

Article

A Novel Prediction Model for Estimating Ground Settlement Above the Existing Tunnel Caused by Undercrossing

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Abstract

A new tunnel undercrossing an existing tunnel not only affects the deformation and stress response of the existing tunnel but also triggers ground settlement due to secondary excavation disturbances. By combining the equivalent layer method with the mirror method and incorporating corrections from numerical simulations based on actual intersection projects, a novel prediction model is developed to consider the impact of the existing tunnel on estimating ground settlement caused by a new tunnel that undercrosses it in an orthogonal manner. The influence of geological conditions, tunnel dimensions, and spatial layout on ground settlement patterns was investigated. The elastic moduli of smaller strata correlate with greater surface settlement. Larger existing tunnel diameters result in reduced settlement within a 15 m area near the new tunnel axis. Conversely, new larger tunnel diameters yield more pronounced settlement. A consistency assessment method was introduced to quantitatively measure the consistency between the prediction model and numerical simulations. The results indicate that the prediction model exhibits high consistency ($CI > 0.9$) under various conditions. Based on an actual engineering case, indoor similarity model tests were designed. When the new tunnel is directly located beneath the existing tunnel, ground settlement begins, with a maximum settlement of 0.17 mm. After the new tunnel traversed the existing one, ground settlement continued to increase within approximately 50 m on both sides of the new tunnel's axis, ultimately reaching a value of about 0.765 mm. The CI between the predictive model and the model test results reached 0.8, confirming the model's robust predictive capability.

Keywords: prediction model; ground settlement; tunnel undercrossing; parameter analysis; adaptability study; case verification



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1. Introduction

With the continuous development of urban underground space and the increasing maturity of railway transportation construction technology, the phenomenon of overlapping tunnels in urban underground environments has become increasingly common. By the end of 2024, the total length of urban rail transit in China had exceeded 10,000 km, with approximately 78% of the lines being underground, often involving complex situations such as overlapping tunnels [1]. Tunnel excavation causes redistribution of loads in the surrounding soil, disturbs the adjacent soil, and leads to ground deformation [2]. However, the presence of existing tunnels above new tunnels inevitably affects the propagation of

disturbances induced by excavation, thereby altering the deformation patterns of the strata, which warrants further investigation.

Currently, research on constructing new tunnels beneath existing tunnels primarily focuses on the impact on the deformation of existing tunnels, including the patterns of deformation changes and settlement behavior [3–5]. The construction of a new tunnel undercrossing an existing tunnel can cause settlement in the existing tunnel, and the settlement range and maximum settlement value are closely related to the angle of intersection [6]. Fang et al. (2015) found that new double-line tunnels cause vertical deformation of existing tunnels to exhibit a “W”-shaped distribution [7]. Feng et al. (2024) found that the deformation of existing tunnels caused by shield tunneling can be accurately predicted using the BO-LGBM model. They identified key influencing factors such as grouting volume and thrust force [8]. Dong et al. (2025) found that ground settlement caused by shield tunneling under existing tunnels in composite soil layers exhibits specific lateral and longitudinal patterns and is significantly influenced by construction parameters [9]. Based on existing engineering data and research results, analytical methods are widely used to predict the deformation of existing tunnels caused by tunnel undercrossing. Liu et al. (2021) proposed a simplified two-stage analysis method that considers tunnel-to-tunnel interaction, which can be used to assess the impact of double-line tunnels undercrossing on the deformation of the existing tunnel above [10]. Li et al. (2024) integrated the equivalent layering method with the Peck formula to predict the deformation of existing tunnel structures under multi-factor conditions [11]. Yao et al. (2024) proposed a WOA-LSTM model for predicting and optimizing tunnel rock mass deformation under high-ground-stress conditions [12]. Chang et al. (2024) proposed a modified Peck formula for assessing the deformation of existing tunnels during the construction of multi-line tunnels [13].

When constructing new tunnels under existing tunnels, it is necessary not only to consider the deformation and stress response of the existing tunnels but also to pay attention to ground settlement caused by secondary excavation disturbances. However, research in this field remains relatively limited. Additionally, the inherent differences between tunnel structures and surrounding geological formations make it highly challenging to precisely quantify the impact of new tunnel excavation on ground settlement patterns beneath existing tunnels. Therefore, current research primarily relies on numerical simulations, laboratory experiments, and analysis of field monitoring data [14–16]. Additionally, calculations of ground settlement caused by new tunnel construction often neglect the presence of existing tunnels, leading to significant discrepancies in the results [17]. This oversight stems from two key factors: first, the stiffness of the tunnel lining is significantly higher than that of the surrounding rock, thereby affecting rock deformation; second, the hollow cylindrical structure of the lining introduces non-uniformity, which has a substantial impact on rock settlement. Zhou et al. (2020) proposed a method for calculating ground settlement caused by tunnel crossing by combining the equivalent layer method with the classic Peck formula [17]. However, since the Peck formula is an empirical formula based on statistical experience, its reliability under different engineering conditions has not been fully verified.

The core of accurate ground settlement prediction in tunnel crossing projects lies in the reasonable modeling of the interaction between the existing tunnel and the surrounding strata. Zhou et al. (2020) ingeniously transformed the existing tunnel into equivalent strata using the equivalent layer method, providing an effective means to simplify this complex challenge [17]. Inspired by this framework, the preliminary work of this study proposed a calculation method combining the equivalent layer method with the mirror method to predict vertical deformation in existing tunnels [18]. However, the effectiveness

of this model in predicting ground settlement directly above existing tunnels caused by new tunnel construction remains an unsolved issue.

