Research Article

Systematic Analysis Method for the Unusual Large Displacement in the Excavations in Soft Soil Area

Xiaodong Ni,1 Jiangfa Lu,1 Chen Wang,1 Songxian Huang,1 and Donghua Tang2

1Hohai University, Nanjing 210000, Jiangsu, China
2Nanjing Metro Co., Ltd., Nanjing 210098, Jiangsu, China

Correspondence should be addressed to Xiaodong Ni; lulingnxd@126.com

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Based on a subway station excavation construction project in the soft soil area in Nanjing, an informationized monitoring scheme was conducted during the construction of excavation, and the theories of displacement prediction were introduced into the scheme for the evaluation of the horizontal displacement of the retaining structure and the settlement of the surroundings around the excavation. Based on these theories and the monitoring data, a numerical simulation based on the commercial FEM numerical analysis software, Midas GTS NX, was conducted to simulate the whole construction process. To handle the large displacement of the retaining structure observed during the construction, the actual soil layers’ status discovered by excavating, which can reflect the physical characteristics of the soil, the construction condition, and the variation trend of the monitoring data, was used in the back analysis of the factors that induced the large deformation of the retaining structure, and the analysis result was fed back to the countermeasurement organization and design such as erecting temporary steel strut. The effectiveness of these measurements in the aspect of the reduction of the deformation rate was verified, which can provide reference to the design and construction of a similar project in soft soil area.

1. Introduction

In recent years, to make full use of underground spaces, the scale of excavations was designed to be deeper and deeper and larger and larger, there are many difficulties and risks in the construction of these excavations, and the demand for the deformation control is more and more strict; thus, it is important to informationize the monitoring scheme of these excavations and predict the deformation of the retaining structure and the surrounding soil in these excavations. To make the informationization of the monitoring scheme come true, the present theories of ground settlement and numerical simulation should be used to predict the deformation of the retaining structure and surroundings around excavations, and the construction scheme should be modified based on the feedback from the real-time monitoring; in other words, the informationization of the excavation scheme requires the combination of present theories, numerical analysis, and real-time feedback from the construction field [1–8]. And many studies have been conducted on the application and applicability of numerical analysis software on actual project [9–12] and the studies of the mechanical responses of rock and soil under different work conditions [13–15].

Many studies on ground settlement and the deformation of retaining structure have been carried out by previous scholars. Peck proposed an empirical formula to estimate the ground settlement by analyzing a great amount of excavation construction projects in Chicago, Oslo, and many other areas and summarized the horizontal displacement of the retaining structure and ground settlement data in different geological conditions and different retaining structures. In this formula, the soils were divided into three types according to their hardness; the softer the soil is, the larger the settlement and the area influenced will be [16]. Based on the monitoring data from seven excavation construction projects, O’Rourke analyzed the ratio relationship of the horizontal displacement of the retaining structure to the
vertical displacement of the surface ground and the influences of different measurements in the whole construction process such as dewatering measurement which was conducted before the excavation construction, the erection of steel strut which was conducted during the construction, and the construction of the overground structure after the excavation construction on the deformation of the soil [17]. By analyzing the monitoring data in thirty-five open cut excavation projects in the floodplain soft soil area in Nanjing, Wan et al. found that the maximum horizontal displacement of the retaining structure was 0.05%–0.69% of the excavating depth, which is far over the control value of the adopted code [18]. Zhang and Li and Chen et al. studied the retaining structure failure accident of the Xianghu Station which is a metro station on Hangzhou metro line 1 and found that there were several human errors made by designers, constructors, and the third party monitoring company of the project that caused the appearance of the tragedy: the retaining structure that was not designed to embed in the hard plastic soil layer was one of the reasons that induced the stability failure of the retaining structure; the soil was over excavated and the soil at the bottom of the excavation was not effectively strengthened according to the designers’ requirements, which caused soil disturbance at the bottom of the excavation and finally induced serious ground collapse; the third party monitoring company did not issue a warning to the constructors and the designers, while the maximum settlement had reached 316 mm and did not conduct a real-time evaluation of the stability of the excavation [19, 20].

Based on the previous studies, in this paper, the regularity of the variation of deformation of the retaining structure and the ground settlement of surroundings around excavation in the floodplain soft soil area in Nanjing was analyzed, a strength reduction numerical model of the excavation was established to evaluate the deformation and displacement, and both the numerical analysis result and the monitoring data were used to verify the applicability of the present theories of the ground settlement and deformation. These works can not only provide effective feedback to the designers and constructors of the excavation in time but also provide important reference to the design, construction, and study of similar projects.

