Research Article

Optimization Analysis of Deformation of Underlying Tunnel in Dewatering and Excavation of Phreatic Aquifer

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Abstract

This paper theoretically analyzed the deformation law of the underlying tunnel caused by dewatering and excavation of deep foundation pit in the phreatic aquifer area, which is based on the Mindlin solution and the double-sided elastic foundation beam theory, and used the finite difference method and the fluid-solid coupling principle to conduct three-dimensional numerical simulation of dewatering and excavation of deep foundation pit with fluid-solid coupling by using FLAC3D5.00. This research shows that the layered and segmented excavation method from the middle to the end by dewatering the skip layer has a better effect on optimizing the deformation of the underlying tunnel through the simulation of three excavation methods and two dewatering schemes crossing each other, which is about 2.5% less than the layer-by-layer dewatering scheme. In addition, the deformation law of the simulated value is the same as the theoretical value, and the simulated value is slightly larger than the theoretical value. Underlying tunnel only just exists vertical deformation at the direct center of the foundation pit, and the maximum deformation is about 3.054 mm under the dewatering well of the jumping layer and W3. With the dewatering of jumping layer and the third excavation mode (W3), underlying tunnel only just exists lateral displacement at the position where is the retaining structure, and the maximum displacement is 1.606 mm.

1. Introduction

In recent years, with the continuous acceleration and development of urbanization, various large-scale construction projects, such as rail transit, have been increasing, and the scale and difficulty of underground space development are becoming larger and larger, and the characteristics of presentation are also different. The construction of subway stations and deep foundations in the vicinity of tunnels will have different degrees of impact on the tunnels, and some will cause excessive deformation of the tunnels and affect normal use. The horizontal displacement, vertical displacement, and radial convergence control values of the tunnel are \(<20\) mm. The relative curvature control value of tunnel deformation is \(<1/2500\) m [1]. Lin and Xiao [2, 3] researched the laws of deformation and force of the deep foundation pit excavation with the underlying tunnel by theoretical calculation, field test, and numerical simulation.

Ding et al. [4] calculated the effect of the length of the uplift pile under the bottom of the foundation pit to reduce the uplift deformation of the tunnel by the ABAQUS finite element software. In addition, in order to avoid safety hazards, dewatering preparation is required before excavation in areas with high groundwater levels. On the other hand, the pore water pressure is dissipated with the decrease in water level, so the effective stress increases, which will in turn increase the surrounding surface settlement. Therefore, the groundwater level needs to be strictly controlled [5]. Chen et al. [6–10] established a calculation model of seepage field and stress field by using three-dimensional finite difference or finite element method, analyzed the influence and law of foundation pit excavation and dewatering on surface subsidence, and provided reference value for actual construction. The influence of deep foundation pit excavation and dewatering on adjacent pile foundation is analyzed through a series of improvement measures about
optimization of dewatering scheme [11]. Wu et al. [12] and Do et al. [13] prove discrete element methods, such as discontinuous deformation analysis (DDA), are powerful way to predict the behaviors of jointed rock masses and its influence by trap-door tests.

The above analysis does not systematically analyze the optimization of the deformation of the underlying tunnel caused by the foundation excavation and dewatering. This study takes the deep foundation pit with the underlying tunnel in phreatic aquifer as an example, based on theoretical calculations and field experiments, establishes the seepage field and the stress field through FLAC3D5.00 finite difference software, analyzes the effect of dewatering scheme and excavation method on controlling the deformation of the underlying tunnel, and provides reference for similar engineering projects.