Based on actual engineering, the ground settlement directly above the existing tunnel was calculated using the calculation method from our preliminary study [18]. The model's errors were demonstrated using measured data and numerical simulations. Subsequently, based on the differences between the preliminary model and the numerical simulation results, an improved model for predicting ground settlement was proposed. Using the modified prediction model, multiple sets of different calculation conditions were designed to investigate the influence of various parameters on ground settlement patterns. The adaptability of the model was evaluated, and the results indicated that it is highly effective in predicting ground settlement caused by the orthogonal crossing of new tunnels and existing tunnels. Finally, based on an actual engineering case, indoor similarity model tests were designed to validate the practical application value of the modified prediction model.

2. Theoretical Basis and Preliminary Model Construction

2.1. The Equivalent Layer Method

The equivalent layer method was introduced to convert non-uniform strata into uniform equivalent strata by replacing the elastic modulus and density of the upper strata with the corresponding parameters of the lower strata, and it has been widely applied and further developed [11,19,20]. The depth of the upper layer should be calculated as follows:

$$h_1' = h_1 \sqrt{\frac{E_1 \rho_2}{E_2 \rho_1}} \quad (1)$$

where h_1 is the thickness of the upper layer; h_1' is the equivalent thickness of the upper layer obtained using the equivalent layer method; E_1 and E_2 are the elastic moduli of the upper and lower layers, respectively; ρ_1 and ρ_2 are the densities of the upper and lower layers, respectively. These parameters are shown in Figure 1.

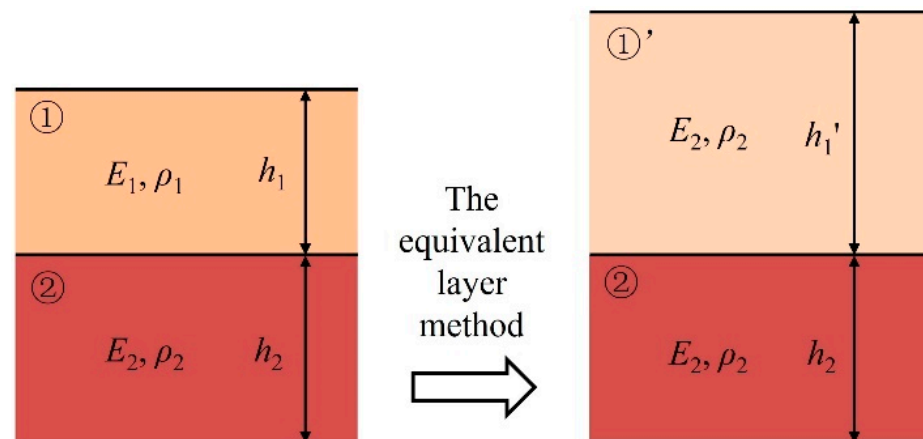


Figure 1. Parameters of the equivalent layer method (① means the upper layer, ② means the lower layer, through the equivalent layer method, the parameter of the upper layer has been changed, and indicated by ①').

It should be noted that the equivalent layer method assumes homogeneous strata, disregarding spatial variability in stratigraphic properties, and is thus an idealized and simplified calculation method.

2.2. The Mirror Method

The mirror method was proposed by Verruijt and Booker (1996) to calculate rock mass deformation caused by tunnel excavation within a single homogeneous layer [21]. Its linear elastic analytical solution was derived to determine rock mass displacement, accounting for the variability in volume changes. Subsequently, González and Sagaseta (2001), Park (2005), Tan et al. (2019), and Li et al. (2023) further utilized the mirror method to study ground deformation caused by tunnel excavation, thereby validating the effectiveness of this method in assessing excavation-induced ground deformation [22–25]. Verruijt and Booker (1996) [21] proposed an expression for calculating the displacement at a specific point in the stratum resulting from tunnel excavation using the mirror method. The calculation formula is as follows:

$$U_z = -\varepsilon R^2 \left(\frac{z_1}{r_1^2} + \frac{z_2}{r_2^2} \right) + \delta R^2 \left[\frac{z_1(kx^2 - z_1^2)}{r_1^4} + \frac{z_2(kx^2 - z_2^2)}{r_2^4} \right] + \frac{2\varepsilon R^2}{m} \left[\frac{(m+1)z_2}{r_2^2} - \frac{mz(x^2 - z_2^2)}{r_2^4} \right] - 2\delta R^2 H \left[\frac{x^2 - z_2^2}{r_2^4} + \frac{m}{m+1} \frac{2zz_2(3x^2 - z_2^2)}{r_2^6} \right] \quad (2)$$

$$U_x = -\varepsilon R^2 x \left(\frac{1}{r_1^2} + \frac{1}{r_2^2} \right) + \delta R^2 x \left[\frac{(x^2 - kz_1^2)}{r_1^4} + \frac{(x^2 - kz_2^2)}{r_2^4} \right] - \frac{2\varepsilon R^2 x}{m} \left[\frac{1}{r_2^2} - \frac{2mzz_2}{r_2^4} \right] - \frac{4\delta R^2 x H}{m+1} \left[\frac{z_2}{r_2^4} + \frac{mz(x^2 - 3z_2^2)}{r_2^6} \right] \quad (3)$$

