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Theoretical Analysis of Adjacent Tunnel Deformation Induced by Foundation Pit Excavation and Dewatering

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ABSTRACT: In urban areas, deep and large foundation pits often require dewatering during excavation. This process alters the free displacement field of surrounding soils, potentially causing uneven deformation in adjacent tunnels. To address this issue, the unloading of soil excavation and dewatering around the pit foundation are taken into consideration, which is clearer to actually foundation pit excavation. Firstly, utilizing the Mindlin method and the effective stress principle, the extra stress along the tunnel's axis is detected induced by the excavation and dewatering of foundation pits. Subsequently, the current tunnel is conceptualized as a Timoshenko beam supported by a three-parameter Kerr foundation, which can better take the shear deformation of the tunnel and the continuity of soil deformation into consideration. Finally, a finite difference method is applied to derive the theoretical solution for tunnel-soil interaction. The validity of the proposed model is confirmed through comparison with field monitoring data, and the results of calculation is closed to the actual measurements. Parametric analysis reveals that increasing excavation depth leads to a nonlinear rise in tunnel uplift and bending moment. A higher soil permeability coefficient results in a gradual reduction in tunnel deformation and internal forces. Enhanced tunnel bending stiffness decreases deformation but increases internal forces.

KEYWORDS: Foundation pit excavation and dewatering; adjacent tunnel; Timoshenko beam; three parameters foundation model; finite difference solution

1 Introduction

As urban soil resources dwindle, excavation endeavors around subway tunnels are consistently on the rise. However, during the excavation of fundamental pits, it's frequently essential to remove water from the pit to guarantee additional excavation. The majority of present studies mainly concentrate on how soil release during adjacent digging affects the tension and distortion of nearby tunnels [1–5], overlooking the influence of the dewatering procedure on the tunnel beneath. Elevated soil stress from dewatering in the foundation pit results in additional distortion of the adjacent tunnel, an aspect critical in practical engineering [6,7]. Consequently, examining the strain and distortion of nearby tunnels due to the digging and drying of foundation pits is an essential scientific research project grounded in practical engineering.

Recent research has delved into the effects of nearby deep foundation pits on current tunnels, employing various techniques including field measurements [8–11], centrifuge test [12,13], finite element method [14–18] and analytical techniques [19–23]. Presently, the majority of theoretical studies on the stress and deformation theory in neighboring tunnels resulting from excavation rely on the two-stage approach [24–26]. The hypothesis regarding the deformation of tunnels resulting from the digging and drying of foundational pits mainly relies on Mindlin's study [27] and the alteration in soil's effective stress to determine its free displacement. During the second phase, the tunnel undergoes free displacement and is then positioned on a foundational model with varied parameters, thus advancing the theoretical resolution of tunnel deformation. Zhang et al. [28] developed an analytical technique to measure the stress and deformation in tunnels located beneath the pit excavation using a single-parameter Winkler foundation model. Liang et al. [29] utilized theories from the Pasternak models and FEM technique to examine and derive the method for calculating interactions between the tunnel and the soil beneath the foundation pit. Zhang et al. [30] examined the nonlinear Pasternak foundation model for analyzing tunnel deformation. Liang et al. [29] utilized the Timoshenko beam model and two-parameter Pasternak foundation models, employing a dual-phase analysis approach to derive the tunnel deformation's analytical reaction. Ou et al. [31] and Zhang et al. [32], drawing from Euler–Bernoulli beam theory, examined how total dewatering at the bottom of the excavation affects the tunnels beneath, demonstrating the importance of considering dewatering in the pit. Xu et al. [33], based on the effective stress principle and the dual-parameter Pasternak models, deduced how single-well dewatering affects the stress and deformation in nearby pipelines.

In contrast to the single-parameter Winkler and Pasternak foundation models, the Kerr foundation model more accurately represents the uplift deformation in current tunnels caused by unloading excavations. Some scholars [34–36] conducted a series of analytical studies on soil–underground structure interactions based on the Kerr foundation model and demonstrated the superiority of the three-parameter Kerr foundation model. Given these conditions, it seems that the Kerr foundation model is the best approach for managing interactions between soil and tunnels. Concurrently, the majority of city subway tunnels are composed of several sequential ring-like segments, leading numerous academics to consider their shear stiffness as significant. As a result, the majority of research utilizes the Timoshenko beam, accounting for tunnel shear stiffness, to mimic tunnel deformation.

In summary, the Timoshenko beam and the three-parameter Kerr foundation model can be adopted at replicating the dynamics between tunnels and adjacent structures. Yet, present hypotheses haven't employed this method to examine how excavation and dewatering impact the stress and deformation of nearby tunnels. Consequently, the Mindlin analytical method and the effective stress principle are employed to ascertain the unbound movement of the adjacent soil. Subsequently, the tunnel's complexity is reduced to a Timoshenko beam applied to the three-parameter Kerr foundation model, utilizing the finite difference method to derive an analytical resolution for the tunnel-soil interaction. Ultimately, this research contrasts and analyzes the real engineering monitoring data with the computations outlined in this study.

