Analysis of Lining Damage and Verification of Control Measures Based on Two-Stage Analysis Method

Xiang Ye[1,*], Fang He[2], Qingyuan Tan[3] , Jinchang Wang[4] and Yazhen Sun[5]

[1]Hangzhou Shangcheng District Urban Construction Development Group Co. Ltd, Hangzhou 310027, China
[2]School of Civil Engineering, Shenyang Jianzhu University, Shenyang 110168, China
[3]Zhejiang design institute of water conservancy & hydroelectirc power CO, LTD, Zhejiang, Hangzhou, 310027, China
[4]Institute of Transportation Engineering, Zhejiang University, Hangzhou 310027, China
[5]School of Transportation and Geomatics Engineering, Shenyang Jianzhu University, Shenyang 110168, China

*Corresponding author's e-mail: 175190453@qq.com

Abstract. In order to determine the structural changes of the tunnel before and after the construction of the adjacent project, and to control the diseases within the specified range, based on the deep foundation pit project around the Jiubao Metro in Hangzhou, the maximum displacement results of the typical cross-section tunnel before and after deep foundation pit excavation and reinforcement calculated in PLAXIS were extracted, and the joint bolts, steel mesh, etc. were considered in the model established by DIANA. The displacement superposition of the refined single ring lining model was carried out, Analyze the damage of tunnel lining structure through two-stage calculations and verify the effectiveness of reinforcement measures. The research results indicate that the structural damage of the tunnel is more severe after deep excavation, and the reinforcement measures have a good control effect on the deformation of the tunnel. The two-stage refined model can quantify the lateral structural damage of the pipe segment and obtain the crack width to study the specific development trend of the pipe.

Keywords: two-stage analysis method; Displacement superposition method; Segment damage; Slant support reinforcement.

1 Introduction

In the operation of urban rail transit, with the increasing number of adjacent projects, there are varying degrees of additional internal forces and displacements generated in the structures of nearby subway stations and section tunnels. As the service life increases, the lining structure suffers from diseases such as water seepage and cracking. Therefore, it is particularly important to determine the structural changes of the tunnel before and after the construction of the adjacent project, and to control the diseases

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within the specified range.

The current numerical calculation methods mainly include load structure method and stratum structure method. Pan Hao et al [1] has shown that adopting methods such as bottom reinforcement, layered and block excavation, and measures to reduce the vertical distance between pits and tunnels can meet the control range of deformation in existing tunnels. Shi [2] studied the stress and strain of the tunnel segment structure at the joints caused by excavation of the lateral foundation pit. Huang Minxiang [3] found through research that the excavation depth of the foundation pit and the vertical distance between the pipeline and the foundation pit have a significant impact on pipeline deformation. He proposed three reinforcement schemes: MJS soil reinforcement, stainless steel lining reinforcement, and bottom loading. Yu Shuangchi [4] found that temporary support and anti pressure measures inside the tunnel can effectively suppress the deformation of the tunnel structure, and can control its structural deformation below the alarm value of 7.5 mm in this project. Li jun [5] used a flexible combination of geological structure method and displacement superposition method to analyze the impact of excavation of nearby foundation pits on tunnels. Du yiming [6] studied the influence of multiple factors on the horizontal displacement of tunnels outside the pit, including tunnel position, excavation depth of the foundation pit, and maximum horizontal displacement of the retaining structure.

In summary, the establishment of models often overlooks complex geological structures and cannot fully simulate actual situations, which can lead to deviations in the calculation results, which may not reflect the actual situation. Taking the excavation project of the deep foundation pit near Jiubao subway station as an example, based on the geological structure method and refined model, this paper analyzes the impact of lateral deep foundation pit excavation on the tunnel and subway station in the vicinity of the station through two-stage calculation, in order to provide guidance and reference for the management of subway lines in the vicinity of the station during operation.

2 Two-Stage Method

In the two-stage calculation, the displacement results of the interaction between the overall geotechnical state and the tunnel structure were first analyzed using the geological structure method. However, since the overall tunnel structure is simulated using plate elements, the response of the discrete single ring lining structure cannot be obtained. Therefore, based on the calculation results of the geological structure method mentioned above, the displacement results are then applied to the refined pipe segment model in the form of loads, Thus obtaining the refined response results of tunnel lining.