where U_z represents the vertical deformation at a certain point in the stratum caused by tunnel excavation; U_x represents the horizontal deformation at the same point in the stratum caused by tunnel excavation; $z_1 = z - H$; $z_2 = z + H$; $r_1^2 = x^2 + z_1^2$; $r_2^2 = x^2 + z_2^2$; $m = \frac{1}{1-2\mu}$; $k = \mu(1 - \mu)$; x and z are the coordinates of the calculation point; R is the tunnel radius; H is the axial depth of the tunnel; μ is the Poisson's ratio of the soil; ε and Δ represent the uniform radial contraction and elliptical deformation of the tunnel, respectively, which can be calculated using the following formulas according to Verruijt and Booker (1996) [21]:

$$\varepsilon = \frac{P_0 - P_i}{2G} \left[1 + 2(1 - \mu) \frac{2\frac{H}{R} \left(\frac{H}{R} - \sqrt{\left(\frac{H}{R} \right)^2 - 1} \right) - 1}{1 - \frac{H}{R} \left(\frac{H}{R} - \sqrt{\left(\frac{H}{R} \right)^2 - 1} \right)} \right] \quad (4)$$

$$\delta = \frac{P_0 - P_i}{2G} (1 - \mu) \frac{1}{\left(\frac{H}{R} \right)^2 - 1} \quad (5)$$

where P_0 is the vertical ground stress at the tunnel axis before excavation; P_i is the support stress after excavation; G is the shear modulus of the soil.

It should be noted that Equations (4) and (5) represent empirical calculation formulas for ε and Δ . Their theoretical analytical solutions and derivation processes can be found in reference [21]. Further elaboration is omitted here.

The above parameters are shown in Figure 2.

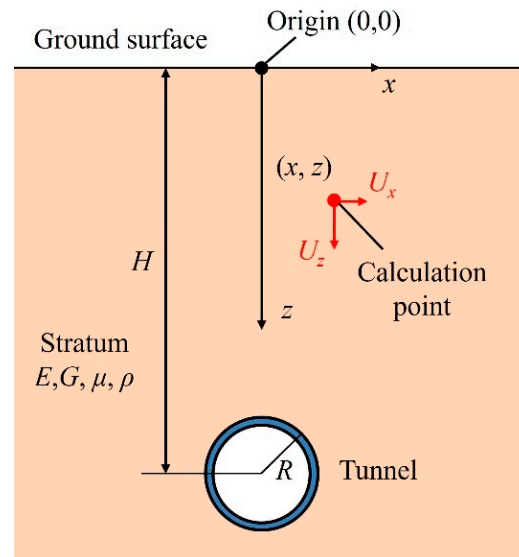


Figure 2. Parameters of the mirror method.

2.3. Preliminary Construction of the Ground Settlement Prediction Model

Based on the equivalent layer method and mirror method, a calculation model for vertical deformation of existing tunnels caused by new tunnels undercrossing was proposed in our previous research [18]. The primary implementation process of this method is as follows:

1. Convert existing tunnels into equivalent strata;
2. Homogenize multi-layer strata, including strata derived from existing tunnels through the equivalent layer method;
3. Calculate the settlement of the existing tunnel layer caused by the excavation of new tunnels in homogenized strata using the mirror method.

The following formula is used to convert existing tunnels into strata:

$$E' \frac{b_0 h_0^3}{12} = E \left[\frac{\pi D^4}{64} (1 - \alpha^4) \right] \quad (6)$$

$$\rho' b_0 h_0 = \rho \frac{\pi}{4} (D^2 - d^2) \quad (7)$$

where E' is the elastic modulus of the equivalent stratum; b_0 and h_0 are the width and height of the equivalent stratum, respectively, and $b_0 = h_0 = D$; D is the outside diameter of the existing tunnel; E is the elastic modulus of the tunnel lining; $\alpha = \frac{d}{D}$, where d is the inside diameter of the existing tunnel; ρ' is the density of the equivalent stratum; ρ is the density of the lining.

After converting the existing tunnel into a stratum, multiple layers of strata with different properties may still exist above the new tunnel. Subsequently, these various layers of strata are homogenized into a single equivalent stratum through the equivalent layer method using the following formula:

$$h = \sqrt{\frac{\rho_n}{E_n} \sum_{i=1}^{n-1} h_i \sqrt{\frac{E_i}{\rho_i}}} \quad (8)$$

where h is the total thickness of the former $n - 1$ layers; ρ_n is the density of the n th layer; E_n is the elastic modulus of the n th layer; h_i is the thickness of the i th layer; ρ_i is the density of the i th layer; and E_i is the elastic modulus of the i th layer.

Finally, the mirror method is used to calculate the settlement of the existing tunnel layer caused by the new tunnel excavation, as per Equation (2). It should be noted that all calculation parameters in Equation (2) are converted parameters.

When z in Equation (2) is equal to 0, the formula becomes:

$$U_z = \frac{2\varepsilon R^2(m+1)}{m} \left(\frac{h'}{r^2} \right) - 2\delta R^2 h' \left(\frac{x^2 - h'^2}{r^4} \right) \quad (9)$$

where $r^2 = x^2 + h'^2$; the remaining parameters are the same as those mentioned above.

Based on Equation (9), a preliminary prediction model was established to estimate ground settlement directly above the existing tunnel caused by the construction of a new tunnel.

It is crucial to emphasize that, as an idealized analytical calculation model, Equation (9) does not account for the potential stress distribution irregularities and settlement variations caused by spatial variations in soil parameters. Particularly in weak or heterogeneous strata, localized weak zones may exacerbate deformation near the crossing section [26].

3. Verification

To validate the practical applicability of the preliminary prediction model, a representative case from Liang (2016) [27] was selected, which involves the orthogonal intersection of a new tunnel and an existing tunnel. Due to the lack of monitored ground settlement data, these values were derived through numerical simulation. The other monitoring data from Liang (2016) [27] were used to verify the accuracy of the simulation.