2. Theories of the Deformation of Excavations

2.1. Peck’s Empirical Theory. Based on a great amount of monitoring data from different excavation construction projects, Peck proposed empirical formulas of the ground settlement of surrounding around excavations [16]; in Peck’s theory, according to the difference of the retaining structure and the geological conditions, the ground settlement of the surroundings was summed up to three types, which were corresponded to the settlement range of area I, area II, and area III given in Figure 1. The three settlement ranges can present three conditions, respectively: firstly, the soil was sand or stiff clay; secondly, the hardness of the soil was soft to very soft; thirdly, a thick soft clay layer was under the excavating surface, while the stability coefficient \( N_{cb} \) is larger than the critical stability coefficient for the upheaval of the bottom soil of the excavations, \( N_{cb} \). Peck’s empirical formula was widely used before when the retaining structure was simple and the requirement of the deformation control was not too strict; however, as the retaining system is being more and more complex and the requirement of surrounding environment protection is being more and more strict, Peck’s empirical formula is no longer suitable for those large excavation construction projects.

To make Peck’s empirical formula suitable for more kinds of retaining structure and more geological conditions, many scholars had modified the formula; among them, the formula modified with the coefficient \( K \) can be used in the real project and have been widely used; the modified formula is as follows:

\[
\theta = 10 \times K \times \mu \times H \text{ (mm)}. \tag{1}
\]

In this formula, \( \theta \) represents the ground settlement; \( K \) represents the modified coefficient, for diaphragm wall; \( K \) is equal to 0.3, for soldier pile wall; \( K \) is equal to 0.7, for steel sheet pile wall; \( K \) is equal to 1.0; \( \mu \) represents the ratio of the ground settlement to the excavating depth; \( H \) represents the excavating depth.

2.2. Modified Stratum Compensation Theory. To analyze the deformation of the retaining structure and the surroundings around excavation, Yang et al. [21] and Yin [22] modified the stratum compensation theory based on the experience of a great amount of excavation construction; they supposed that the deformation of the retaining structure and the surroundings around excavation consisted of triangular region and concave region as shown in Figure 2. In Figure 2, \( \delta_0 \) represents the maximum horizontal deformation of the top of the retaining wall, \( H_r \) represents the excavation’s depth, and \( H_r \) represents the distance between the fixed point and the bottom of the excavation. Suppose that the fixed point is the intersection of the extended line of the line connecting the maximum displacement point of the retaining wall and the bottom of the retaining wall after the occurrence of the displacement of the retaining wall and the extended line of the retaining wall before the occurrence of the displacement of the retaining wall. The distance between the intersection and the bottom of the retaining wall represents the influence depth of the soil, and the distance between the intersection and the top of the retaining wall represents the depth of the fixed point.

2.2.1. Calculation of the Triangular Region. Figure 3 illustrates the calculation method of the triangular displacement field of the retaining wall and surroundings around excavation. Equation (2) shows the relationship between depth and horizontal displacement in triangular displacement region of the retaining structure.
\[ \delta_{w1} = f_{w1}(z) = \delta_0 \cdot \left(1 - \frac{z}{H_{total}}\right). \]  
\[ \delta_{h1}(x,z) = \delta_{v1}(x,z) = \delta_0 \cdot \left(1 - \frac{\sqrt{x^2 + z^2}}{H_{total}}\right). \]  

In the same way, the vertical displacement \( \delta_v \) and horizontal displacement \( \delta_h \) of the surroundings around excavation can be calculated with

\[ \delta_{h1}(x,z)|_{z=0} = \delta_{v1}(x,z)|_{z=0} = \delta_0 \cdot \left(1 - \frac{x}{H_{total}}\right). \]  

2.2.2. Calculation of the Concave Region. In the stratum compensation theory proposed by Peck, it was supposed that the total volume of the soil behind the retaining structure would not change, so that the envelope area of the displacement curve of the surroundings around the excavation is equal to the envelope area of the displacement curve of the retaining structure; their relationship is illustrated in Figure 4.

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**Figure 1:** Peck's empirical curves.

**Figure 2:** Illustration of the displacement field of the excavation.

**Figure 3:** Illustration of the calculation method of the triangular displacement region.
Supposing that the equation of the displacement curve of the concave region was $\delta w = f w(x,z)$, Yang et al. introduced the modified coefficient of volumetric shrinkage $\alpha$ into the equation to modify the stratum compensation theory, and the circular arc sliding method was changed to elliptical arc sliding method. The vertical displacement can be calculated with these equations, the total displacement of the surface of the surroundings around excavation can be calculated with

$$\delta_v = \delta_v(x,z)|_{z=0} = \frac{1}{\alpha} \cdot f w_2(\alpha x), \quad (9)$$

and with these equations, the total displacement of the surface of the surroundings around excavation can be calculated with

$$\delta_h = \delta_h(x,z)|_{z=0} = \delta_0 \cdot \left(1 - \frac{x}{H_{total}}\right) + \frac{1}{\alpha} \cdot f w_2(\alpha x), \quad (10)$$

$$\delta_h = \delta_h(x,z)|_{z=0} = \delta_0 \cdot \left(1 - \frac{x}{H_{total}}\right). \quad (11)$$

