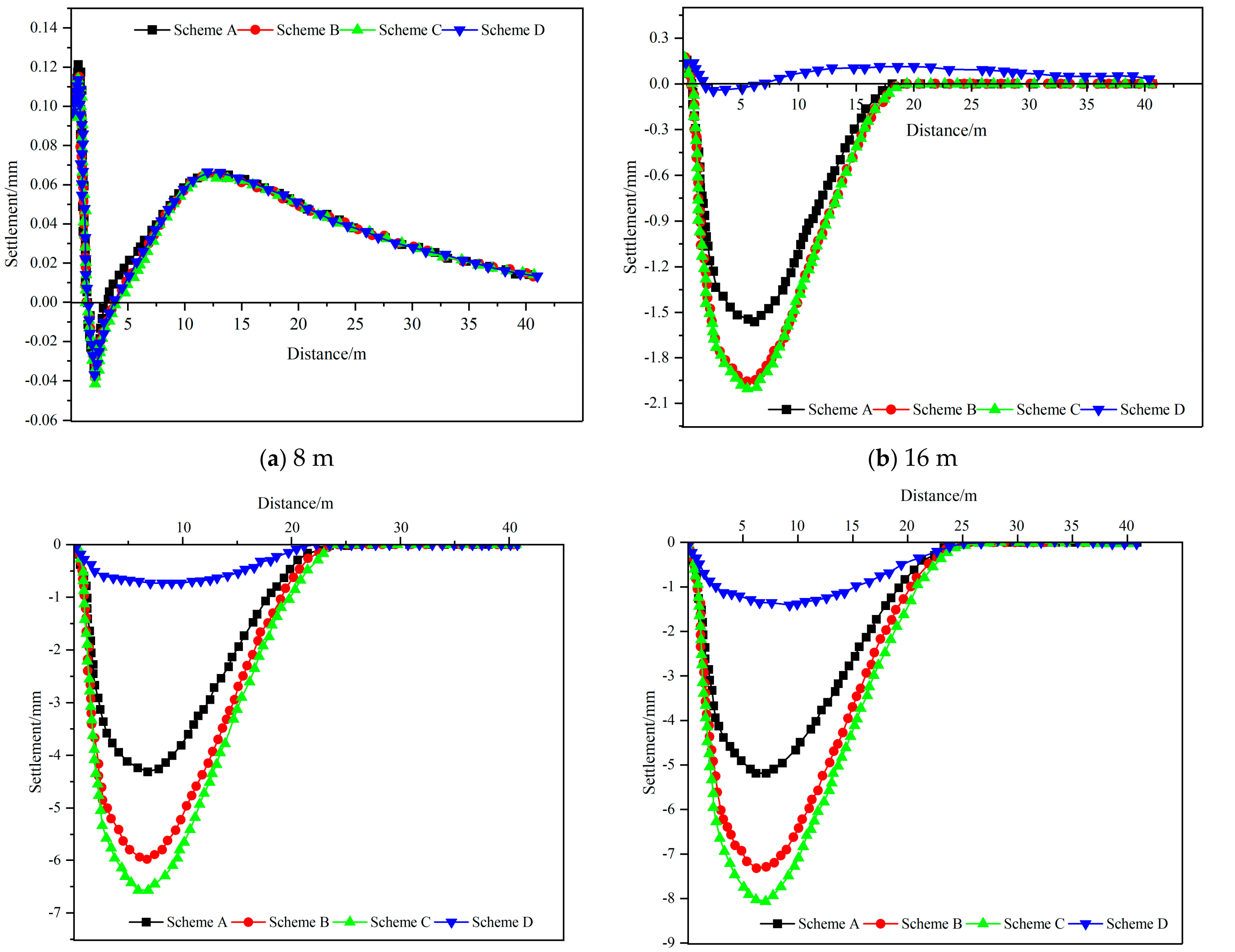
Figure 8. Lateral displacements of the surrounding rocks at excavation depths of 24 m and 31 m.