3.1. Numerical Simulation

The numerical simulation model was constructed using the FEM software Midas-GTS NX 2021 [28] and then imported into the FDM software FLAC^{3D} 5.0 [29] for numerical calculations. Figure 3 shows the main dimensions and spatial relationships of the tunnels. As shown in the figure, the existing tunnel is buried at a depth of 9.9 m, with a minimum clearance of only 1.85 m between the new and existing tunnels. This layout is highly likely to significantly impact the existing tunnel and surrounding rock layers during excavation. Both tunnels have a diameter of 6.0 m, with the lining consisting of shield segments that are 0.3 m thick and 1.5 m wide.

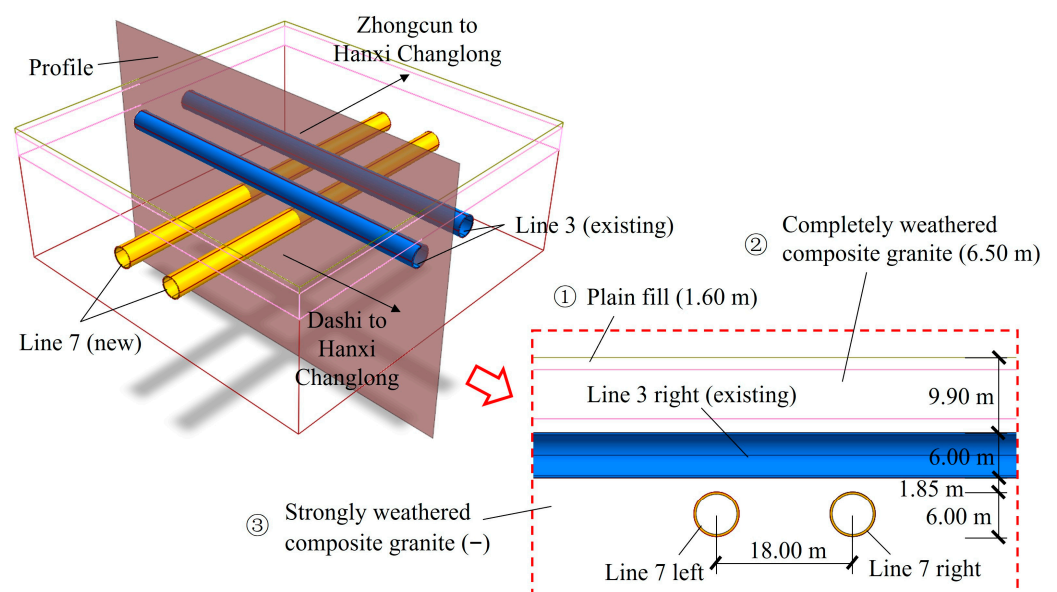


Figure 3. Location and size of the existing and new tunnels.

Given that the two lines of Line 7 were constructed independently, and that the right line of Line 7 was excavated before the left line and was not affected by it, the numerical model focuses solely on the engineering of the right line of Line 7 crossing the right line of Line 3. In subsequent analyses, the right line of Line 7 will be marked as the new tunnel, while the right line of Line 3 will be referred to as the existing tunnel.

To minimize boundary effects, the model's length and width were extended to 3 to 5 times the diameter of the tunnel. Considering the shallow bedrock surface, a model height of 40.0 m was sufficient. The final model dimensions were 51.0 m $\times$ 51.0 m $\times$ 40.0 m (length $\times$ width $\times$ height). A hierarchical mesh approach was adopted, where finer elements (0.5 m grid size) were assigned near the new tunnel lining and excavation face. In comparison, existing tunnel sections utilized a 1.5 m grid size. Coarser elements (maximum 3.0 m) were applied to the outer model region to optimize computational efficiency. Mesh refinement was determined through convergence studies, ensuring that changes in ground settlement and tunnel deformation remained below 2% when element sizes were further reduced, thereby meeting the geotechnical simulation accuracy requirements [2]. The final numerical model comprises 79,145 elements and 45,598 nodes. The established 3D simulation model is shown in Figure 4.

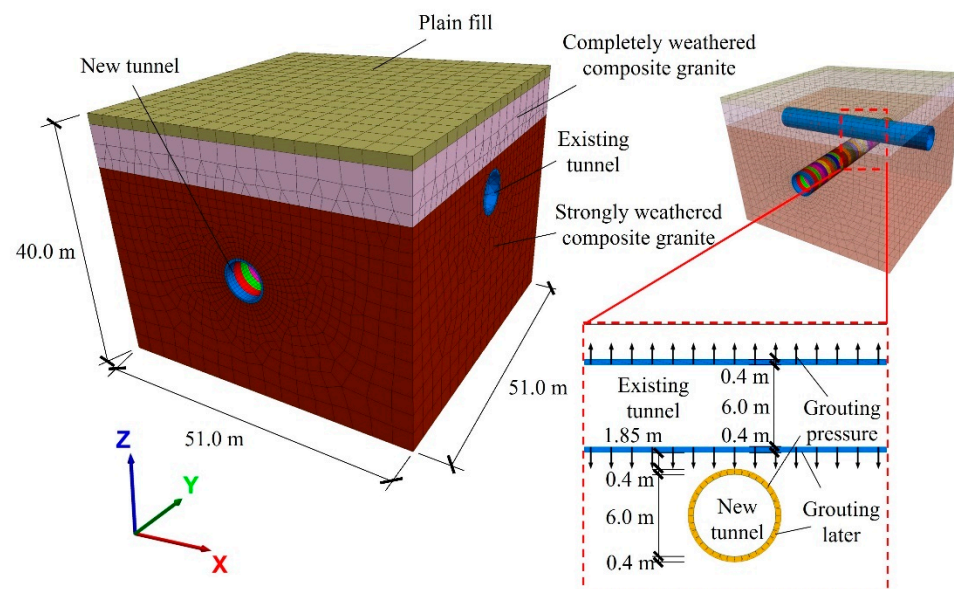


Figure 4. The established 3D simulation model.