### 3. A Case Study of Informationized Monitoring Scheme

#### 3.1. Engineering Profile

The project studied in this paper was the phase I project of deep excavation of a subway station in Nanjing; this excavation is located in the floodplain area of Yangtze River which is mainly composed of soft soil area. The subway station is an underground double-deck station with island platform, and the total length of the excavation of the phase I project is 140 m. The width of the typical section of the excavation is 20.1 m, the width of the partition wall on the excavation side is 20.1 m, the length and length of the shield shaft on another side are 24.5 m and 14 m, respectively, the ratio of the length to the width of the typical section is 6.3:1, the depth of the excavation ranges from 15.5 m to 16.2 m, the excavation was constructed from south to north and from top to bottom, and the erection of the strut would be conducted with the excavating process. There are a lot of building and municipal pipelines in the surroundings around the excavation.

#### 3.2. Geological Condition

In the aspect of topography, the subway station is located on a col landform area between two terraces; in this area, the elevation ranges from 12.23 m to 16.30 m, the terrain is slightly fluctuating, and the elevation...
of this area is decreasing from north to south. Figure 5 illustrates the soil layer been traversed by the subway station. The bottom of the subway station mainly lied on the clayey silt layer ②-3c2-3, the silty clay layer ②-3b2-3, and the silty clay layer ②-1b1-2, the soil layer at the bottom of the subway station fluctuates, and the physical parameters of these soils are very different, which means that the foundation of the subway station is an uneven foundation. As shown in Figure 5 and Table 1, the physical parameters of the miscellaneous fill soil and those soft soil are poor, and some of these soft soils contain a lot of humus, some of these soft soils were of soft plastic state, others were of flow plastic state, and all of these soft soils have high compressibility and poor strength and are easy to be disturbed which would induce uneven settlement and the failure of the retaining wall. Due to the poor geological condition, the construction in these soil layers was difficult.

3.3. Informationized Monitoring Scheme. As the construction period of the excavation of subway station is long and the strength of the soil around the excavation is poor while there are many buildings, main road and underground structures need to be protected during the construction of the excavation; to handle these adverse factors and make sure of the safety of the construction of the excavation and surroundings around the excavation, the monitoring scheme should be informationized to fulfill these requirements.

The process of the informationized monitoring scheme is shown in Figure 6. In this scheme, the trend of the deformation of the excavation would be analyzed based on the real-time monitoring data from the construction field, then these analysis results would be used in the back analysis based on numerical analysis method, and the back analysis result would be used to optimize the construction method.

There are several monitoring objects in the construction monitoring stage, such as the horizontal displacement of the retaining structure, the settlement of the surroundings around the excavation, the vertical and horizontal displacement of the top of the retaining pile, the variation of the water level, and the variation of the axial force of the reinforced concrete strut and steel strut.

In this paper, to explain the process of the informationized monitoring, the inclination measurement of the retaining pile and the settlement measurement of the surface ground were used in the following analysis.

3.3.1. Analysis of the Horizontal Displacement of the Retaining Piles. In the phase I project of the subway station, the form of the horizontal displacement of both the retaining pile and the soil is like a bow, which means that the displacement of the top and the bottom is small, while the displacement at the middle is large, and this form of displacement is basically in agreement with the theory of combined displacement proposed by Clough and Tsui [23]. To study the variation of displacement of the retaining pile during the whole process of the excavation construction, the monitoring data in different construction stage of the monitoring points ZQT32 and ZQT08 which have the largest accumulated displacement among all the monitoring points were used in the following analysis. Figure 7 shows the displacement curves of both monitoring points.

As shown in Figure 7, the retaining pile was constantly deforming toward the inside of the excavation along with the increasing of the pressure difference between the outside and inside of the excavation which mainly consists of the soil pressure difference caused by excavating and the dewatering inside the excavation. And the horizontal displacement of the retaining piles was increased with the excavating depth.