2. Engineering Background

2.1. Deep Foundation Pit and Tunnel. The existing tunnels of the project are double-lined tunnels running through the north-south direction. The total length of the tunnel is 27.36 km. This study takes a section of the north-south as the research object. The centers of the two tunnels are 16.0 m apart. The net spacing is 10.0 m and the vertical distance, that is, from the tunnel vault to the surface, is about 17.0 m. Both subway tunnels have an outer diameter of 6.0 m, an inner diameter of 5.7 m, and a segment lining thickness of 0.3 m. There is a new deep foundation pit with a length of 106.0 m, a width of 28.0 m, and a depth of 14.0 m located above the two-line tunnel. The distance is approximately 5.0 m, that is, from the bottom of the deep foundation pit to tunnel vault. The total depth of excavation of the foundation pit is 14.0 m. The vertical retaining structure of the deep foundation pit is to be drilled with φ1 000@800 mm. The length of pile is about 18.0∼24.0 m, which is filled with C30 concrete. The horizontal support system adopts a three-way support design. The first horizontal support adopts a reinforced concrete support system with a size of 800 mm × 900 mm, which is located around 2 m below the surface. The second and third horizontal support are supported by steel with a diameter of φ 609 mm and a wall thickness of 18 mm, which are located around 4 m and 6 m below the surface, respectively. The floor plan is shown in Figure 1. The foundation pit adopts a double-row high-pressure rotary jetting pile to cut off the groundwater and reduces the groundwater level by dewatering at the outside of foundation pit. In order to meet the dewatering demand, there are 22 square dewatering wells outside the foundation pit. The side length is equivalent to 1.0 m, and the dewatering well is about 3.0 m away from the outer edge of the foundation pit. The adjacent dewatering wells are about 10.0 m apart, and the depth of the dewatering well is 17.0 m. In addition, in order to prevent excessive deformation and force during the excavation of the underlying tunnel, 4 rows of ×10 uplift piles are arranged on both sides of the existing tunnel, and the uplift piles are φ900 @2 000-mm bored pile. The length of the pile is 25 m, and the bottom of the foundation pit is reinforced. The concrete weight plate is placed on the upper part of the uplift pile, and the pile head is anchored into the plate to form a whole force to reduce the vertical deformation of the tunnel.

2.2. Geological Conditions. According to the geotechnical engineering investigation report, the construction area is mainly quaternary sediment, and surface feature is II level terrace, flat on the ground, and did not reveal artificial holes such as air-raid shelter. Geological conditions are relatively stable, and there is no fracture. The survey revealed a depth of 40 m, and soil mainly included miscellaneous fill, sand, and gravel in the survey area. The stratigraphic characteristic parameters are shown in Table 1 and the stratigraphic section is shown in Figure 2.

2.3. Hydrogeological Conditions. The region has a dry climate with less rainfall, and the groundwater type is quaternary loose-layer pore phreatic water [14], flowing to the east and west, and receiving rainfall and lateral runoff recharge. The initial groundwater level is about 2.011 m below the surface according to geological exploration, which is about 14 m above the base of the proposed foundation pit. The inflow of the single well in the process of dewatering is 500~1200 m³/d, and the annual variation of the groundwater level is 0.6~0.8 m. The depth of the dewatering well is about 14 m (about 2 m [15] below the base of the foundation pit), and the hydrological parameters of the strata (according to the geotechnical investigation report) are listed in Table 2.

3. Theoretical Analysis of Deformation of Underlying Tunnel Caused by Dewatering and Excavation

3.1. Additional Stress in the Underlying Tunnel Caused by Dewatering and Excavation

3.1.1. Establishment of Mechanical Model. The unloading of excavation and dewatering of foundation pits can be equivalent to the loading effect, that is, increasing and decreasing of uniform load in the calculation formula. When a concentrated force acts on the foundation, the additional stress in the foundation is calculated with the help of the elasticity theory—semi-infinite elastic body (R. D. Mindlin, 1936) [16]. The mechanical model of dewatering and excavation of the foundation pit is shown in Figure 3. Here, d is the excavation depth of the foundation pit, d₀ is the distance from the bottom of the foundation pit to the bottom of the envelope structure, H is the total depth of the envelope structure, Z₁ is the distance from the surface to the centerline of the tunnel, and S is the distance from the envelope structure to the side wall of the tunnel, and D is the outer diameter of the tunnel. It is assumed that in the calculation, (1) the soil is homogeneous within the elastic half space; (2) the tunnel structure is an infinitely long homogeneous elastomer, the section remains unchanged, and the side of the foundation pit is parallel to the longitudinal axis of the tunnel; (3) only the additional stress after foundation excavation and dewatering is considered, regardless of the influence of the tunnel side on the additional stress of the
3.1.3. Analysis of Unloading at the Bottom of the Pit. The soil stress is released at the bottom of the pit after dewatering. While calculating, it is equivalent to a uniform load in the vertical direction. The formula [18] is as follows:

\[ \sigma = \gamma_1 d_1 + \sum \gamma_j h_j + [P_a - (1 - \xi_z)U_{ai}] \],

where \( \gamma_1 \) is the unit weight of the soil in the A1 area, \( d_1 \) is the thickness of the soil in the A1 area, \( \gamma_j \) is the unit weight of each layer of the soil in the area A2, \( h_j \) is the thickness of each layer of the soil in the area A2, \( P_a \) is the atmospheric pressure, generally value is 101 kPa, \( \xi_z \) is the saturation parameter at the calculation point in the Z direction, which can be calculated according to the survey report, and \( U_{ai} \) is the absolute pore gas pressure at the calculation point.