Boundary conditions are defined as follows: lateral boundaries are fixed to prevent lateral deformation, simulating a semi-infinite medium; the bottom boundary is fully fixed to restrain vertical and horizontal displacements; and the top boundary is unconstrained. Following Liang's methodology [27], a uniform 0.2 MPa load was applied to the excavation face to simulate shield support forces. An outward radial pressure of 0.3 MPa was applied to the outer wall of the 0.4-meter-thick grout layer to simulate grouting pressure, consistent with shield tunnel construction simulations [5].

The Mohr-Coulomb (M-C) model features clear parameter definitions, straightforward parameter acquisition, and high computational efficiency. It is particularly well-suited for describing the mechanical behavior of weathered granite formations and demonstrates excellent applicability in numerical simulations of tunnel undercrossing. Consequently, it is widely adopted as the constitutive model for rock in tunnel numerical simulations [30–32]. Therefore, the M-C model was employed for computations, with simulation parameters detailed in Table 1.

Table 1. Physical and mechanical parameters of the strata in simulation.

Layer	Soil	Density $\rho/(\text{kg/m}^3)$	Elastic Modulus E/MPa	Friction Angle $\varphi/^\circ$	Cohesion c/kPa	Poisson's Ratio $\mu/-$	Strength of Extension T/MPa	Thickness h/m
①	Plain fill	1880	6.0	10.0	15.0	0.38	0.0	1.6
②	Completely weathered composite granite	1890	27.0	21.0	28.0	0.28	8.0	6.5
③	Strongly weathered composite granite	1940	30.0	26.7	40.0	0.25	10.0	-

The shield lining segment is constructed using C50 concrete and simulated via shell elements. The grout layer is modeled using solid elements and an elastic constitutive model. Since no relative displacement occurs between the grout layer and the surrounding strata, no contact interface was defined, as outlined in [5]. The interaction between the lining and grout layer is reflected through the computational parameters of the shell elements. The relevant parameters are shown in Table 2.

Table 2. Calculation parameters of the lining segments and grouting layer.

Materials	Thickness t/m	Density $\rho/(\text{kg/m}^3)$	Elastic Modulus E/MPa	Poisson's Ratio $\mu/-$
Lining segments	0.3	2500	34,500	0.25
Grouting layer	0.4	2300	1000	0.25

The simulation primarily focuses on ground settlement above the existing tunnel caused by the excavation of the new tunnel. Therefore, the existing tunnel was excavated in a single operation, while the new tunnel was excavated in sections, with each section having the same width as the actual section width. The excavation step length for the new tunnel is 1.5 m, and the grouting layer was activated after each segment was excavated. To accurately capture ground settlement caused solely by the excavation of the new tunnel, all initial displacements were reset to zero before the new tunnel excavation began.

The detailed simulation process is as follows:

1. Initialize the ground stress equilibrium and reset the initial displacement;
2. Excavate the existing tunnel at one time, install the lining segments, define the grouting layers, and simultaneously apply grouting pressure; calculate and reset the displacement;
3. Excavate the new tunnel in stages, install the lining segments in each step, define the grouting layers, and apply grouting pressure until completion;
4. Extract the ground settlement directly above the existing tunnel.

3.2. Verification of the Simulation

The accuracy of the simulation was verified using vertical deformation data monitored in existing tunnels. Seventeen monitoring sections were set up for each line, with five measurement points in each section. Figure 5 shows the layout of the monitoring sections

for the right line of Line 3, where sections R11, R12, and R13 are located directly above the underpass section.

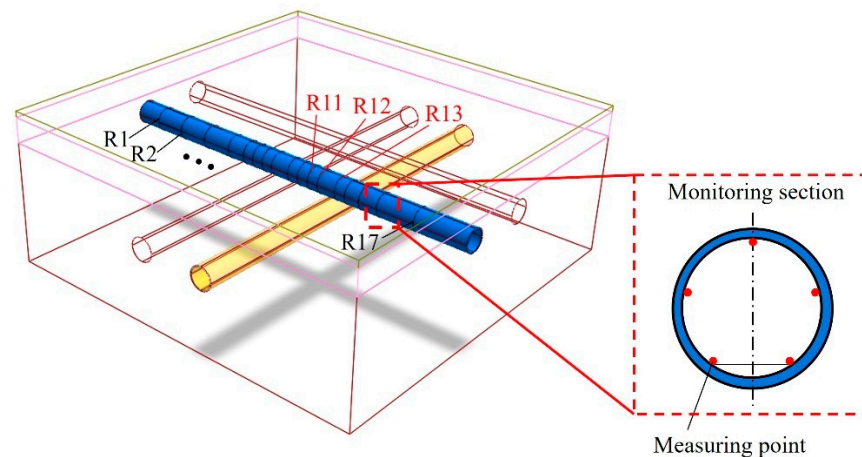


Figure 5. The monitoring section layout of the right line of Line 3.