According to the displacement curves of ZQT08 and ZQT32, when the excavating depth came to 6 m, the maximum horizontal displacement toward the inside of the excavation of both monitoring points occurred near the excavating surface, and the maximum horizontal displacement at this stage was close to 40 mm, the control value of the adopted code. When the excavating depth came to 10 m, the horizontal displacement of both piles increased by 10 mm–15 mm. When the excavating depth came to 13 m, because the fourth steel strut (from top to bottom) was not erected in time, the maximum horizontal displacement of the piles increased to 160 mm which was far over the control value mentioned above. When the excavating came to the bottom line of the excavation, although the footwall had been poured and constructed on time, the horizontal displacement of the two monitoring points still increased by 20 mm–30 mm, and the depth of the maximum horizontal displacement point tended to be stable. The construction of the main structure of the subway station began immediately after the excavating stage finished; during the finished excavating stage to the finished construction of the main structure of the subway station, due to the redistribution of the stress of the retaining piles induced by the removing of the steel strut, small displacement of the retaining piles was monitored. Although ZQT08 and ZQT32 were on the same monitoring and their displacement was very close not only to the displacement curve but also to the value of the displacement, there were some differences between them. The displacement toward the inside of the excavation of the bottom of ZQT08 was over 80 mm and larger than that of the ZQT32, which was a typical “kicking displacement,” while the displacement toward the inside of the excavation of the bottom of ZQT32 was small and tended to be stable since the excavation depth came to 6 m. As the two retaining piles were on the same section along the length direction of the excavation, which means that the construction schedules of them were almost the same so that it could be deducted that the soil distribution and the soil strength of both sides of the excavation were different, and the soil strength of the bottom of the retaining pile ZQT32 was too poor to completely restrict the displacement of the bottom of the retaining pile.

The horizontal displacement of both of the two monitoring points was very large, and the final maximum horizontal displacement of both was increased to 430% of the control value of the adopted code, especially when the excavating depth came to 13 m and the horizontal displacement came to 160 mm, which was very unusual. By analyzing the geological conditions and the construction process of this section, the reason for the unusual
displacement was as follows: on the one hand, the soil layer 6 m–13 m below the bottom of the excavation mainly consisted of muddy silty clay and the strength of this soil was poor, which means that soft clay creep would probably occur in this soil layer when the soil has been disturbed by the construction of the excavation, and this reason can also be deducted from the deformation lag of the retaining structure; on the other hand, overbreak was found in the construction of this section and both depth and area of the overbreak were large, while the steel struts were not erected on time, the excavation has been exposed too long, and all these factors finally induced the unusual large displacement.

The relationship between the depth of the maximum horizontal displacement point and the excavating depth is

Table 1: Physical parameters of the soil.

| Soil layer number | Soil type                        | γ (kN/m²) | E (MPa) | c (kPa) | φ(°) | K (10⁻⁵ m/s) |
|-------------------|---------------------------------|-----------|---------|---------|------|-------------|
| ①-1               | Miscellaneous fill              | 17.5      | 4.8     | 18.5    | 22.6 | 0.5         |
| ①-1b2-3          | Silty clay                      | 18.7      | 5.94    | 22.0    | 16.6 | 0.3         |
| ①-1c3            | Clayey silt                     | 19.2      | 8.55    | 12.0    | 20.0 | 0.003       |
| ①-2b3-4          | Silty clay                      | 17.5      | 3.85    | 11.0    | 12.1 | 0.035       |
| ①-3c2-3          | Clay silt                       | 19.0      | 8.13    | 13.0    | 20.0 | 0.003       |
| ①-3b2-3          | Silty clay                      | 19.4      | 6.27    | 21.0    | 21.0 | 0.4         |
| ①-1b1-2          | Silty clay                      | 19.4      | 7.59    | 35.0    | 20.1 | 3           |
| ①-2b2            | Silty clay                      | 19.0      | 6.84    | 32.0    | 19.6 | 0.4         |
| ①-3b1-2          | Silty clay                      | 19.5      | 8.69    | 52.0    | 22.1 | 2           |
| ①-3d2            | Silty sand with silt            | 20.1      | 7.09    | 18.0    | 16.8 | 2           |
| ①-4b1-2          | Silty clay                      | 19.3      | 7.76    | 32.0    | 17.8 | —           |
| ①-4c1-2          | Gravel-bearing silty clay       | 19.5      | —       | 28.0    | 30.0 | —           |
| K1g-2             | Strongly weathered sandstone and mudstone | 22.0 | — | 41.0 | 30.0 | — |
| K1g-3n            | Moderately weathered sandy mudstone | 25.0 | — | 260.0 | 35.0 | — |
| K1g-3s            | Moderately weathered argillaceous sandstone | 24.5 | — | 400.0 | 38.0 | — |
| K1g-3s’           | Moderately weathered argillaceous sandstone (broken) | 24.5 | — | 200.0 | 35.0 | — |
| K1g-3s-1          | Moderately weathered argillaceous sandstone with soft interlayer | 23.0 | — | 150.0 | 32.0 | — |

γ = bulk density, E = elasticity modulus, c = cohesive force, φ = internal friction angle, and K = hydraulic conductivity.