Since the tunnel in the project is directly below the bottom of the foundation pit and the distance between them is small, the obstruction effect of the retaining structure at the bottom of the foundation pit can be neglected [19]. There is always residual stress in the soil before and after unloading. According to the concept of residual stress coefficient [20], it is calculated as follows:

\[ \alpha = \frac{\text{Residual stress}}{\text{Total unloading stress}} \leq 1.0. \]  

(2)

According to the measured data of a large number of foundation pits in the area, the following relationship was found [21]:

\[ H_r = f(H) = \frac{H}{0.0612H + 0.19}. \]  

(3)

where \( H \) is the excavation depth (m) of the foundation pit and \( H_r \) is the influence depth of residual stress (m) [21].

\[ \alpha = \begin{cases} 
\frac{a_0 + \frac{0.95 - \partial_0 h_0^2}{H_r^2}}{1}, & (0 \leq h_0 \leq H_r), \\
1.0, & (h_0 > H_r), 
\end{cases} \]  

(4)

where \( h_0 \) is the thickness of the overlying soil layer at the calculation point \( a_0 = 0.31 \). Therefore, the stress release of soil unloading is equivalent to

\[ \sigma = \gamma_1 d_1 + \sum \gamma_j h_j + [P_a - (1 - \xi_z)U_{ai}] (1 - \alpha). \]  

(5)

According to the basic stress solution of Mindlin’s vertical uniform load, the vertical stress of a point \((L_1, \gamma_1, z_0)\) at the axis of the tunnel is available by integral calculation under the action of force \( d\xi d\eta d\zeta \) at a point \((\xi, \eta, \zeta)\) on the horizontal surface of the deep foundation pit. The vertical additional stress of the longitudinal axis of the underlying tunnel structure can be expressed as follows [22]:

\[ \sigma_z = \frac{\sigma}{8\pi (1 - \nu)} \left\{ (1 - 2\nu)(z_0 - H) \right\} \left\{ \int_T d\xi d\eta \right\} T_1 \]
\[ + 3(z_0 - H)^3 \left\{ \int_T d\xi d\eta \right\} T_2 \]
\[ + \left[ 3(3 - 4\nu)z_0(z_0 + H)^2 - 3H(z_0 + H)(5z_0 - H) \right] \]
\[ \cdot \left\{ \int_T d\xi d\eta + 30 d\zeta(z_0 + H)^3 \right\} \int_T d\xi d\eta T_2^3, \]  

(6)

where

\[ T_1 = \sqrt{(L_1 - \xi)^2 + (\gamma_1 - \eta)^2 + (z_0 - H)^2}, \]
\[ T_2 = \sqrt{(L_1 - \xi)^2 + (\gamma_1 - \eta)^2 + (z_0 + H)^2}. \]  

(7)

Here, \( \nu \) is the Poisson’s ratio and \( T \) is the integral region corresponding to the bottom of the foundation pit.
The integral in equation (6) can be calculated by the five-node Gauss–Legendre numerical integration method. The calculation process is as follows, taking \[\int_1^T d\xi d\eta/T_1^2\] as an example:

\[
\int d\xi d\eta/T_1^2 = \int_{-B/2}^{B/2} \int_{-L/2}^{L/2} \frac{d\xi d\eta}{[(x_1 - \xi)^2 + (L_1 - \eta)^2 + (z_0 - d)^2]^{3/2}}
\]

\[
= \int_{-B/2}^{B/2} \int_{-L/2}^{L/2} \frac{\xi - x_1}{(L_1 - \eta)^2 + (z_0 - d)^2} \sqrt{(x_1 - \xi)^2 + (L_1 - \eta)^2 + (z_0 - d)^2} \left| \frac{L/2}{-L/2} \right| d\eta d\xi
\]

\[
= \sum_{i=2}^{3} \frac{\omega_i ((L/2) - x_1)(B/2)}{((B/2)t_i - L_1)^2 + (z_0 - d)^2 \sqrt{((L/2) - x_1)^2 + ((B/2)t_i - L_1)^2 + (z_0 - d)^2}} - \frac{\omega_i ((L/2) - x_1)(B/2)}{((B/2)t_i - L_1)^2 + (z_0 - d)^2 \sqrt{(-((L/2) - x_1)^2 + ((B/2)t_i - L_1)^2 + (z_0 - d)^2}}
\]
where \( w = [0.2369 \ 0.4786 \ 0.5688 \ 0.4786 \ 0.2369], \ t = [-0.9061 \ -0.5395 \ 0 \ 0.5385 \ 0.9061] \).