2 Tunnel and Foundation Model

2.1 The Additional Stress Distribution along the Tunnel

The additional stress $q_1(x)$ due to soil unloading from excavation is derived from existing literature. This study focuses on the additional stress caused by dewatering, which has received less attention [37,38].

As shown in Fig. 1, dewatering significantly affects soil displacement. Ou et al. [31] only considered dewatering inside the pit, neglecting hydraulic connections outside. However, certain studies [32,33] deemed the dewatering process of foundation pit as the large-diameter dewatering well by using the Dupuit

assumption, and rationality about this method has been verified by *FEM* method and actually measurements, so the calculation formula of the influence of the dewatering process on the surrounding soil displacement field can be satisfied as:

$$h(r) = \sqrt{h_{\infty}^2 - (h_{\infty}^2 - h_k^2) \frac{\ln \frac{R_{\infty} + R_0}{r}}{\ln \frac{R_{\infty} + R_0}{R_0}}} \quad (1)$$

where r is the horizontal distance from any point to the pit center, h_{∞} and h_k are the initial and post-dewatering water levels, R_0 is the equivalent dewatering radius, and R is the influence radius, with $R_{\infty} = 2(h_{\infty} - h_k) \sqrt{Kh_{\infty}}$. The increase in effective stress $q_2(x)$ due to water level drawdown is:

$$q_2(x) = (h_{\infty} - h) \cdot \Delta\gamma \quad (2)$$

here, $\Delta\gamma$ as the change in effective unit weight per unit dewatering depth.

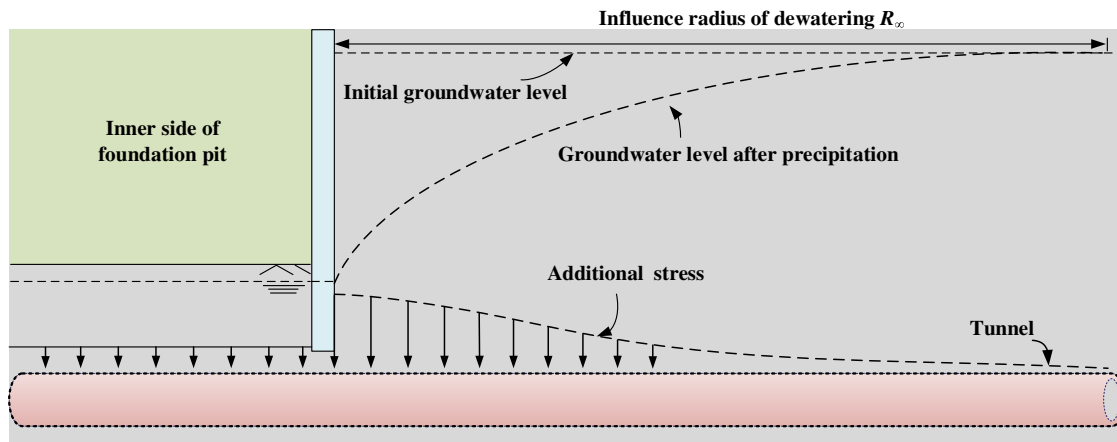


Figure 1: Influence of dewatering inside and outside the foundation pit on the underlying tunnel.

Considering the difference between inside and outside drawdown curves, the additional stress at the tunnel axis from dewatering is:

$$q_6(x) = \begin{cases} \Delta\gamma \left(h_{\infty} - \sqrt{h_{\infty}^2 - (h_{\infty}^2 - h_k^2) \frac{\ln \frac{R_{\infty} + R_0}{r}}{\ln \frac{R_{\infty} + R_0}{R_0}}} \right), & r \geq \sqrt{R_0^2 - S^2} \\ h_1 \cdot \Delta\gamma, & r < \sqrt{R_0^2 - S^2} \end{cases} \quad (3)$$

where h_1 is the vertical distance from the lowered water level inside the pit to the pit bottom.