Meanwhile, the lateral displacements of the surrounding rocks under the supporting conditions of scheme A were less than those under the supporting conditions of scheme B, and those under supporting conditions of scheme C were the largest, demonstrating that when the excavation depth was greater than 16 m, the transverse support arrangements of scheme C were less effective in terms of restraining the lateral displacements of the surrounding rocks compared to those of scheme D. Furthermore, when the excavation depths were 24 m and 31 m, the maximum lateral displacements of the surrounding rocks were 7 mm and 8.1 mm, respectively, which are significantly lower than those for excavating deep foundation pits in soft soils or rocks reported in [22], with maximum lateral deformation being more than 60 mm when adopting FE numerical calculation model; in [23], with maximum lateral displacements being more than 50 mm when adopting PLAXIS v.9 software; in [24], with maximum horizontal displacement being 34.5 mm at a depth of 5 m when adopting Midas GTS NX 2022 R1; and in [25], with the maximum horizontal displacement observed being about 35 mm when excavation was completed.

4.2. Settlement of the Surrounding Ground

(1) At an excavation depth of 8 m

The settlement values of the surrounding ground at different excavation depths are shown in Figure 8. It can be obtained from Figure 9 that soil heave of the surrounding ground appeared when the excavation depth was 8 m, and the maximum heave value was 0.12 mm. Excavation unloading will lead to heave or settlement of the surrounding ground. If the settlement is lower than the heave, then the surrounding ground will present heave characteristics. According to the lateral deformations of the surrounding rocks illustrated in Figure 5, when the exaction depth was 8 m, the maximum lateral deformation was less than 0.2 mm, indicating that the influences of excavation unload on the settlement of the surrounding ground might be less severe than those on heave. Therefore, when the excavation depth was 8 m, heave took place in the surrounding ground.



(**a**) 8 m (**b**) 16 m

(**c**) 24 m (**d**) 31 m

Figure 9. Settlement of the surrounding ground.

(2) At an excavation depth greater than 8 m

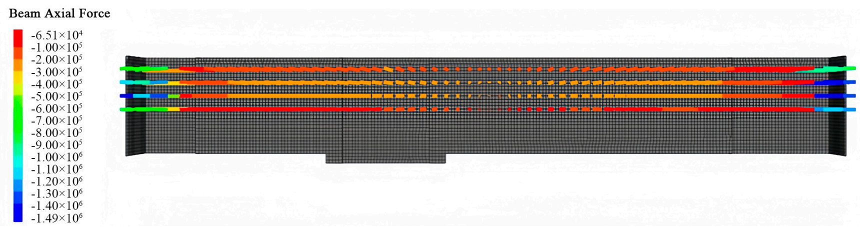
When the excavation depth was greater than 8 m, the settlement changes of the surrounding ground were basically identical when different transverse supporting schemes were adopted. The settlement of the surrounding ground at different distances from the foundation pit presented a funnel shape, and the influential areas of the foundation pit excavation were within 1 to 1.25 times the excavation depth.

Meanwhile, the influential areas of the surrounding ground during the foundation pit excavation process increased with an increasing excavation depth. When the excavation depth was 16 mm, the settlement of the surrounding ground under transverse supporting schemes A, B, C, and D occurred within 20 m, 22 m, 22 m, and 10 m away, respectively, from the foundation pit. Meanwhile, soil heave also took place when the excavation depth was 16 m when supporting scheme D was adopted, and this might have been caused by prestresses of the prestressed anchorages. Upon further increasing the excavation depth to 24 m and 31 m, the influences of excavation unloading on the settlement of the surrounding ground were more severe than those of heave, and no heave took place around the foundation pit. Meanwhile, influential areas of the surrounding ground were basically identical and within 25 m from the foundation pit under the four transverse supporting schemes, which was because when the excavation depth was 24 m, all the transverse support elements had been set, and the settlement of the surrounding ground was therefore restrained due to the deformation-restraining effects of the transverse support elements.