Figure 6a presents a comparison between the numerical simulation results and monitoring data along the existing tunnel's axis direction. In contrast, Figure 6b displays the corresponding data from measurement points R11, R12, and R13 during the excavation process. In Figure 6b, the excavation steps in the numerical simulation have been converted to the actual excavation timeline to ensure alignment. As shown in the figure, the vertical deformation of the existing tunnel simulation is highly consistent with the monitoring values along the tunnel axis direction and throughout the entire excavation sequence. The maximum vertical deformation occurs directly above the crossing section, then gradually decreases toward points farther from this area until it approaches zero, forming a “trench-like” settlement curve. For sections R11, R12, and R13, vertical deformation suddenly increased from 0 to approximately 2.5–3.5 mm during the construction phase of the crossing section (19–22 August), followed by settlement stabilizing at around 3.5–4.5 mm. The primary source of differences in Figure 6a stems from the simulation's failure to account for complex loads, such as ground loads, vehicle loads within the existing tunnels, and construction loads [18,33].

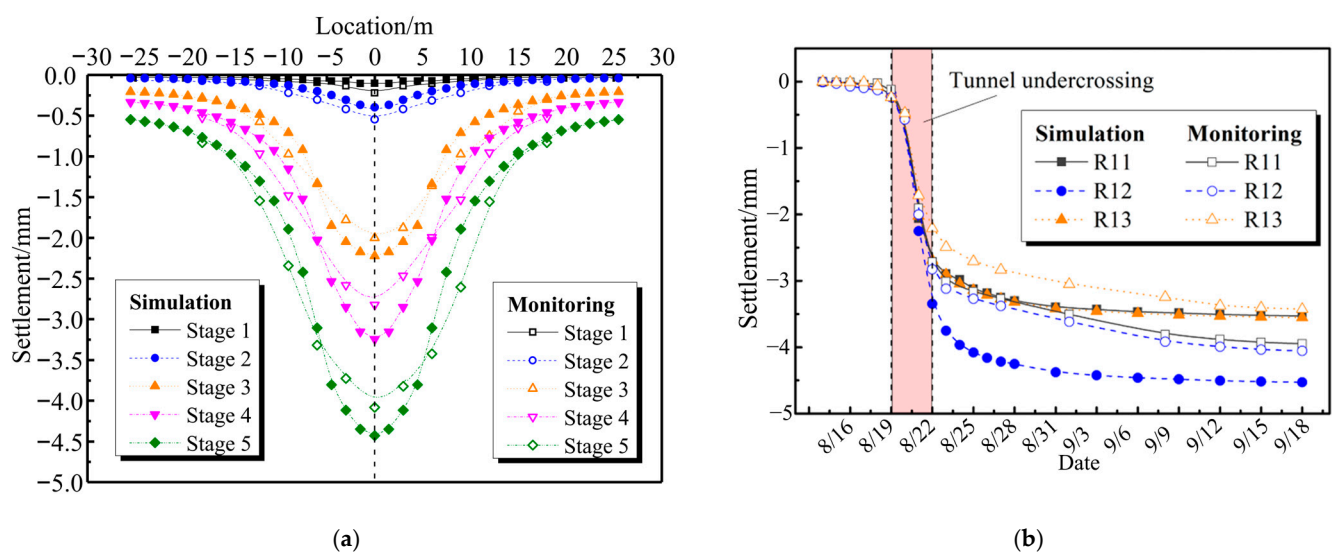


Figure 6. Comparison between the numerical simulation and monitoring: (a) Along the existing tunnel axis. (b) R11, R12, R13 during excavation.

3.3. Verification of the Prediction Model

After verification with monitoring data, the numerical simulation model was used to assess the accuracy of the preliminary prediction model. Ground settlement directly above the existing tunnel was extracted and compared with the calculation results through the preliminary prediction model, as shown in Figure 7. To obtain the calculation results through the preliminary prediction model, the equivalent elastic modulus E' and equivalent density ρ' were first calculated using Equations (6) and (7), followed by the calculation of ground settlement directly above the existing tunnel using Equations (4), (5), (8), and (9). The calculation range extends 25.5 m on either side of the new tunnel axis, with adjacent calculation points spaced 1.5 m apart, corresponding to the width of the shield segment.