The total additional stress along the tunnel is:

$$q(x) = q_1(x) - q_2(x) \quad (4)$$

2.2 Underlying Tunnel-Soil Interaction

Using the Kerr model and Timoshenko beam theory (Fig. 2), the governing equation for tunnel uplift deformation is:

$$\begin{aligned} \frac{d^6 w_2(x)}{dx^6} - \left(\frac{c+k}{G} + \frac{Dc}{(\kappa GA)_{eq}} \right) \frac{d^4 w_2(x)}{dx^4} + \left(\frac{Dc}{(EI)_{eq}} + \frac{Dck}{(\kappa GA)_{eq} G} \right) \frac{d^2 w_2(x)}{dx^2} \\ - \frac{Dck}{(\kappa GA)_{eq} G} w_2(x) = - \frac{Dc}{(EI)_{eq} G} q(x) + \frac{Dc}{(\kappa GA)_{eq} G} \frac{d^2 q(x)}{dx^2} \end{aligned} \quad (5)$$

where $w_2(x)$ represents the Kerr model's shear layer deformation, D denotes the tunnel's diameter, G is the shear layer's modulus, and c and k denote the stiffness of the upper and lower layer springs, respectively. The equations $(EI)_{eq}$ and $(\kappa GA)_{eq}$ represent the corresponding flexural rigidity and shear rigidity in tunnels, respectively.

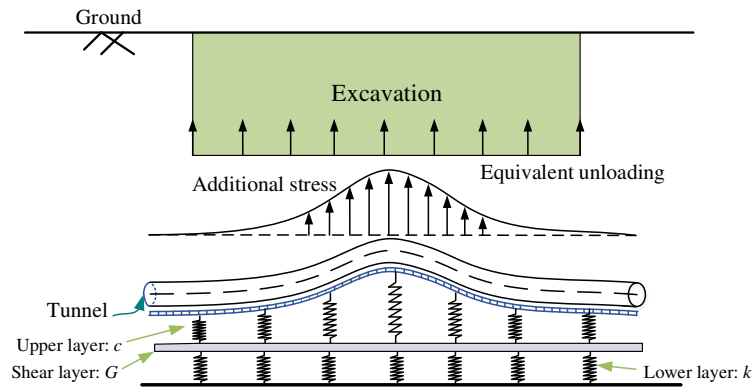


Figure 2: Kerr foundation model.

Eq. (5) represents a sixth-order differential equation resolved using the finite difference technique. As depicted in Fig. 3, the tunnel is segmented into n parts, each with a length of $l = L_1/n$. To manage boundary conditions, virtual components are incorporated at both ends.

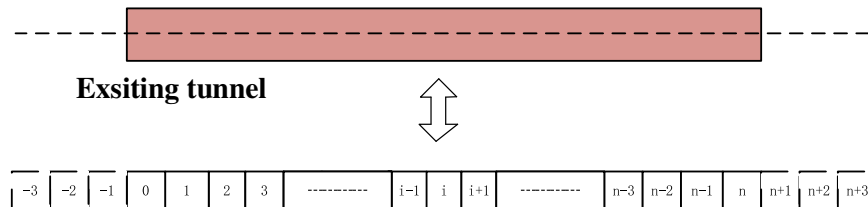


Figure 3: The tunnel discretization.

Based on finite difference method, the Eq. (5) can be rewritten as

$$\begin{aligned}
 & \frac{(w_2)_{i+3} - 6(w_2)_{i+2} + 15(w_2)_{i+1} - 20(w_2)_i + 15(w_2)_{i-1} - 6(w_2)_{i-2} + (w_2)_{i-3}}{l^6} \\
 & - \left(\frac{c+k}{G} + \frac{Dc}{(\kappa GA)_{eq}} \right) \frac{(w_2)_{i+2} - 4(w_2)_{i+1} + 6(w_2)_i - 4(w_2)_{i-1} + (w_2)_{i-2}}{l^4} \\
 & + \left(\frac{Dc}{(EI)_{eq}} + \frac{Dck}{(\kappa GA)_{eq} G} \right) \frac{(w_2)_{i+1} - 2(w_2)_i + (w_2)_{i-1}}{l^2} - \frac{Dck}{(\kappa GA)_{eq} G} (w_2)_i \\
 & = -\frac{Dc}{(EI)_{eq} G} q_i + \frac{Dc}{(\kappa GA)_{eq} G} \frac{q_{i+1} - 2q_i + q_{i-1}}{l^2}
 \end{aligned} \quad (6)$$

where $i = 0, 1, \dots, n-1, n$. $(w_2)_i$ and q_i are the deformation and unloading stress, respectively. Free-end boundary conditions imply zero shear force, bending moment, and shear layer moment at both ends. Using finite differences, Eq. (13) is discretized accordingly.

$$\begin{cases} M_{s0} = M_{sn} = - (EI)_{eq} \left. \frac{d^2 w_2}{dx^2} \right|_{i=0/n} = 0 \\ M_0 = M_n = - (EI)_{eq} \left(\left. \frac{d^2 w}{dx^2} + \frac{(q(x) - p_1(x)) D}{(\kappa GA)_{eq}} \right|_{i=0/n} \right) = 0 \\ Q_0 = Q_n = \left. \frac{dM}{dx} \right|_{i=0/n} = 0 \end{cases} \quad (7)$$