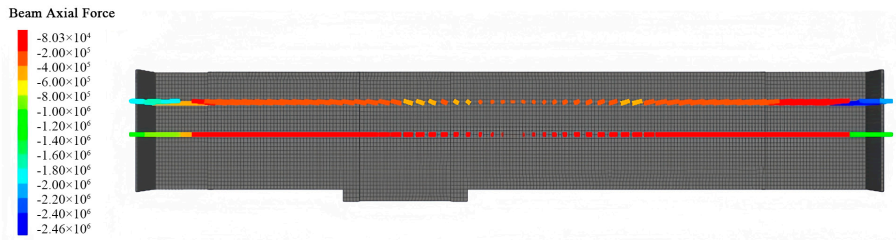
Furthermore, it can be obtained from Figure 9b–d that in terms of controlling the settlement of the surrounding ground during the foundation pit excavation process, transverse supporting scheme D was the most effective, followed by scheme A and scheme B, and scheme C was the least effective. Meanwhile, when the excavation depths of the foundation pit were 16 m, 24 m, and 31 m, the settlement of the surrounding ground under transverse supporting scheme C was the highest among all the other transverse supporting schemes, which were 2 mm, 6.6 mm, and 8 mm, and all these maximum settlement values were significantly lower than those reported in [26], with the maximum numerically calculated settlement being 20 mm with an excavation depth of 18 m, and in [27], with the maximum numerically calculated settlement being about 24 mm.

4.3. Axial Force of the Steel Bracings

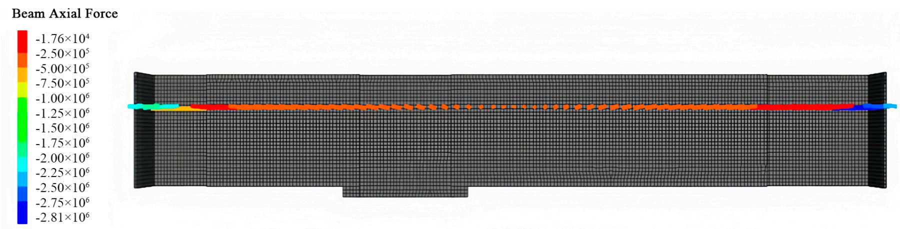
The axial forces of the steel bracings when the excavation of the foundation pit reached 31 m beneath the ground are shown in Figures 10 and 11. According to Figure 10, the axial forces at both ends of the steel bracings were high, while those at the middle of the steel bracings were low. According to Figure 11, when transverse supporting scheme A was adopted, the axial forces of the steel bracings at 4 m, 8 m, 12 m, and 16 m were respectively 258 kN, 344 kN, 320 kN, and 177 kN. When transverse supporting scheme B was adopted, the axial forces of the steel bracings at 8 m and 16 m were 454 kN and 253 kN, respectively. When transverse supporting schemes C and D were adopted, the axial forces of the corresponding steel bracings at 9 m and 16 m were 536 kN and 160 kN, respectively. Therefore, the axial forces of the steel bracings were closely related to the arrangements of the supporting elements.



(**a**) Scheme A.

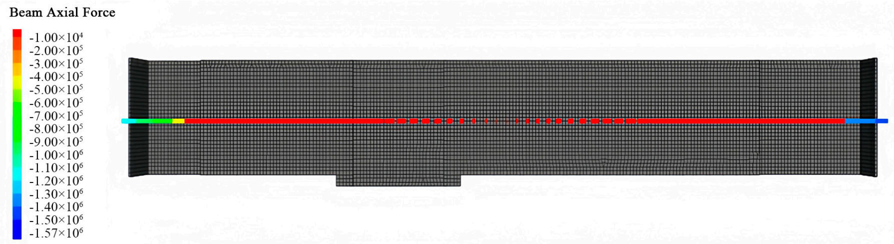


(**b**) Scheme B.