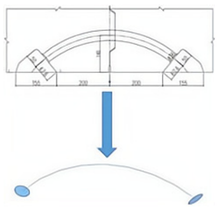
2.1 Refined Single Loop Model

In order to analyze the specific situation, the maximum longitudinal displacement results were selected for the up and down lines of Metro Line 1 after the completion of foundation pit construction using PLAXIS. A total of two cross-sections were subjected to displacement superposition calculation analysis. The section of the up line is 44 meters away from the station, and the section of the down line is 45.6 meters away from the station, which are sections 1 and 2 in sequence. The tunnel segment numbers at the location are the up line 36 ring and the down line 37 ring.

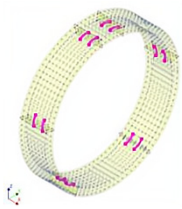
The outer diameter of the tunnel is 6.2 meters, the thickness is 0.35 meters, the ring width is 1.2 meters, and the concrete grade of the pipe segments is C50. The single ring tunnel consists of 6 pipe segments, including a top block F (center angle of 20°), 3 standard blocks (center angle of 67.5°), and 2 adjacent blocks (center angle of 68.75°). Using the tunnel arch as the origin and clockwise as the positive direction, the 180° ~360° direction is near the foundation pit side.

2.2 Two Stage Analysis Based on Displacement Superposition Method

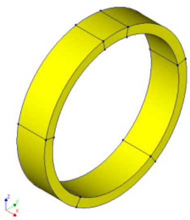
In order to analyze the influence of single ring tunnel structure under different superimposed displacements, DIANA finite element software was used for simulation. The basic model is shown in Figure 1, where the pipe segments are connected by 12 M30 bent bolts, with a bolt length of 0.71 m, a diameter of 0.036 m, a nut diameter of 0.1176 m, a strength grade of 5.8, and a yield stress of 400 MPa. Both the pipe segments and bolts are solid elements. Due to the circumferential and tangential interactions between segments, we use interface elements to simulate the contact interaction between segments. When the pressure is less than or equal to 0, we define contact surface separation. This method can effectively simulate the relative deformation between pipe segments, such as misalignment and seam opening.



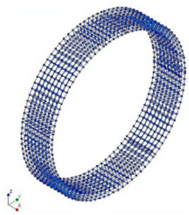
(a) Circumferential joint



(b) Circumferential joint position



(c) Concrete



(d)Rebar

Fig. 1. Single-loop tunnel model

2.3 Total Strain Crack Model

The total strain crack model originates from the modified pressure field theory, and its proposal comes from the theoretical concept of total strain. When the concrete material is not cracked, this constitutive model considers the concrete as a linear elastic material and has the same properties in all directions. Firstly, define a strain increment tensor in the global coordinate axis Δε xyz, Then, regarding the principal strain εi(ε1, ε2, ε3) Principal stress бi(б1, б2, б3) And determine the corresponding direction. If the principal stress in different directions exceeds the predetermined failure criterion, concrete will crack. Once cracks appear, concrete is considered an orthotropic material and it is assumed that the cracks form in a direction perpendicular to the maximum principal stress. Afterwards, use the direction transformation matrix T to calculate the total strain εxyz Transform to the nst direction. Total strain along the crack direction εnst Obtain stress using бnst Finally, transform the stress into the xyz coordinate system and calculate the stress throughout the entire coordinate system бxyz.

During this process, the focus is on establishing a stress-strain relationship along the crack direction. In the total strain crack model, it is assumed that the uniaxial stress-strain relationship along the j direction is as follows:

$$\sigma_{\mathrm{j}} = \mathrm{f}_{j}(\alpha, \varepsilon_{nst}) \cdot g_{j}(\alpha, \varepsilon_{nst}) \tag{1}$$

The basic strength in the direction of the crack is multiplied by the loading unloading function to obtain the stress. After the appearance of cracks, the stiffness of concrete will undergo irreversible reduction. At this time, even the same total strain will result in different corresponding stresses under different loading and unloading conditions. To reflect this characteristic, we will introduce a physical quantity α, By using responsive representation, the mentioned damage description is transformed into damage variables or internal variables to record its loading or unloading state.