According to finite differential method, Eq. (13) can be described as:

$$(K_1 - K_2 + K_3 - K_4) \cdot w_2 = (Q_1 - Q_2 - Q_3) \quad (8)$$

where

$$w_2 = [(w_2)_0 (w_2)_1 \dots (w_2)_{n-1} (w_2)_n]^T \quad (9)$$

$$K_1 = \frac{1}{l^6} \begin{bmatrix} -6 & 14 & -10 & 2 & 0 & 0 & 0 & \dots & 0 \\ 5 & -14 & 14 & -6 & 1 & 0 & 0 & \dots & 0 \\ -4 & 14 & -20 & 15 & -6 & -1 & 0 & \dots & 0 \\ 1 & -6 & 15 & -20 & 15 & -6 & 1 & \dots & 0 \\ \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots \\ 0 & \dots & 1 & -6 & 15 & -20 & 15 & -6 & 1 \\ 0 & \dots & 0 & -1 & -6 & 15 & -20 & 14 & -4 \\ 0 & \dots & 0 & 0 & 1 & -6 & 14 & -14 & 5 \\ 0 & \dots & 0 & 0 & 0 & 2 & -10 & 14 & -6 \end{bmatrix} \quad (10)$$

$$K_2 = \frac{\chi_1}{l^4} \begin{bmatrix} T_1 & -2T_2 & T_1 & 0 & 0 & 0 & 0 & \cdots & 0 \\ 2 & 5 & -4 & 1 & 0 & 0 & 0 & \cdots & 0 \\ 1 & -4 & 6 & -4 & 1 & 0 & 0 & \cdots & 0 \\ 0 & 1 & -4 & 6 & -4 & 1 & 0 & \cdots & 0 \\ \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots \\ 0 & \cdots & 0 & 1 & -4 & 6 & -4 & 1 & 0 \\ 0 & \cdots & 0 & 0 & 1 & -4 & 6 & -4 & 1 \\ 0 & \cdots & 0 & 0 & 0 & 1 & -4 & 5 & 2 \\ 0 & \cdots & 0 & 0 & 0 & 0 & T_1 & -2T_2 & T_1 \end{bmatrix} \quad (11)$$

$$K_3 = \frac{\chi_2}{l^2} \begin{bmatrix} -E_1 & 2E_2 & 0 & 0 & 0 & 0 & 0 & \cdots & 0 \\ E_2 & -2 & 1 & -1 & 0 & 0 & 0 & \cdots & 0 \\ 0 & 1 & -2 & 1 & 0 & 0 & 0 & \cdots & 0 \\ 0 & 0 & 1 & -2 & 1 & 0 & 0 & \cdots & 0 \\ \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots \\ 0 & \cdots & 0 & 0 & 1 & -2 & 1 & 0 & 0 \\ 0 & \cdots & 0 & 0 & 0 & 1 & -2 & 1 & 0 \\ 0 & \cdots & 0 & 0 & 0 & -1 & 1 & -2 & E_2 \\ 0 & \cdots & 0 & 0 & 0 & 0 & 0 & 2E_2 & -E_1 \end{bmatrix} \quad (12)$$

$$K_4 = \begin{bmatrix} W & 0 & 0 & 0 & 0 & 0 & 0 & \cdots & 0 \\ 0 & W & 0 & 0 & 0 & 0 & 0 & \cdots & 0 \\ 0 & 0 & W & 0 & 0 & 0 & 0 & \cdots & 0 \\ 0 & 0 & 0 & W & 0 & 0 & 0 & \cdots & 0 \\ \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots & \ddots \\ 0 & \cdots & 0 & 0 & 0 & W & 0 & 0 & 0 \\ 0 & \cdots & 0 & 0 & 0 & 0 & W & 0 & 0 \\ 0 & \cdots & 0 & 0 & 0 & 0 & 0 & W & 0 \\ 0 & \cdots & 0 & 0 & 0 & 0 & 0 & 0 & W \end{bmatrix} \quad (13)$$

$$Q_1 = \frac{Dc}{(EI)_{eq} G_P} [q_0 \ q_1 \ q_2 \dots q_{n-2} \ q_{n-1} \ q_n]^T \quad (14)$$

$$Q_2 = \frac{Dc}{(\kappa GA)_{eq} G_P} \begin{Bmatrix} q_1 - 2q_0 + q_{-1} \\ q_2 - 2q_1 + q_0 \\ \vdots \\ q_n - 2q_{n-1} + q_{n-2} \\ q_{n+1} - 2q_n + q_{n-1} \end{Bmatrix} \quad (15)$$