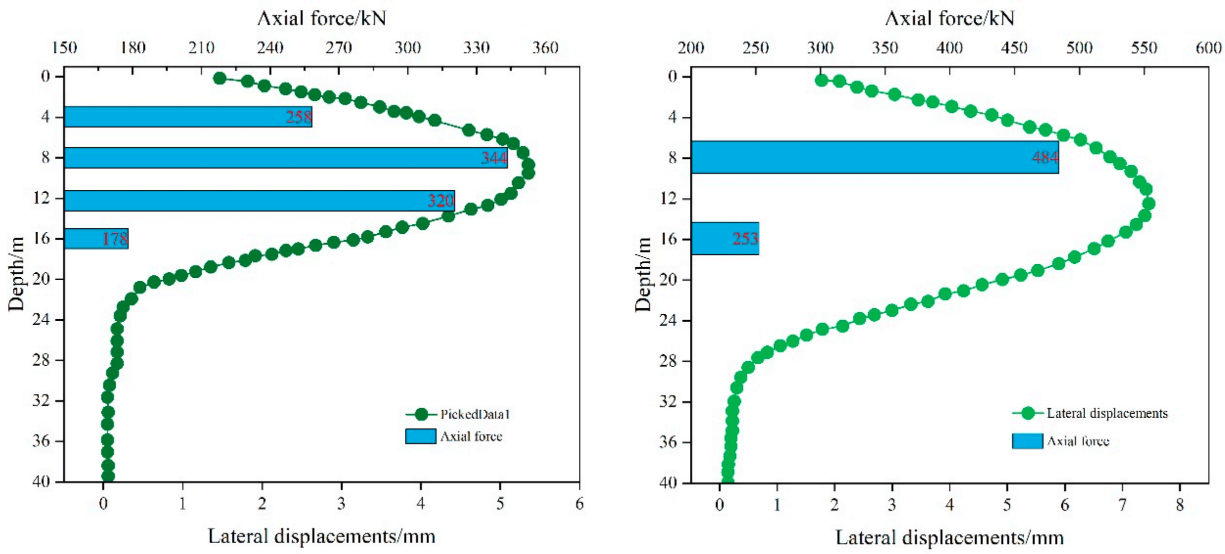
(**c**) Scheme C.

Figure 10. *Cont.*

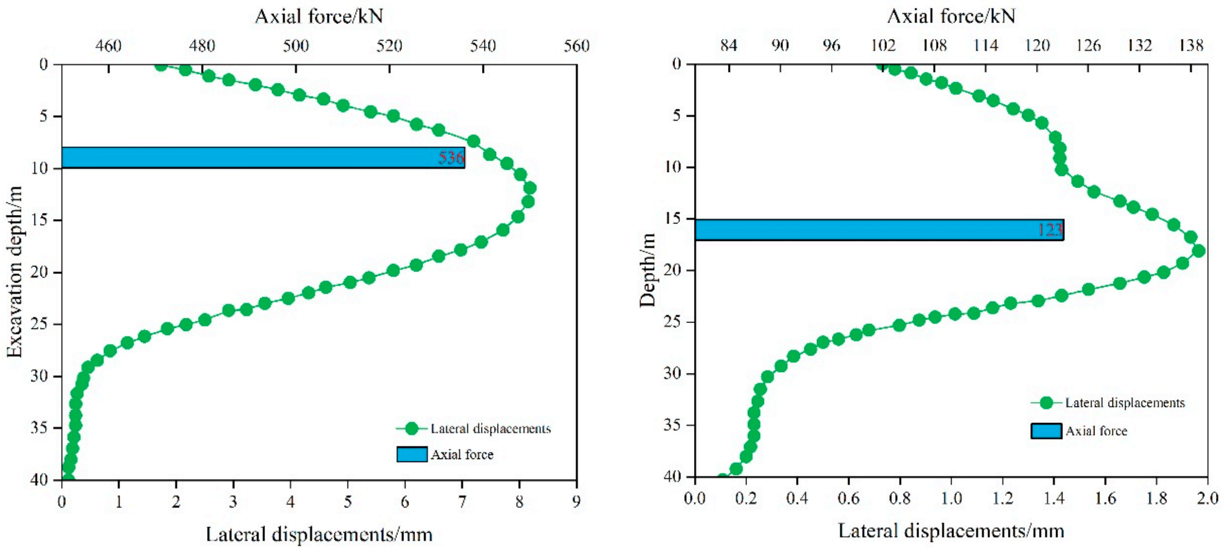


(**d**) Scheme D.

Figure 10. Axial force contour of the steel bracing.



(**a**) Scheme A (**b**) Scheme B



(**c**) Scheme C (**d**) Scheme D

Figure 11. Maximum axial forces of the steel bracings.