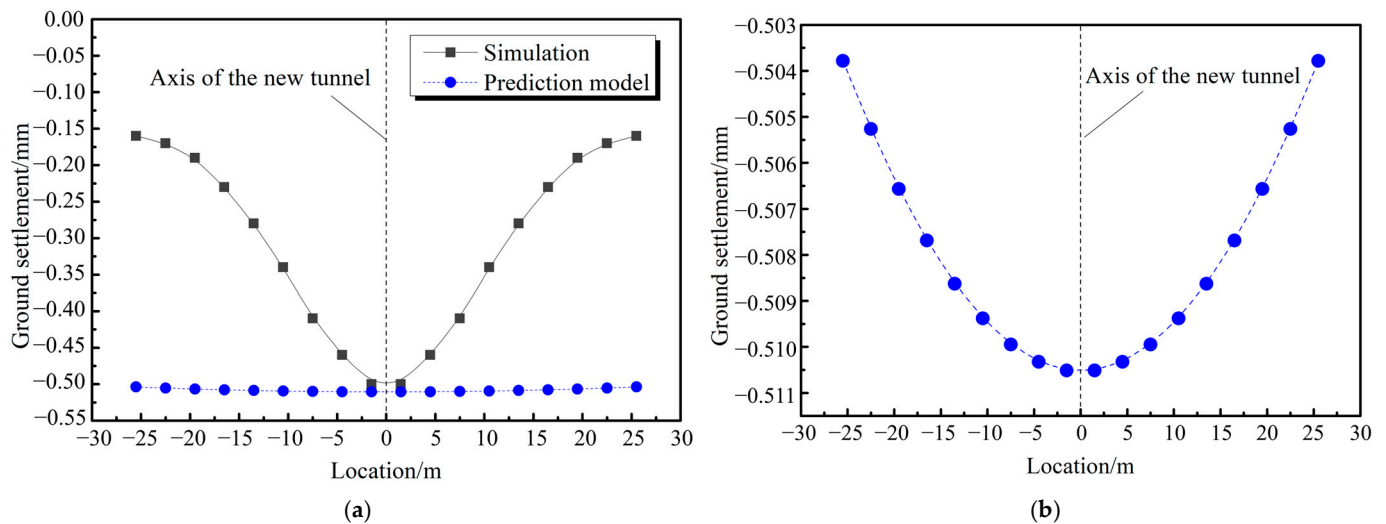


Figure 7. Comparison of settlement curves obtained by numerical simulation and the preliminary prediction model: (a) Comparison. (b) The preliminary prediction model.

As shown in the figure, the ground settlement curves obtained through the preliminary prediction model exhibit poor consistency with the numerical simulation results. Although the values near the new tunnel axis are similar, the overall trend is significantly smoother. To facilitate analysis, Figure 7b presents the ground settlement curve from the preliminary prediction model separately, indicating a gradual decrease from 0.511 mm at the axis to 0.504 mm at 25.5 m, corresponding to a change of approximately 1.37%. This discrepancy primarily stems from the preliminary prediction model equating the existing tunnel with the surrounding strata using the stiffness equivalence method, thereby overestimating the stiffness difference between the tunnel and the strata, as well as the overall coordinated deformation capacity of the tunnel lining. Therefore, the model requires modification to enhance its predictive accuracy for ground settlement.

4. Modification of the Prediction Model

Peck (1969) proposed that the ground settlement curve caused by tunnel excavation follows a normal distribution—a concept that has since been widely adopted and refined [11,34–36]. The basic form of the Peck formula is:

$$S = S_{\max} e^{-\frac{x^2}{2i^2}} \quad (10)$$

$$S_{\max} = \frac{V_s}{i\sqrt{2\pi}} \approx \frac{0.313V_l D^2}{i} \quad (11)$$

where S is the ground settlement at any location; $S_{\max}$ is the maximum ground settlement directly above the tunnel axis; v_s is the volume of ground loss per unit length; V_l is the

ground loss rate; D is the tunnel diameter; i is the horizontal distance from the tunnel axis to the inflection point of the settlement curve; and x is the horizontal distance from the tunnel axis to any point on the ground surface.

O'Reilly and New (1982) proposed the following formula to describe the relationship between i and the depth of the tunnel center z [37]:

$$i = Kz \quad (12)$$

where K is the width coefficient of the settlement trough.

Various factors, including tunnel excavation conditions, geological formation characteristics, the burial depth-to-diameter ratio, and the grouting rate, influence the ground loss rate. Similarly, K is also affected by geological formation conditions. Based on a statistical analysis of numerous tunnel engineering projects, Wu and Zhu (2019) noted that in soil pressure balance shield tunnels in Guangzhou, China, the average ground loss rate V_l and settlement slot width coefficient K were approximately 0.95% and 0.48, respectively [38].

Use Equation (10) to fit the numerical simulation curve in Figure 7. The fitting formula is:

$$S = -0.47 \times e^{-\frac{x^2}{2 \times 14.65^2}} \quad (13)$$

The correlation coefficient R^2 is 0.954, which validates the fitting effect.

In Equation (13), i is 14.65, indicating the specific location of the point where the curve bends in the opposite direction.

Find the second derivative of x in Equation (9), then

$$\begin{aligned} \frac{\partial^2 U_z}{\partial x^2} = & -\frac{4\epsilon R^2 h' (m+1)}{m r^4} + \frac{16\epsilon R^2 h' x^2 (m+1)}{m r^6} - \frac{4\delta R^2 h'}{r^4} \\ & + \frac{32\delta R^2 h' x^2}{r^6} - \frac{8\delta R^2 h' (h'^2 - x^2)}{r^6} + \frac{48\delta R^2 h' x^2 (h'^2 - x^2)}{r^8} \end{aligned} \quad (14)$$

When $\frac{\partial^2 U_z}{\partial x^2} = 0$, the value of x is the coordinate position of the point where the curve bends in the opposite direction. By substituting the corresponding parameters in the item, the value of x can be calculated as 229.47.

The difference in inflection point locations between the numerical simulation (Equation (13)) and the prediction model (Equation (9)) results in significant differences in the curve shapes. To optimize the prediction model, the curve shape can be adjusted by moving the inflection point to a closer position. Let the inflection point of the settlement curve in the numerical simulation be denoted as i_s , and the inflection point in the prediction model be denoted as i_p . Then

$$\frac{i_p}{i_s} = \frac{229.47}{14.65} = 15.66 \quad (15)$$

Substitute it into Equation (9), then

$$U_z = \frac{2\epsilon R^2 (m+1)}{m} \left(\frac{h'}{r^2} \right) - 2\delta R^2 h' \left(\frac{Ax^2 - h'^2}{r^4} \right) \quad (16)$$

where $r^2 = Ax^2 + h'^2$; A is the correction coefficient, with a value of 240. Other parameters are the same as in Equation (9). Equation (16) is the modified prediction model.