$$Q_3 = [\Omega_1 \ \Omega_2 \ 0 \dots 0 \ \Omega_3 \ \Omega_4]^T \quad (16)$$

$$\begin{aligned}
\chi_1 &= \frac{c+k}{G_p} + \frac{Dc}{(\kappa GA)_{eq}}, \quad \chi_2 = \frac{Dc}{(EI)_{eq}} + \frac{DcK}{(\kappa GA)_{eq} G_p}, \quad T_1 = \frac{2\gamma c}{\chi_1 G_p} \\
E_1 &= \frac{Dkc}{\chi_2 (\kappa GA)_{eq} G_p}, \quad E_2 = \frac{Dc}{\chi_2 (EI)_{eq}}, \quad W = \frac{Dkc}{(\kappa GA)_{eq} G_p}, \quad \gamma = 1 + \frac{k}{c} + \frac{DG_p}{(\kappa GA)_{eq}} \\
\Omega_1 &= \frac{Dc}{(\kappa GA)_{eq} G_p l^2} [q_1 + q_0 - q_{-1}], \quad \Omega_2 = \frac{Dc}{(\kappa GA)_{eq} G_p l^2} q_0 \\
\Omega_3 &= \frac{Dcl}{(\kappa GA)_{eq} G_p l^2} q_n, \quad \Omega_4 = \frac{Dc}{(\kappa GA)_{eq} G_p l^2} [q_{n-1} + q_n - q_{n+1}]
\end{aligned}$$

When upper layer spring c ignored, the model reduces to the Pasternak model (T-P model). The Timoshenko beam will reduce to the Euler beam as $\kappa GA \rightarrow \infty$. Moreover, when the dewatering of excavation is not taken into consideration, the result of this proposed method will degrade as tunnel deformation without dewatering.

2.3 Parameters Determination of Suggested Model

To improve accuracy, Kerr model parameters are carefully selected. Feng et al. [3,4] proposed relationships between spring constants:

$$c = 7k, \quad k = 4E_s/3z_0, \quad G = 2E_s z_0/9(1 + \nu) \quad (17)$$

where E_s and ν denote the soil elastic modulus and Poisson's ratio, z_0 is the depth of tunnel centerline.

3 Verification

Ou et al. [31] reported monitoring data on the deformation of a tunnel immediately underlying and adjacent to a foundation pit during excavation and dewatering in Shenzhen. Fig. 4 illustrates the project's setup. The foundational pit measured 30 meters in length, 20 meters in width, and 8 meters in depth, with the tunnel situated right below the pit's base, aligned parallel to its width. This tunnel measured 6 meters in diameter and featured a burial depth of 14 meters. Based on the tunnel parameters and the methods described in references [39,40], the flexural rigidity and shear modulus of the tunnel were determined to be $7.55 \times 10^5 \text{ MN}\cdot\text{m}^2$ and $5.9 \times 10^3 \text{ MPa}$, respectively. The elastic modulus of the soil layer where the tunnel was located was 18 MPa, with a permeability coefficient of 1 m/d and a Poisson's ratio of 0.3. The natural unit weight and saturated unit weight of the soil were 19.9 and 20.4 kN/m³, respectively. The initial groundwater level far outside the pit was 1 m below the ground surface, and after dewatering of the foundation pit, the water level was lowered to 1 m below the base of the pit. For improved processing speed and accuracy, the tunnel has been reduced in length to 200 m, causing the sides of the central point to extend as much as 100 m.

For confirming the logic behind the suggested approach, refer to the monitoring information provided by Ou et al. [31] underwent a comparison with the outcomes derived from the suggested technique and its deteriorated solutions. As shown in Fig. 5, the tunnel heave displacement graphs, derived through different techniques, all showed patterns aligning with the true tunnel deformation, peaking at the tunnel's central point. Additionally, it's noticeable that the greatest tunnel heave displacement determined through the suggested technique is 15.3 mm, closely aligning with the recorded value of 13.7 mm. Conversely, the greatest displacements of tunnel heaves achieved with the deteriorated T-P model in the suggested technique (where the tunnel rests on a dual-parameter Pasternak base) and the approach that overlooks the impact of dewatering in the foundation pit markedly differ from actual measurements, registering at 19.3 and 23.8 mm, respectively. The hypothesis is that dewatering the foundation pit elevates the soil's effective stress, which

in turn diminishes tunnel heave. On the other hand, the Kerr model with three parameters differs from the dual-parameter Pasternak model, more precisely represents soil stress diffusion effects, yielding results more closely mirroring real engineering scenarios. Consequently, the precision of the suggested technique is confirmed by contrasting it with empirical data.