3 Analysis of Segment Damage in the Second Stage

Overlay the displacement of the tunnel before and after excavation, as well as after reinforcement, into the DIANA refined model, introduce a cracking factor to characterize the damage state of the tunnel, and analyze the quantitative values of pipe misalignment, joint opening, and pipe cracking.

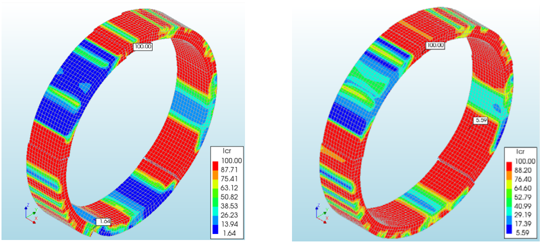
3.1 Analysis of Damage to Pipe Segment Structure Before Excavation

In order to more accurately predict the future trend of crack propagation, a concept called Icr "cracking factor" was introduced. Its definition is shown in equation (2):

$$I_{\mathrm{cr}} = \frac{\mathrm{f_t}}{\sigma_1} \tag{2}$$

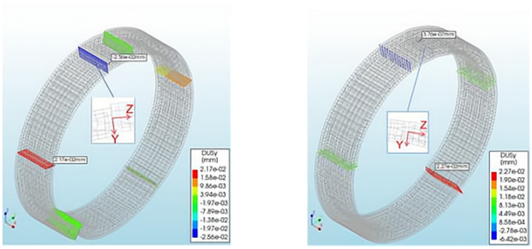
In the formula, ft is the tensile strength of concrete; б1 is the maximum principal stress of the current unit. The cracking factor can be used as the safety factor for concrete units. If its value approaches 100, it indicates a lower probability of cracking.

From Figure 2, it can be seen that the distribution of cracking factors for section 1 of the upward line and section 2 of the downward line before excavation is generally consistent. The cracking factors for the side near the foundation pit and the bottom of the arch are relatively small, indicating that the blue damage zone will further expand in subsequent working conditions. Local damage may occur on the outer side of the left arch waist and the inner side of the right arch waist, and these damages may be relatively scattered. The deformation of misalignment within the two section segments is shown in Figure 3. According to the detection of misalignment, the misalignment near section 1 reached 0.53 mm, and the misalignment near section 2 reached 0.43 mm. The calculated results for section 1 and section 2 were approximately 0.03 mm and 0.02 mm, respectively. The results indicate that the calculated misalignment of the two sections is consistent with the results obtained from on-site structural testing.



(a) section 1 (b) section 2

Fig. 2. Cloud map of tunnel cracking factors in two sections

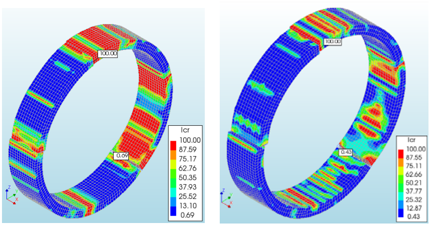


(a) section 1 (b) section 2

Fig. 3. The amount of misplaced platform in two cross-sectional tunnels

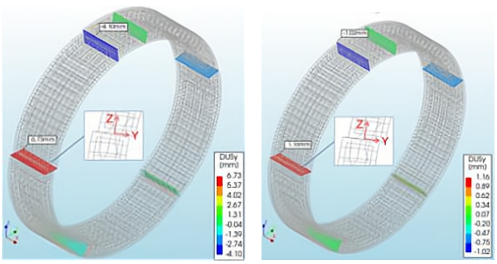
3.2 Analysis of Damage to Pipe Segment Structure After Excavation

As shown in Figure 4, compared with before excavation, the damage of the two section segments has increased to a certain extent. The safety zone of section 1 only appears at the arch and inside the segments, and both are relatively small discrete areas; The location of the safety zone in section two is roughly the same as that in section one, and the safety zone is relatively continuous and complete, indicating that the pipe segments on the near excavation side have caused significant damage from the cracking factor, while the pipe segments on the far excavation side have suffered less damage. After excavation, as shown in Figure 5, it was found that the displacement of the two cross-sections at the arch crown position increased to a certain extent. The displacement of the two cross-sections at the arch crown was 4.1 mm and 1.02 mm respectively, which increased by 4.07 mm and 1.14 mm compared to before excavation. The maximum displacement occurred at the left arch waist position, which was 6.73 mm and 1.16 mm, respectively. Due to excavation, the convergence deformation caused by section 2 is smaller than that of section 1. After excavation, the pipe segments all show smaller displacement amplitude. Before excavation, the maximum convergence deformation of section 1 reached 46.5 mm, with the most severe damage and the largest increase in displacement.