Therefore, the five-node Gauss–Legendre numerical integration method is used to calculate the vertical additional stress and horizontal additional stress at any point on the longitudinal axis of the tunnel caused by foundation pit excavation.

### 3.2. Theoretical Analysis of Tunnel Vertical Deformation

#### 3.2.1. Presentation of Foundation Model

Because the upper layer of the tunnel exists as soil, its role cannot be ignored. In addition, the soil of underlying tunnel will affect the deformation of tunnel. A more actual theory is used, that is, double-sided elastic foundation beam theory, which is based on the Winkler hypothesis, and the theory satisfies the following assumptions: (1) the tunnel structure can be simplified as an elastic beam; (2) the foundation model of the two adjacent soil masses above and below the tunnel conforms to the Winkler foundation model, that is, the pressure \( p \) is proportional to the foundation settlement; (3) the deflection of the beam is equal to the deformation value of the foundation under the external load of the foundation beam; (4) the foundation beam conforms to the assumption of the plane section, and the theory of elastic mechanics and material mechanics can be used to calculate the

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**Table 2: Soil layer hydrological parameters.**

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil type</th>
<th>Top elevation (m)</th>
<th>( K_H ) (m/d)</th>
<th>( K_v ) (m/d)</th>
<th>Adopted method</th>
<th>Aquifer type</th>
<th>Water level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>①</td>
<td>Miscellaneous fill</td>
<td>5.5~1.57</td>
<td>6</td>
<td>6</td>
<td>Experiment test</td>
<td>Phreatic aquifer</td>
<td></td>
</tr>
<tr>
<td>②1</td>
<td>Silty fine sand</td>
<td>1.57~4.3</td>
<td>16</td>
<td>8</td>
<td>Experiment test</td>
<td>Phreatic aquifer</td>
<td></td>
</tr>
<tr>
<td>②2</td>
<td>Silty fine sand</td>
<td>-4.3~9.92</td>
<td>16</td>
<td>8</td>
<td>Experiment test</td>
<td>Phreatic aquifer</td>
<td></td>
</tr>
<tr>
<td>③</td>
<td>Pebble</td>
<td>-9.92~</td>
<td>20</td>
<td>10</td>
<td>Empirical value</td>
<td>Phreatic aquifer</td>
<td></td>
</tr>
</tbody>
</table>

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**Figure 3: The front of view of mechanical model.**

**Figure 4: The planform of mechanical model.**
displacement and force of the beam; (5) the friction between the beam and the foundation is ignored, that is, the foundation reaction is perpendicular to the contact surface.

3.2.2. Vertical Deformation Equation of Tunnel. The double-sided elastic foundation beam model and the microelement of the tunnel are shown in Figures 5 and 6. The microelement generates a displacement \( w \), under the action of the additional load \( q \). The upper elastic foundation acts on the beam as \( p_2 \), and the elastic beam is subjected to the reaction force of the lower spring foundation as \( p_1 \). According to equilibrium conditions,

\[
\sum F = 0 \quad \text{(considering downward direction as the positive direction)}:
\]

\[
Q + dQ - Q + p_2 D \frac{dM}{dy} + q dx - p_1 D \frac{dM}{dy} - F_{\text{flotage}} = 0,
\]

\[
Q + dQ - Q + p_2 D \frac{dM}{dy} + q dx - p_1 D \frac{dM}{dy} - \left( \frac{\rho g \pi D^2 dx}{4} \right) = 0,
\]

\[
\frac{dQ}{dx} = (p_1 - p_2) D - q + \left( \frac{\rho g \pi D^2}{4} \right). \tag{9}
\]

\[
\sum M = 0 \quad \text{(to the right section)}:
\]

\[
M + dM - M - Q dx + p_2 D \frac{dM}{dy} + q dx - p_1 D \frac{dM}{dy} = 0
\]

\[
- p_1 D \frac{dx}{2} - \left( \frac{\rho g \pi D^2 dx}{4} \right) \frac{dx}{2} = 0. \tag{10}
\]