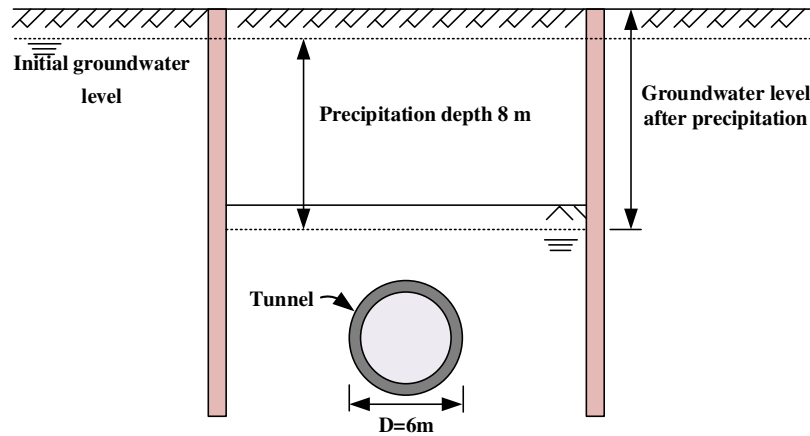


Figure 4: Relative location of the underground passage and tunnel.

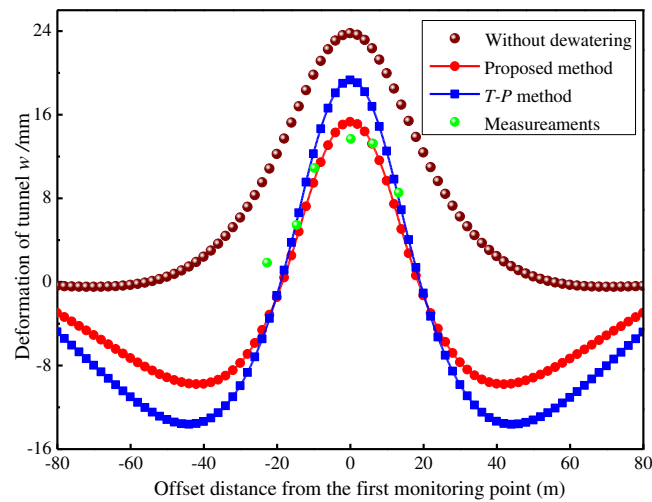


Figure 5: Prediction comparison between the suggested method and measurements in Shenzhen project.

4 Discussion

To investigate the effects of depth of excavation, soil permeability coefficient, and tunnel bending stiffness on the adjacent tunnels, a parameter analysis based on the single-variable method was conducted within the context of the Shenzhen Metro project.

4.1 Depth of Excavation

Fig. 6 shows the heave deformation and bending moment curves of the existing tunnel under the influence of adjacent excavation and dewatering when the excavation depth varies. This is visible in Fig. 6, with the gradual deepening of the excavation, there's a steady rise in both the tunnel's heave displacement and bending moment, accompanied by an accelerating rate of increase. This is because a greater excavation depth

leads to additional unloading of the excavated soil, thereby increasing the additional stress at the tunnel axis. Furthermore, with the diminishing gap between the tunnel and the excavation, the extra strain on the nearby tunnel escalates progressively.

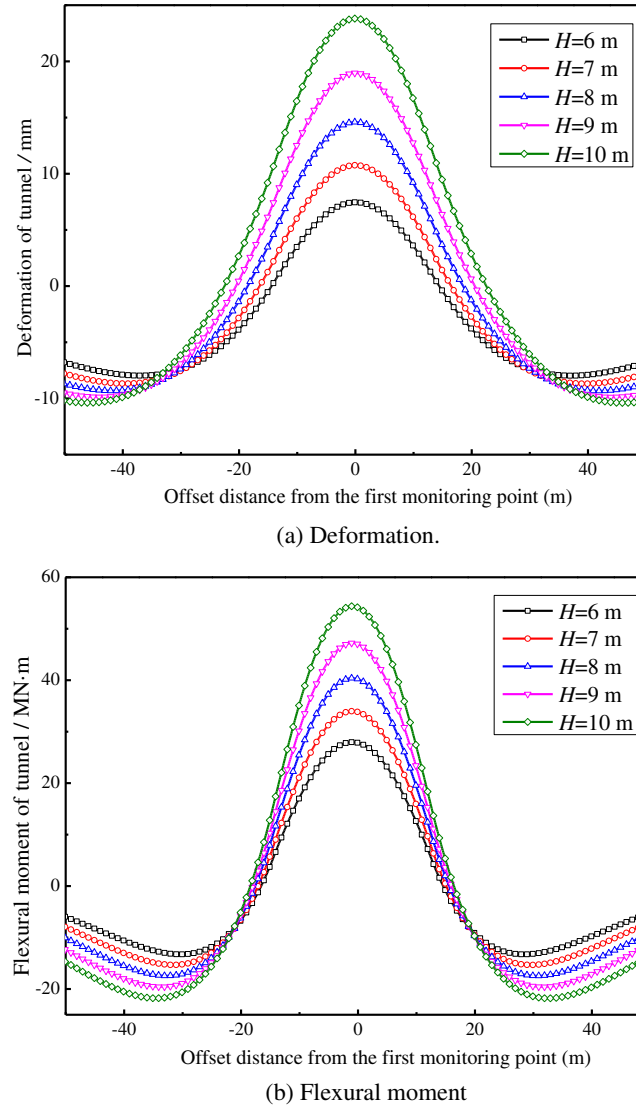


Figure 6: Influence curve of excavation depth on tunnel-soil interaction.