Meanwhile, it can also be obtained from Figure 10 that the axial forces of the steel bracings near the prestressed anchorages were obviously lower compared to those located at other layers, which further indicated that prestressed anchorages were effective in terms of improving the integrity of the surrounding rocks and thus avoid excessive displacements of the surrounding rocks.

4.4. Displacements at the Tops of the Drilled Grouting Piles

Displacements at the tops of the drilled grouting piles are closely related to settlement of the surrounding ground, thus making the latter an important parameter for evaluating the stability of a foundation pit at different excavation stages. Considering that the foundation pit was excavated in separate sections from south to north, the displacements at the tops of the drilled grouting piles corresponding to different sections of the foundation pit at different excavation depths were also calculated, including the south end, north end, standard section, and transfer section. The displacements at the tops of the drilled grouting piles corresponding to different sections at different excavation depths are illustrated in Figure 12.

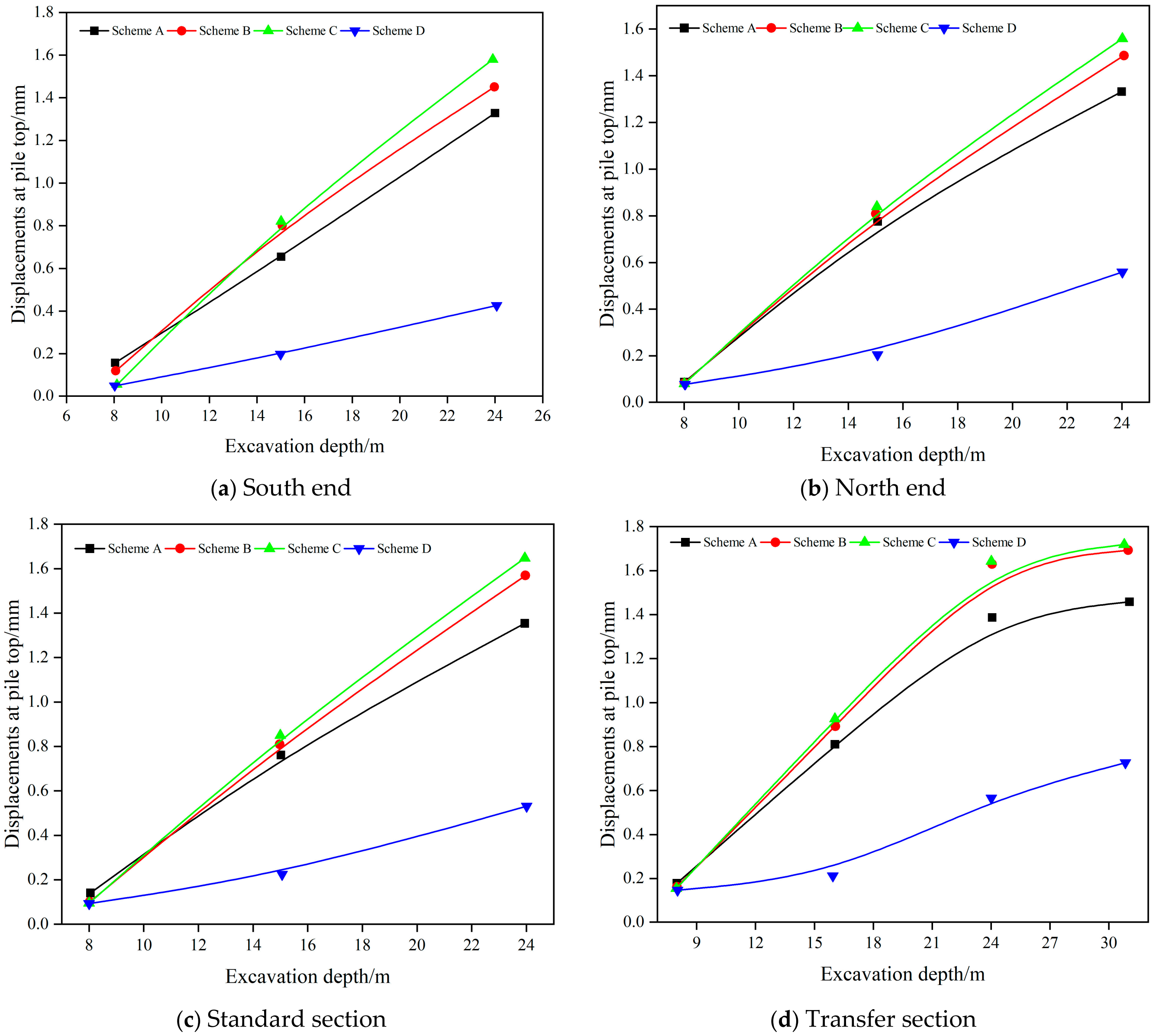


Figure 12. Displacements at tops of the drilled grouting piles at different sections.

It can be obtained from Figure 12 that the displacements at the tops of the drilled grouting piles corresponding to the four typical sections gradually increased with an increasing excavation depth. Meanwhile, when transverse supporting scheme D was adopted, the maximum displacements at the tops of the drilled grouting piles corresponding to the four typical sections were the smallest, which was because prestressed anchorages set at 8 m underground could effectively restrain the movements of the upper weathered limestone layer and thus reduced displacement of the surrounding rocks. When transverse supporting Scheme C was adopted, the maximum displacements at the tops of the drilled grouting piles corresponding to the four typical sections were 1.63 mm, 1.68 mm, 1.71 mm, and 1.73 mm, which all meet the displacement requirements of 20 mm stipulated in GB50007-2011 [10].

5. Conclusions

In this investigation, numerical finite element calculations were adopted to analyze the stability of a deep foundation pit with hard surrounding rocks under fixed vertical supports of drilled grouting piles and varied transverse supports of steel bracings and prestressed anchorages. The parameters used for stability evaluation included the lateral displacement of the surrounding rocks, settlement of the surrounding ground, axial forces of the steel bracings, and displacement at the tops of the drilled grouting piles. According to the simulation results, the following conclusions could be drawn.