Ignoring the higher order indefinite small,

\[
\frac{dM}{dx} = Q. \tag{11}
\]

According to formula (9) and formula (11),

\[
\frac{d^2 M}{dx^2} = (p_1 - p_2) D - q + \left( \frac{\rho g \pi D^2}{4} \right). \tag{12}
\]

According to the hypothesis (2),

\[
p_1 = k_1 s, \quad p_2 = k_2 s, \tag{13}
\]

where \( k_1 \) and \( k_2 \) are the foundation bed coefficients of the upper and lower part of the foundation beam, respectively, which can be calculated by the modified formula of load plate experiment, looking up at the table given in [23] and the empirical formula method of Vesic [24].

According to the material mechanics knowledge,

\[
\omega'' = -\frac{M}{EI}. \tag{14}
\]

According to formulas (12), (13), and (14),

\[
EI \frac{d^4 w(y)}{dy^4} + (k_1 + k_2) Dw(y) = q(y) - \left( \frac{\rho g \pi D^2}{4} \right). \tag{15}
\]

In this paper, the coefficients of the foundation bed under tension and compression are not distinguished for the same soil layer, and the same value is adopted, namely, \( k_1 = k_2 = k \). Therefore, formula (15) can be written as follows:

\[
EI \frac{d^4 w(y)}{dy^4} + 2Kw(y) = q(y) - \left( \frac{\rho g \pi D^2}{4} \right). \tag{16}
\]

Here, \( K \) is the product of the foundation bed coefficient \( k \) and the tunnel outer diameter \( D \), \( EI \) is the tunnel equivalent bending stiffness, \( w(y) \) is the tunnel vertical displacement, and \( q(y) \) is the vertical distributed load on the tunnel caused by dewatering and excavation of foundation pit and that is obtained by multiplying formula (6) by the outer diameter \( D \) of the tunnel.

Equation (16) is a nonhomogeneous fourth-order ordinary differential equation, which can be solved by the finite difference method.

4. Field Pumping Test and Parametric Analysis

4.1. Pumping Test. In the pumping test, in order to avoid the \( R \) (influence radius) value, which is difficult to be accurate and to reduce the influence of well damage in the pumping well, it is better to drill two observation holes near the representative pumping well, so that the parameters obtained are more reliable. However, it should be noted that the observation hole should not be too far away from the pumping well. Otherwise, when the pumping time is insufficient, the flow \( Q \) in the cross section of water through the observation hole is much smaller than that of the pumping well, and the calculated value of \( k \) will be larger.

**Figure 5: Double-sided elastic foundation beam model.**

**Figure 6: Diagram of forces on microelements.**
5. Numerical Simulation of Pumping Test

5.1. Mathematical Model. The differential equation of the unsteady flow of groundwater [25] is as follows:

\[
\frac{\partial}{\partial x} \left( K_{xx} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_{yy} \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_{zz} \frac{\partial h}{\partial z} \right) - W = S_x \frac{\partial h}{\partial t}, \quad (x, y, z) \in \Omega,
\]

\[
h(x, y, z) = h_0(x, y, z, t), \quad (x, y, z) \in \Gamma_1,
\]

\[
K_{xx} \frac{\partial h}{\partial x} + K_{yy} \frac{\partial h}{\partial y} + K_{zz} \frac{\partial h}{\partial z} \bigg|_{\Gamma_1} = q(x, y, z, t), \quad (x, y, z) \in \Gamma_2,
\]

\[
h(x, y, z, t) \bigg|_{t=t_0} = h_0(x, y, z), \quad (x, y, z) \in \Omega.
\]

(18)

Here, \(K_{xx}, K_{yy}, \) and \(K_{zz}\) are the permeability coefficients (cm/s) in the \(x, y, \) and \(z\) directions, respectively; \(h\) is the water level at the position \((x, y, z)\) and at time \(t\); \(h_0\) is the initial water level at the position \((x, y, z)\); \(W\) is the groundwater recharge or exchange (1/day); \(S_x\) is the special storage (1/m) at the position \((x, y, z)\); \(\Omega\) is the calculation area; \(\Gamma_1\) and \(\Gamma_2\) are the first and second types of boundary conditions, respectively; \(q\) is the lateral replenishment of each unit area on the boundary \(\Gamma_2\); and \(n_x, n_y, \) and \(n_z\) are vectors perpendicular to the \(x, y, \) and \(z\) directions on the boundary \(\Gamma_2\), respectively. In this numerical simulation, it is assumed that the soil layer is isotropic in the horizontal direction, that is, \(K_{xx} = K_{yy}\).