Figure 5: Geological profile of the excavation.

Figure 6: Illustration of the process of informationized monitoring scheme.
shown in Figure 8. The depth of the maximum horizontal displacement points was range from $H-5$ to $H+2.5$ ($H$ means excavating depth) which can meet the requirement of the adopted code of Nanjing that the depth should range from $H-8.8$ to $H+4.5$. The depth of the maximum horizontal displacement point was increased with the increase of the excavating depth. After the finish of the end of the excavating stage, the depth of maximum horizontal displacement points of all monitoring points ranged from 10 m to 13 m, and all these points were between the third steel strut layer and the fourth steel layer.

3.3.2. Analysis of the Vertical Displacement of the Surface Ground. The surface ground settlement curves of excavation construction can be summed up to triangular curve and groove curve [24–29]. The triangular curve always occurs in those excavations that have been excavated without the erection of strut, while the groove curve always occurs in those excavations that have been excavated with the erection of the strut system. Figure 9 shows 8 sets of monitoring data from 8 monitoring sections, respectively. It was shown that the settlement of both ends of those sections was small, while the settlement of the middle of those sections was large, which was exactly the characteristic of the groove curve. The impact zones in the surroundings around excavation of the excavation construction are also shown in Figure 9. The area that has been covered by the settlement curve of this excavation is wider than those that have been covered by Hsieh’s curve, Ou’s curve [30], and settlement prediction curve of Shanghai area [31]. Soft soil is also widely distributed in Shanghai area; although the characteristics of soft soil in Nanjing and Shanghai are very close, there are some differences between the physical parameters of the soft soil in the two areas. Compared to the soft soil distributed in Shanghai area, the soft soil in Nanjing area has a little larger liquid and plastic limit water content and void ratio and a little smaller compressive coefficient and inner friction angle. According to the monitoring level of the excavation which can be determined by the adopted code, the width of the monitoring area of this excavation ranges from $2H$ to $2.5H$ ($H$ represents the excavating depth), while the width of the impact zone been observed was far over $2H$, which means that the influence of the excavation construction on the surroundings around excavation was underestimated in the design of the excavation.

The maximum surface ground settlement was increasing with the increase of the horizontal displacement of the retaining structure; the ratio of the maximum surface ground settlement $\delta_{sm}$ to maximum horizontal displacement $\delta_{hm}$ ranges from 0.2 to 1.6, which is in agreement with this ratio in Nanjing area summed up by Xu et al. which ranges from 0.09 to 0.35 [32]. Meanwhile, the ratio ranges from 0.2 to 1.6 under every excavating stage, and it can be supposed that the relationship between the maximum surface ground settlement and maximum displacement of...
retaining piles was stable. So that the displacement in the later construction stage can be predicted by the relationship concluded in the early construction stage. And in this excavation, the settlement curves of the surroundings around the excavation were similar to the displacement curves of the retaining structure, which can verify the applicability of the modified stratum compensation method mentioned previously; thus, it was reasonable to apply the stratum compensation theory to deduct the surface ground settlement curve from the horizontal displacement curve of the retaining structure. When the unusual displacement is recorded in the monitoring data, the validity of the monitoring data can be verified by the comparison of the horizontal displacement of the retaining structure and the surface ground settlement, while the displacement of the retaining structure in the following construction can be predicted by numerical simulation and the formula of the surface ground settlement can be deducted through the curve fitting of the horizontal displacement curve of the retaining structure.

There were 117 surface ground settlement monitoring points set in the monitoring scheme, 95 effective monitoring points remained after screening out of those monitoring points with unusual monitoring data or have been destroyed.
during the construction process. Figure 10 illustrates the probability distribution of the final vertical displacement of the 95 monitoring points. The number of monitoring points of settlement in the range of 10–20 mm accounts for 21.05% of the total monitoring points, which is the section with the highest proportion; 22.11% of the 95 monitoring points have a maximum settlement over 100 mm, including 4 monitoring points with the maximum settlement over 180 mm; the largest final settlement of them came to 255.1 mm.

In the early stage of the excavation construction, as the excavating depth was small, the displacement of the monitoring points was still kept at a small level. Upheaval even is observed in some monitoring points due to the great support force from the reinforced concrete strut. With the progress of the construction and the increase of excavation depth, the settlement was also increasing, but the increase rate of the settlement was smooth in this stage and gradually approaching the control value. Then, when the construction came to the depth between the second steel strut and the third steel strut, the increase rate of the settlement increased and the maximum settlement occurred on the monitoring points 12 m away from the edge of the excavation; the accumulated settlement of part of these points reached 200 mm. With the end of the excavating stage and the construction of the footwall, the variation of surface ground settlement tends to be smooth.