(1) After setting the two-layer prestressed anchorages to 8 m and 22.5 m underground and one-layer steel bracings to 16 m beneath ground, the lateral displacements of the surrounding rocks were the lowest, while setting the one-layer steel bracings to 9 m underground and one-layer prestressed anchorages to 22.5 m beneath the ground led to the highest lateral displacements of the surrounding rocks.

(2) The four different transverse supporting schemes of different layers of steel bracings and prestressed anchorages were effective in terms of controlling lateral displacements of the surrounding rocks and settlement of the surrounding ground, the values of which at different excavation stages were relatively lower compared to those required in the Chinese standard of GB50007-2011.

(3) Though the transverse supporting scheme consisting of setting two-layer prestressed anchorages at 8 m and 22.5 m beneath ground and one-layer steel bracings at 16 m underground could effectively control lateral displacements of the surrounding rocks and settlement of the surrounding ground, the corresponding construction techniques were rather complicated, and the foundation pit was over-supported compared with the other three transverse supporting schemes, leading to a waste of supporting elements and labor resources, prolonged construction durations, and increased construction costs.

(4) Though the displacements of the surrounding rocks were the second smallest when setting four-layer steel bracings at 4 m, 8 m, 12 m, and 16 m underground and one-layer prestressed anchorages at 22.5 m underground, the four-layer steel bracings occupied too much space for excavation, which obviously negatively influenced the excavation techniques applied to the foundation pit.

(5) Though the lateral displacements of the surrounding rocks and settlement of the surrounding ground were the highest when setting one-layer steel bracings at 9 m underground and one-layer prestressed anchorages at 22.5 m beneath ground, the corresponding values were relatively lower compared to those illustrated in the Chinese standard of GB50007-2011.

(6) Compared with the deformations of the retaining structures for excavating deep foundation pits in soft soils or soft rocks, those for excavating the deep foundation pit with hard surrounding rocks were significantly reduced when adopting a conventional support scheme of retaining structures and inner supports. Therefore, when excavating deep foundation pits with hard surrounding rocks, appropriately reducing inner supports could be beneficial for lowering costs, construction duration, and so on.

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K.L.; writing—review and editing, F.G.; funding acquisition, Z.M. and P.G. All authors have read and agreed to the published version of the manuscript.

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