5.2. Numerical Model. In this paper, the finite difference software FLAC3D5.00 is used for 3D analysis and calculation. According to the Saint-Venant principle, the numerical model size is simplified to 208 m × 142 m × 61 m. The calculation model is divided into 61,724 elements, including 73,321 nodes, as shown in Figure 7. The horizontal and vertical displacements at the bottom of the model are constrained. The bottom surface of the model is set to a watertight boundary. The horizontal displacement around the model is constrained, and the initial water level is set to the fixed head boundary at the 2.011 m below the surface. The upper surface of the model is a free boundary. The pumping well wall adapts to the node seepage boundary condition. The stress-strain relationship of the soil is in accordance with the Mohr–Coulomb constitutive theory. The stress-strain relationship of lining segments, uplift piles, and horizontal inner layer supports under foundation pit excavation and dewatering is in accordance with the linear elastic constitutive theory. In the numerical simulation, the tunnel lining, the foundation pit floor, etc. are simulated by structural elements, and the envelope structure is equivalent to diaphragm wall with a thickness of 1 m. Specific parameters that were calculated and measured are shown in Table 5. Pumping test and numerical simulation of water level reduction are shown in Figures 8 and 9. It can be clearly seen that the curves obtained by the test and numerical simulation in the pumping well and the observation well are basically consistent, indicating that the parameter selection in the numerical model is in accordance with the actual situation. It can be used as the calculation parameter for the next numerical simulation.

6. Optimization Analysis of Deformation of Underlying Tunnel

With the excavation unloading of the deep foundation pit, the bottom of the pit will have a springback deformation, which will cause the uplift deformation of the underlying tunnel. Because the pore water pressure is dissipated as the water level decreases, according to the principle of effective stress, the effective stress increases, which is equivalent to compacting the soil, which is beneficial to reduce uplift deformation of the underlying tunnel. This paper mainly considers three kinds of excavation methods and two kinds of dewatering schemes, which include six combined schemes. One of the schemes of the least deformation to the underlying tunnel could be obtained through numerical simulation analysis, which provides reference for practical engineering.
6.1. Excavation Method. According to the characteristics of narrow and long foundation pits, appropriate layered and segmented excavation methods are selected that present a stepped excavation mode. The time of excavation and support formation of each layer is strictly limited, generally ranging from 12 h to 36 h. Various construction parameters of layered and segmented excavation have been proved safe and reliable by a large number of engineering practices [26]. In this paper, three kinds of layered and segmented excavation methods are adopted, as shown in Figures 10–12. The numbers in Figures 10–12 represent the sequence of excavation.

According to Figures 10 and 11, the deep foundation pit is divided into 12 regions along the long side direction, respectively, from A to L, and the deep foundation pit is divided into 7 layers along the depth direction, and the thickness of each layer is 2 m. Compared with W1, the biggest difference of W2 is that continuous excavation without subsection can be adopted for the soil layer above the bottom of the first reinforced concrete support, which has been proved to be effective for the overall stability of the foundation pit [26]. Therefore, single excavation is first used in the soil layer in numerical simulation. After the strength of the reinforced concrete support reaches the design requirements, the lower layer of the soil is excavated by layered and segmented method, which is from one end to the other. In addition, when the excavation process is completed, the concrete weight plate should be applied to the bottom surface of the foundation pit immediately, and forms a whole with the uplift pile, and the concrete weight plate should be applied to other bottom surface of the foundation pit. It is more effective in reducing the floating deformation of the tunnel.

The third excavation mode is shown in Figure 12. First, the foundation pit is divided into 13 areas along the long side of the foundation pit, which are from A to M, and the foundation pits divided into 7 soil layers along the depth direction, and the thickness of each layer is 2 m. The first layer of soil adopts continuous excavation without subsection, and the lower soil layer adopts layered and segmented method, which is from the middle to the both ends, and the excavation method can effectively accelerate the speed of construction.