4.2 Permeability Coefficient of Soil

Fig. 7 illustrates how changes in the soil permeability coefficient affect the tunnel's peak displacement and bending moment, which are caused by digging and drying processes. This is observable in Fig. 7 indicates that with a steady rise in the soil permeability coefficient, the tunnel's peak displacement diminishes from 16.1 to 14.1 mm, marking a 12.4% decrease in this peak displacement. Likewise, the tunnel's maximum bending moment diminishes from 44.1 to 36.6 MN·m, marking a 17% decrease in the peak bending moment. Concurrently, it is noticeable that the speed at which the tunnel's peak displacement and bending moment decrease steadily stabilizes. The reason is that with the rise in the soil permeability coefficient, there's a gradual reduction in the scope of dewatering effects. Nonetheless, the extra stress caused by dewatering at the center

of the tunnel shows no notable reduction. Consequently, the impact of alterations in the soil permeability coefficient on the tunnel's response slowly lessens.

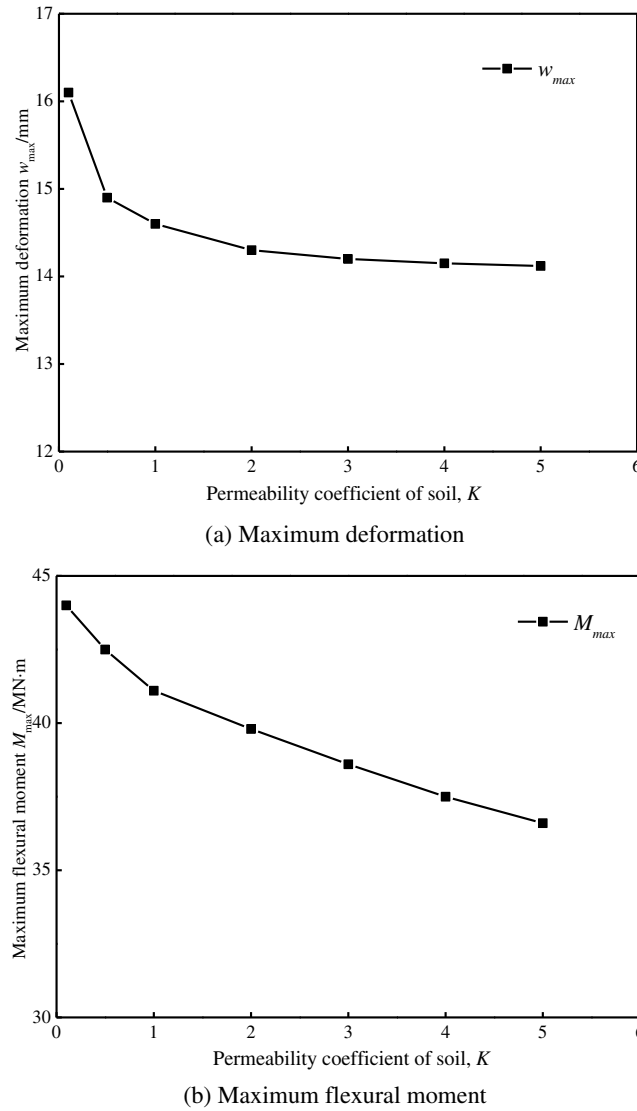
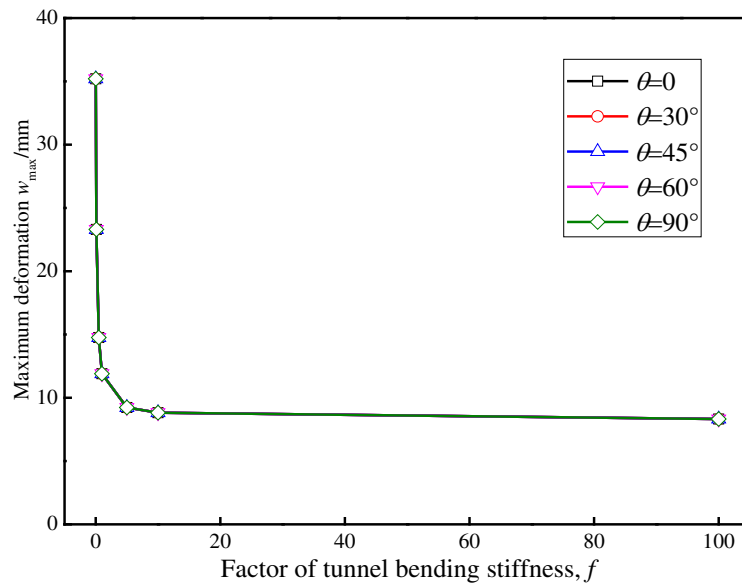


Figure 7: Influence curve of soil permeability coefficient on tunnel-soil interaction.