(a)section 1 (b)section 2

Fig. 4. Cloud map of tunnel cracking factors in two sections after excavation the tunnel joint tensioning amount

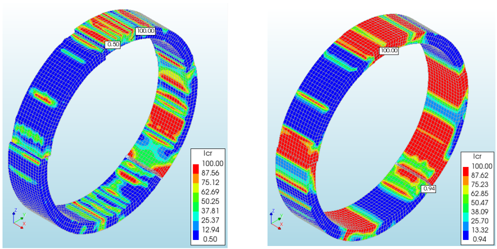


(a)section 1 (b)section 2

Fig. 5. The amount of misplaced platforms in the two sections of the tunnel after excavation of tunnel after excavation

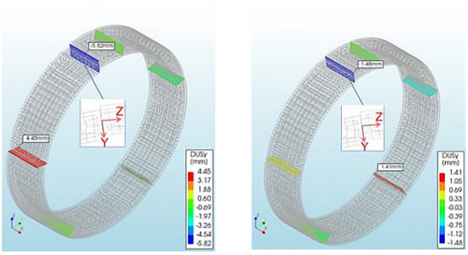
3.3 Damage Analysis of Reinforced Pipe Segment Structure

After reinforcement by slant support, the damage nephogram, dislocation amount, joint opening and crack width results of the uplink and downlink of the tunnel at the bottom of the pit in South Zone 2 are shown in Figure 6. The distribution of cracking factor nephogram after reinforcement is roughly the same as that before reinforcement, in which the inner side of the ascending line at the arch waist and the outer side of the arch waist at the side of the descending line near the foundation pit are the most obvious changes compared with the unreinforced results, and the "safety zone" of the red part is more continuous, indicating that the slant support bears a part of the stress of the excavated soil, and the horizontal internal force is dominant. As shown in Figure 7, the displacement of the arch crown of the two sections after reinforcement is 5.82 mm and 1.48 mm respectively, increasing by 1.72 mm and 0.46 mm respectively compared with that of the foundation pit without slant support reinforcement after excavation, and different from the results of the excavation of the foundation pit without reinforcement, the maximum displacement of the single ring of the tunnel both appear at the arch crown, It shows that the adoption of foundation pit slant support reinforcement in this project has a certain inhibitory effect on the horizontal displacement of the existing tunnel under the excavation of the side foundation pit.



(a)section 1 (b)section 2

Fig. 6. Cloud map of tunnel cracking factor in two sections after reinforcement



(a)section 1 (b)section 2

Fig. 7. The amount of misalignment between the two sections of the tunnel after reinforcement

4 Conclusion

This article is based on a two-stage analysis method, combined with the results of the tunnel structure operation period inspection and the calculation of the geological structure method under the excavation of the foundation pit, to superimpose the displacement of a single ring tunnel and obtain the lateral response of the tunnel structure. The specific conclusions are as follows:

(1) A new method for analyzing the impact of lateral deep foundation pit excavation on existing tunnels is proposed by combining large-scale finite element software PLAXIS and DIANA, and using displacement superposition method. This method takes into account the needs of macroscopic large-scale modeling of complex systems of deep foundation pits and multi line tunnels, as well as the analysis of lining segment structures, and improves computational efficiency.

(2) The two-stage calculation analysis based on displacement superposition method shows that, considering the nonlinear influence of the structure, all section pipe segments of the foundation pit show certain damage before excavation. The displacement superposition of the single ring refined models before excavation, after excavation, and after reinforcement is carried out to obtain the quantification of tunnel lining damage. The calculation results of the single ring pipe segments of the tunnel are basically consistent with the measured results, and the development trend of pipe segment damage after reinforcement is reduced to a certain extent.

Acknowledgments

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