6.2. Dewatering Schemes. Dewatering adopts two kinds of schemes, which are layer-by-layer dewatering and dewatering of jumping layer. Layer-by-layer dewatering reduces the groundwater level to 2 m below the excavation depth before each excavation, and the dewatering is done only once. While excavating a layer, dewatering of the jumping layer is a one-time reduction of the groundwater level to about 2 m below the bottom of the foundation pit.

7. Optimization Analysis of Numerical Simulation

The numerical model and parameters are the same as in Figure 8 and Tables 1, 2, and 5. The method of setting the seepage velocity of the well wall of the dewatering well is adopted to realize the dewatering simulation. By compiling fish language, when the pore water pressure at the center of the foundation pit of a certain dewatering depth is zero, dewatering is stopped to meet the requirement of controlling the dewatering depth. Model null command was used to simulate the excavation process.

7.1. Vertical Deformation Analysis of Tunnels with Different Excavation Methods under Layer-by-Layer Dewatering. The vertical deformation analysis of the tunnel under three different excavation modes under the condition of layer-by-layer dewatering is shown in Figure 13.

In Figure 13, the vertical deformation of the tunnel first increases and then decreases with the increase of the tunnel length, presenting a parabolic shape. The maximum value is

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**Table 3: Pumping rate, time, and depth in three steps.**

<table>
<thead>
<tr>
<th>Step</th>
<th>Pumping time (min)</th>
<th>Pumping rate (m³/d)</th>
<th>Drawdowns of pumping well $s_w$ (m)</th>
<th>Drawdowns of observation well 1 $s_1$ (m)</th>
<th>Drawdowns of observation well 2 $s_2$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>540</td>
<td>1192</td>
<td>12.5</td>
<td>1.06</td>
<td>0.80</td>
</tr>
<tr>
<td>2</td>
<td>640</td>
<td>782</td>
<td>10.5</td>
<td>0.93</td>
<td>0.73</td>
</tr>
<tr>
<td>3</td>
<td>360</td>
<td>509</td>
<td>5.5</td>
<td>0.78</td>
<td>0.61</td>
</tr>
</tbody>
</table>

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**Table 4: Analysis and calculation results of two well combinations.**

<table>
<thead>
<tr>
<th>Well combination</th>
<th>Calculated permeability coefficient (m/d)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Step 1</td>
</tr>
<tr>
<td>One pumping well</td>
<td>15.56</td>
</tr>
<tr>
<td>Two observation wells</td>
<td>35.12</td>
</tr>
</tbody>
</table>
obtained when the tunnel length is 71 m, and the uplift trend is shown when the tunnel length is 56–84 m where is under the foundation pit. In the area outside the foundation pit, the tunnel presents a sinking trend. The reason is that the pore water pressure of the soil decreases, the effective stress increases, and the soil compacts and consolidates with the

Table 5: Calculation parameters of structural elements.

<table>
<thead>
<tr>
<th>Name</th>
<th>Density (g·m⁻³)</th>
<th>Elastic modulus (GPa)</th>
<th>Poisson ratio</th>
<th>Cross-sectional area (m²)</th>
<th>Moment of inertia (m⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel lining</td>
<td>2.50</td>
<td>30</td>
<td>0.22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation pit floor</td>
<td>2.71</td>
<td>35.5</td>
<td>0.24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete support</td>
<td>2.50</td>
<td>30</td>
<td>0.18</td>
<td>7.2 × 10⁻¹</td>
<td>8.7 × 10⁻³</td>
</tr>
<tr>
<td>Steel support</td>
<td>7.86</td>
<td>200</td>
<td>0.24</td>
<td>3.0 × 10⁻³</td>
<td>2.6 × 10⁻³</td>
</tr>
<tr>
<td>Diaphragm wall</td>
<td>2.50</td>
<td>30</td>
<td>0.16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uplift pile</td>
<td>2.50</td>
<td>30</td>
<td>0.21</td>
<td>5.0 × 10⁻¹</td>
<td>2.0 × 10⁻²</td>
</tr>
</tbody>
</table>

Figure 8: Change in water level in pumping well.