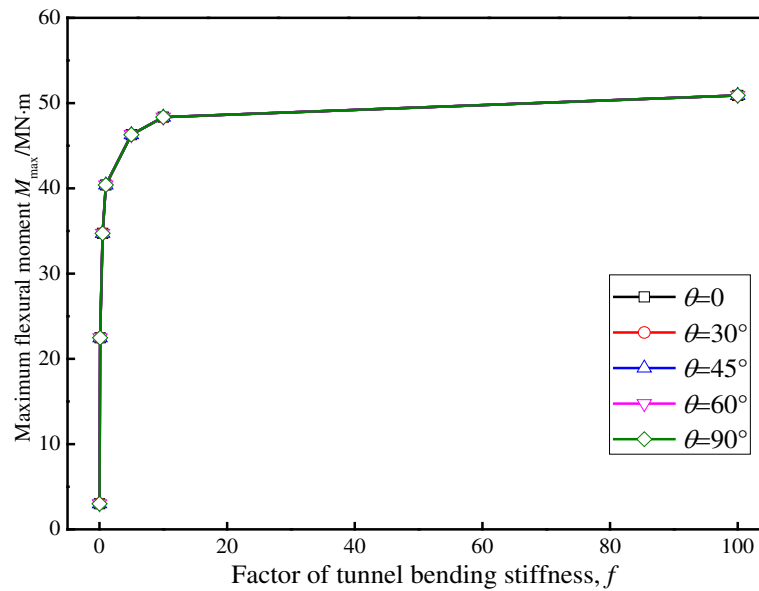
4.3 Tunnel Bending Stiffness and the Angle between the Excavation Center and the Tunnel

Fig. 8 illustrates how the rigidity of tunnel bending and the angle between the excavation site and the tunnel affect the tunnel's peak displacement and bending moment caused by excavation and dewatering. In Fig. 8, f represents the assumed factor by which the tunnel bending stiffness is a multiple of that of the existing tunnel in the Shenzhen case, with f taking values of 0.01, 0.1, 0.5, 1, 5, 10, and 100. It can be observed from Fig. 8 that as the tunnel stiffness rapidly increases, the maximum peak displacement of the tunnel gradually decreases from 35.2 to 8.3 mm, a reduction of 76.4% in the peak displacement. However, the peak bending moment of the tunnel gradually increases from 3.2 to 50.9 MN·m, an increase of nearly 15 times in the peak bending moment. The reason is that heightened stiffness improves the tunnel's deformation resistance, leading to reduced tunnel deformation but heightened internal forces. Meanwhile, it can also be

seen from Fig. 8 that changes in the angle between the excavation center and the tunnel do not alter the peak values of stress and deformation of the tunnel. This may be because a change in the angle cannot effectively modify the additional stress at the tunnel center point. Therefore, the influence of the angle on the stress and deformation of the tunnel is very limited.



(a) Maximum deformation



(b) Maximum flexural moment

Figure 8: Influence curve of the bending stiffness of the tunnel and the angle between the center of the foundation pit and the tunnel on the tunnel-soil interaction.

5 Conclusions

A refined method is proposed to detected tunnel-soil interaction due to overlying excavation and dewatering. The main findings are:

- (1) The combined effect of excavation and dewatering on additional stress at the tunnel axis is considered. Using a Timoshenko beam on a Kerr foundation, a theoretical solution is derived via the Fourier series method.
- (2) Compared to models that ignore dewatering or use the degraded T-P model, the proposed method yields tunnel displacement predictions in better agreement with field measurements.
- (3) Increasing excavation depth nonlinearly amplifies tunnel heave and bending moment. Higher soil permeability reduces both deformation and bending moment gradually. Greater tunnel bending stiffness decreases deformation but increases internal forces. The angle between the pit center and tunnel has negligible impact on peak tunnel deformation.
- (4) The proposed model assumes greenfield conditions, but the nearby structures significantly distort the groundwater drawdown funnel and inward movement of the pit's retaining wall are not taken into consideration, which would alter the input free-field displacement for this proposed method.

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Availability of Data and Materials: All data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

Ethics Approval: Not applicable.

Conflicts of Interest: The authors declare no conflicts of interest.

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