3.4. Numerical Simulation of the Whole Construction Process. To alleviate the large displacement of the retaining structure, a numerical simulation was conducted based on the geotechnical exploration report and the blueprint of the excavation through the commercial geotechnical numerical analysis software Midas GTS NX. The analysis result would be used to compare with the control value of the adopted code and the monitoring data from the construction field to verify the appropriateness of the design and analyze the characteristic of the displacement occurring in the construction of this excavation.

The influence of dewatering and spatial effect was also taken into consideration of the simulation; Modified Mohr-Coulomb (MMC) constitutive model was applied to the soil material of the three-dimensional numerical model which needed 11 physical parameters of the soil; among the 11 parameters, the secant stiffness in triaxial test represented by $E_{50}^{\text{ref}}$, the tangential stiffness primary oedometer test loading represented by $E_{\text{ood}}^{\text{ref}}$, and the Elastic Modulus at unloading represented by $E_{\text{ur}}^{\text{ref}}$ are the key parameters and have a great influence on the calculation result. The determination of the three parameters depends on the empirical formula; $E_{50}^{\text{ref}}$ ranges from 0.9 times of $E_1^{1.2}$ to 1 time of $E_1^{1.2}$ which was usually provided by geotechnical exploration report, for the sandy soil, $E_{50}^{\text{ref}}$ is equal to $E_{\text{ood}}^{\text{ref}}$, $E_{\text{ur}}^{\text{ref}}$ ranges from 3 times to 5 times of $E_{50}^{\text{ref}}$, when it comes to clayey soil, $E_{50}^{\text{ref}}$ ranges from 3 times to 5 times of $E_{\text{ood}}^{\text{ref}}$, and $E_{\text{ur}}^{\text{ref}}$ range from 4 times to 6 times of $E_{50}^{\text{ref}}$. As shown in Figure 11, the middle of the long side of the excavation, and the settlement curve from the edge of the excavation to the edge of the numerical model was very close to the groove curve mentioned in Section 3.3.2. The maximum settlement was 39.2 mm which was very close to 40 mm, the control value.

The horizontal displacement of the retaining structure was large at the middle and small at both ends which had reflected the spatial effect. The maximum horizontal displacement of the retaining structure occurred at the middle of the excavation in the length direction, and the value of it was 27.8 mm, which is under 40 mm, the control value mentioned previously. The maximum settlement also occurred in the middle of the long side of the excavation, and the settlement curve from the edge of the excavation to the edge of the numerical model was very close to the manual curve mentioned in 3.3.2. The maximum settlement was 39.2 mm which was very close to 40 mm, the control value.

The horizontal displacement of the retaining structure of the excavation was an important indicator of the evaluation of the safety of the excavation, and the surface ground settlement was an important indicator of the evaluation of the safety of the surroundings around the excavation. The analysis result shows that the design of the excavation can meet the requirement of the adopted code; the form of the displacement curve in the numerical simulation was also similar to that of the monitoring data; however, the value of the monitoring data was far over that of the analysis result, which means that, to gain the simulation result close to the reality, the parameters of the numerical model should be modified by back analysis based on the monitoring data.

3.5. Back Analysis. Previous studies showed that the main reasons for the unusual large displacement were the deterioration of the strength of the soil during the excavation construction, the stiffness reduction of the retaining structure, and the local overbreak in the construction process; the back analysis focusing on these reasons was conducted based on monitoring data.

According to the construction condition and the surrounding condition, it was found that the filling coefficient of the retaining piles was high during the construction
period, many retaining piles intruded into the soil over the design boundary, and the elimination of these portions of the retaining piles and the erection of the strut took too much time, which induced the reduction of the strength provided by the retaining piles. Based on the experience from similar projects, the strength of the retaining piles was decreased to 70%–90% of the design strength.

During the excavation construction, heavy precipitation occurred in Nanjing area in July 2020 to handle the waterlogging of the excavation; constructors were pumping out the water from the excavation during the heavy precipitation period. The retaining piles were exposed in the following construction in early August, the exposure shows that the piles did not lock each other very well, and a small volume of mud and water even can traverse some intervals between the piles. It is shown that, during the long time of water pumping, some fine soils were also dragged out by water, which induced the large margin reduction of the soil strength. It was supposed that the soil strength was decreased to 20%–40%.