Figure 9: Change in water level in observation well 1 and well 2.
However, due to the unloading of the soil in the foundation pit, the lower soil appears to rise. Because of the uplift deformation of the underlying tunnel in the foundation pit area, it indicates that the uplift value of the tunnel caused by foundation pit excavation is larger than that caused by dewatering. The minimum uplift value of the tunnel caused by W3 excavation method is 3.131 mm, which is about 15% and 23% lower than that of W1 and W2, respectively. The settlement size of the tunnel below the outer region of the foundation pit is basically the same, indicating that the layered and segmented excavation method from the middle
to the two ends is better than the other two excavation methods.

7.2. Analysis of Vertical Deformation of Tunnels under Different Excavation Methods in the Dewatering of Jumping Layer. The vertical deformation analysis of the tunnel under three different excavation modes under the condition of dewatering of jumping layer is shown in Figure 14.

Under the condition of the dewatering of jumping layer, the vertical deformation of the tunnel is the same as that of the layer-by-layer dewatering, which is parabolic. The vertical deformation of the tunnel is the smallest under the W3 excavation mode. The difference is the vertical deformation value. Now under the W3 excavation mode, the influence of dewatering of jumping layer and the influence of layer-by-layer dewatering on the deformation of the tunnel and the theoretical value are compared, as shown in Figure 15.

In Figure 15, the impact of dewatering of jumping layer on the deformation of the underlying tunnel is less than that of the layer-by-layer dewatering under the W3 excavation mode, which is reduced by about 2.49%. In addition, the theoretical value is smaller than the simulated value. The deformation contour of dewatering of jumping layer of the tunnel under the W3 excavation mode (magnification 300 times) is shown in Figure 16.

7.3. Deformation Analysis of Tunnel Length Y of 55 m and 70 m. When Y is 70 m, that is, the tunnel is directly below the center of the foundation pit. The tunnel structure only deforms in the vertical direction, and the deformed contour of the tunnel is oval, as shown in Figure 17. When Y is 55 m, that is, at the tunnel structure under the enclosure structure, there is almost no upward deformation of the tunnel structure, which is mainly manifested as lateral deformation. Moreover, at the top of the tunnel, the lateral deformation of the structure at the bottom of the tunnel is larger, as shown in Figure 18. This is because it has the existence of the compaction of the retaining structure directly above the tunnel, inhibiting the rise of the tunnel at that point. In addition, the department of tunnel structure locates at the border, that is, at the nonexcavation and excavation areas. The nonexcavation area adopted dewatering, which resulted in settlement of the tunnel structure. But the excavation area adopted unloading of soil, which caused rising of the tunnel structure. So the tunnel deformation at Y = 55 m is a lateral deformation with an angle to the vertical direction.

8. Conclusion

Through the above research and analysis, for the project, the following conclusions can be drawn.

(1) Both dewatering schemes have the minimum impact on the deformation of the tunnel under the excavation mode of W3, indicating that the layered and segmented excavation mode from the middle to the two ends is better when there are underlying tunnels.
**Figure 14:** Tunnel deformation under different excavation methods with dewatering of jumping layer.

**Figure 15:** Comparison of deformation and theoretical value of tunnel in W3.
Figure 16: Contour of tunnel deformation in the excavation mode of W3 by dewatering of jumping layer.

Figure 17: Front view of pipe piece deformation contour at $Y = 70$ m.

Figure 18: Deformation contour of pipe piece at $Y = 55$ (m). (a) Front view of pipe piece deformation at $Y = 55$ m. (b) Side view of pipe piece deformation at $Y = 55$ m.
(2) Under the excavation mode of W3, the effect of the dewatering of jumping layer is better than that of layer-by-layer dewatering scheme in reducing the vertical deformation of the underlying tunnel, which is reduced by about 2.5%. In the actual construction, the jumping layer dewatering scheme can be adopted.

(3) The layered and segmented excavation from the middle to the two ends under the dewatering of jumping layer scheme has a better optimization effect in reducing the uplift deformation of the tunnel. In addition, the theoretical value is slightly less than the simulated value, but the deformation law is basically the same.

(4) At $Y=55$ m of tunnel length, there is a retaining structure just above the tunnel, and only lateral displacement occurs in the tunnel, in which the top of the tunnel is the smallest, the bottom of the tunnel is the largest, and the maximum value is about 1.606 mm.

(5) At $Y=70$ m of tunnel length, the tunnel is directly under the foundation pit, the tunnel only has upward deformation, and this is the location where the maximum deformation occurs, with a maximum value of about 3.054 mm. In the construction, the uplift deformation should be monitored all the time to reduce the potential safety hazard.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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