Simulating with different strength parameters that have been deteriorated again and again, it was found that the analysis result was close to the monitoring data when the strength of the retaining pile decreased to 75% of the design strength and the strength parameters of the soils decreased to 30% of the parameters before. And some modifications of the simulation were conducted based on the construction conditions: the preaxial force of the steel strut was increased by 250 kN; the overbreak of the fourth construction section and fifth construction section was set to 2 m. The comparison between the final simulation result and monitoring data of ZQT08 in different construction stage is shown in Figure 12(a). It was shown that the displacement curve of numerical simulation and monitoring data were very close. The analysis result from the same strength parameters deteriorated the numerical model of ZQT01, which was in the

| Settlement (mm) | Ratio of points in each settlement range to total points (%) |
|----------------|----------------------------------------------------------|
| -10~0          | 0.47%                                                     |
| 0~10           | 12.63%                                                    |
| 10~20          | 8.42%                                                     |
| 20~30          | 5.26%                                                     |
| 30~40          | 6.32%                                                     |
| 40~50          | 3.16%                                                     |
| 50~60          | 5.26%                                                     |
| 60~70          | 2.11%                                                     |
| 70~80          | 3.16%                                                     |
| 80~90          | 2.11%                                                     |
| 90~100         | 2.11%                                                     |
| 100~110        | 2.11%                                                     |
| 110~120        | 2.11%                                                     |
| 120~130        | 3.16%                                                     |
| 130~140        | 1.05%                                                     |
| 140~150        | 4.21%                                                     |
| 150~160        | 2.11%                                                     |
| 160~170        | 2.11%                                                     |
| 170~180        | 2.11%                                                     |
| Above 180      | 4.21%                                                     |

Figure 10: Probability distribution of the surface ground settlement.

Figure 11: Numerical model of the excavation.
end shaft section, shown in Figure 12(b), and it was also very close to the monitoring data. To some extent, the two comparisons between the numerical analysis result and the monitoring data had verified the applicability of the informationized monitoring scheme in this paper which evaluated the stability of the excavation dynamically based on numerical analysis and monitoring data.

To verify whether the displacement of the bottom of the ZQT08 can meet the requirement of the adopted code under the work condition based on the blueprint and the geotechnical exploration report, a numerical simulation was conducted and the displacement of the bottom of the retaining pile in the simulation result is shown in Figure 13. Although the displacement of the bottom of the retaining pile was smaller than the control value in the adopted code, the displacement of the bottom of the retaining pile was relatively large. In the monitoring data, the displacement was 60% of the maximum displacement of the retaining pile, the simulation result showed that the displacement was 51% of the maximum displacement, and displacement of the bottom of the retaining pile can also be found in Figure 12, which means that such displacement in simulation was in accordance with the actual situation and the analysis result would be considerably unsafe when the dewatering measurement was taken into consideration.

### 3.6. Feedback to the Construction Organization Design

Based on the numerical back analysis, it was verified that the reasons for the unusual large displacement of the excavation are the deterioration of the physical-mechanical parameters of the soil and the retaining structure and local overbreak in the construction process. The analysis result was fed back to the designers and constructors and referenced for the modification of the construction scheme. Some of the modifications were as follows.

#### 3.6.1. Temporary Steel Strut Erection

The excavating work was stopped when the constructor received the warning notification, and the emergency strength measurements for the excavation were conducted. Steel strut was erected near the depth of the maximum horizontal displacement point which was between the third steel strut and fourth steel strut; thus, the vertical distance between the steel strut was decreased to 1.5 m from 3 m. Consider that relatively large displacement has been observed in the following construction section while the thickness of the muddy silty clay with poor physical-mechanical parameters was thicker so that the vertical distance between the third steel strut and fourth steel strut of the following construction stage was also modified to 1.5 m from 3 m. The prestressing of steel...
strut should also be made sure of during the erection of the steel strut.

3.6.2. Overbreak Prohibition. To ensure the safety of the following construction, the adopted code should be strictly obeyed in the following construction, the excavating work should be conducted with small-sized excavator or long reach excavator after the erection of the strut and strictly obey the construction scheme, and overbreak should not occur in the following construction.

3.6.3. Optimization of the Division of the Construction Section. The optimization of the division of the construction section was aimed at shortening the construction time of the footwall of every construction section which can shorten the exposure time of the bottom of the excavation and reduce the horizontal displacement of the retaining structure. The construction section was divided according to the position of the construction joint in the blueprint, and the length of the fourth and fifth construction sections was 25.3 m and 26 m which were relatively longer. In the optimization scheme, the two construction sections were divided into three construction sections.

Eighty percent of the excavating work had been completed when the unusual large displacement of the retaining structure was observed in the monitoring data, but no evidence could be found in the monitoring data, which can deduct the appearance of the unusual large displacement, taking the excavating schedule and the characteristic of the soft soil into consideration. It was supposed that the reason for the sudden appearance of the unusual large displacement was the rheological properties of the soft soil which means that progressive deformation of the soil would appear once the soil has been disturbed or the stress status of the soil has been changed. In other words, the displacement of the retaining structure was induced by the deformation and displacement of the soil, while the progressive deformation of the soil was induced by the excavating construction which would disturb the soil and change the stress status of the soil; thus, the displacement of the retaining structure was lag behind the excavating construction. To solve this problem, the steel strut should be erected in time and the emergency measurements should be strictly conducted in the time window.

After the modifications above were conducted, the increase rate of the displacement of the retaining structure finally decreased to 1 mm/d and did not increase over the control value anymore in the following construction stages. As shown in Figure 14, the numerical analysis result based on the physical parameters deteriorated numerical model showed that the strength measurements can restrict the displacement of the retaining structure effectively, as the displacement of the retaining structure in the simulation...
under the work condition with the conducting of strength measurements was smaller than the one under the work condition without the conducting of strength measurements. The maximum horizontal displacement of ZQT08 under the work condition with the conducting of strength measurements was 190.4 mm, and the value would increase to 221.9 mm under the work condition without the conducting of strength measurements. In other words, according to the numerical analysis result, with the conducting of the strength measurements, the displacement of the retaining structure would be decreased by 16.5%. And the monitoring result of the following construction showed that the maximum horizontal displacement of ZQT08 was 198.1 mm, and the displacement curve in the monitoring data was very close to that of the analysis result, which reflected the applicability of the numerical model. If the large displacement of the retaining structure was not predicted by the analysis and the retaining structure strengthen measurements were not conducted, local failure of the retaining structure would have occurred. The comparison of the monitoring data of the retaining structure in the following construction and the numerical analysis result had shown the importance and necessity of the application of the informationized monitoring scheme in the construction process.

4. Conclusion

(1) The informationized monitoring scheme of the excavation construction is one of the critical measurements to guarantee the safety of the excavation construction which can provide early warning for the risks occurring in the construction process in time and provide key information for the elimination of the risks. So that the informationized monitoring scheme should be used in the prediction of the displacement of the excavation and the surroundings around the excavation and the verification and modification of the design of the excavation. To those excavations that have made unusual displacement or other risks, it is suggested that the monitoring data of them should be used in the back analysis of the strength parameters concerned, and verification of
the applicability of the design and the construction scheme based on the comparison among the back analysis, monitoring data, original design, and geotechnical exploration report should be conducted.

(2) Peck’s empirical formula and the modified stratum compensation method were deducted from the analysis of a great amount of excavation construction project, and the applicability of both of the two methods had been verified by many other projects; however, the two methods are not completely suitable for all excavation construction project, and both of the methods need to be modified; to take the unique geotechnical conditions of the excavation into consideration and calculation, mutual geotechnical numerical analysis software would be a better choice, and the analysis result would be closer to the monitoring data compared to the result of the two methods.

(3) This paper presents a case study of a deep subway station excavation construction project in soft soil area in Nanjing. The horizontal displacement and surface ground settlement in this project were far over the control value of the adopted code, the horizontal displacement of the retaining pile ranged from 0.35% to 1.5% of the excavating depth, the maximum surface ground settlement point was about 12 m away from the edge of the excavation, which is close to the depth of the maximum horizontal displacement points of the retaining structure, and the form of the settlement curve is in agreement with the settling trough in Peck’s theory.

(4) The total displacement induced by factors of construction cannot be seen as the summation of the displacement induced by each single factor, as there is a strong relationship between these factors, for example, because the concrete intruded into the soils out of the design boundary during the construction of the retaining piles, this concrete would influence the following construction and should be eliminated, while the elimination of this concrete would reduce the strength of the piles and take a lot of time, which would postpone the erection of the strut and lengthen the exposure time of the excavation. Thus, it is necessary to improve the quality of the excavation construction in soft soil area, and the critical point in the construction process should be strictly controlled.

(5) The local unusual large displacement of retaining structure would be induced by the deterioration of the strength of the soil during the excavation construction, the stiffness reduction of the retaining structure, and the local overbreak in the construction process, which would also have an adverse influence on the stability of the excavation. According to the feedback from the construction field and monitoring data, to a great extent, the back numerical analysis had predicted the displacement of the excavation very well, which had verified the applicability of the informationized monitoring scheme that evaluated the stability of the excavation dynamically based on numerical back analysis and monitoring data. Back analysis can not only reassess the safety of the excavation but also predict the variation of the displacement of the excavation in the following construction, which can be the reference to the scheme of the displacement restriction.

Data Availability
The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest
The authors declare that they have no conflicts of interest.

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