Figure 8 presents a comparison among the modified prediction model, the preliminary prediction model, the numerical simulation results, and the prediction results based on the Peck formula. It is worth noting that the Peck formula does not account for the influence of existing tunnels, resulting in significant discrepancies from the actual situation, which is manifested as an overestimation of the maximum settlement value and a steeper overall

curve trajectory. This highlights the key role of the stiffness of existing tunnels in mitigating and coordinating ground deformation. Compared to the Peck formula, the preliminary prediction model produces a significantly smoother ground settlement curve, with values near the crossing section closely approximating the actual values; however, the overall trend exhibits a notable deviation. Overall, the modified prediction model demonstrates good consistency with measured data, making it suitable for predicting ground settlement resulting from the orthogonal crossing of the new tunnel and the existing tunnel.

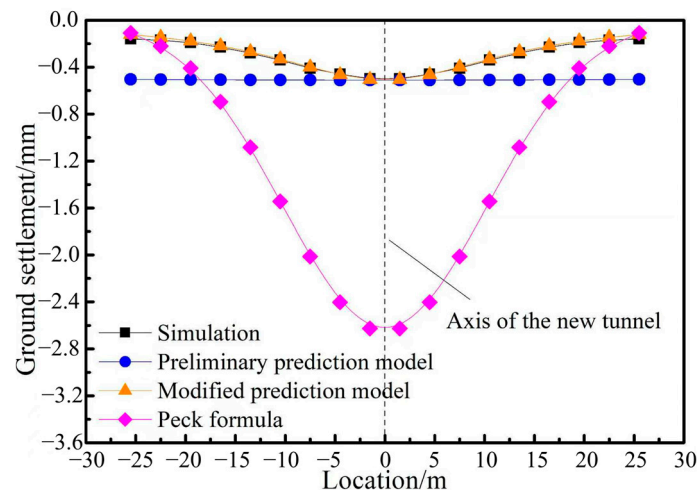


Figure 8. Comparison between the modified prediction model, the preliminary prediction model, the Peck formula, and the numerical simulation.

5. Parameter Analysis and Adaptability Study

To further investigate the influence of parameters on settlement patterns in the modified prediction model, multiple sets of working conditions with different parameter configurations were designed. The modified prediction model was used to calculate the ground settlement, and the results were subjected to detailed comparative analysis. Additionally, corresponding numerical simulation models were established to assess the model's adaptability under various conditions. In the following text, the improved prediction model will be referred to directly as the prediction model.

5.1. Elastic Modulus of the Strata

The elastic modulus and Poisson's ratio of the strata are the primary parameters influencing stratum deformation. Since Poisson's ratio varies relatively slightly, the elastic modulus was selected as the variable, and the prediction model was used to obtain the ground settlement curve. Based on the actual elastic modulus of the strata, coefficients of 0.25, 0.5, 2, and 4 are multiplied to establish the calculation conditions table.

Figure 9 shows the ground settlement curves under different soil elastic modulus conditions, along with corresponding numerical simulation results for comparison. The prediction model exhibits high consistency with the numerical simulation results, thereby fully validating the model's accuracy. Changes in soil elastic modulus have a minor impact on the shape of the settlement curves but a significant influence on the overall settlement magnitude. Softer rock layers result in greater ground settlement at all locations. The stiffness and coordinated deformation capacity of existing tunnels can mitigate settlement near the crossing section. However, settlement increases at locations farther from the crossing section, particularly in softer rock layers. In practical engineering applications, especially when existing structures such as buildings, railways, or bridges are located near the crossing section, special attention should be given to mitigating uneven settlement issues.

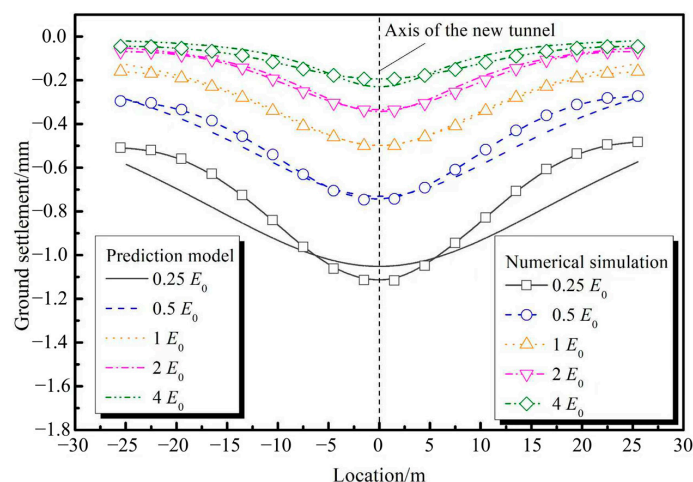


Figure 9. Ground settlement curve with different strata elastic modulus.

The maximum settlement value, $S_{\max}$, and the horizontal distance from the inflection point of the settlement curve to the tunnel axis, i , constitute the two key parameters characterizing the ground settlement curve. Therefore, the prediction model was further adopted to analyze the variation of $S_{\max}$ and i under different soil elastic moduli, as shown in Figure 10. For the convenience of analysis, parameter a is introduced to represent the ratio of the elastic modulus of each strata group to the reference value. The relationships between $S_{\max}$ and a , as well as between i and a , both exhibit a clear inverse trend; however, the specific behavioral characteristics are quite different. As the increases (indicating harder strata), the maximum ground settlement value gradually increases, but the rate of increase gradually slows down. Meanwhile, the inflection point gradually moves closer to the new tunnel axis, and the rate of approach decreases with time. These findings can provide a basis for predicting and evaluating the impact of crossing under different geological conditions.

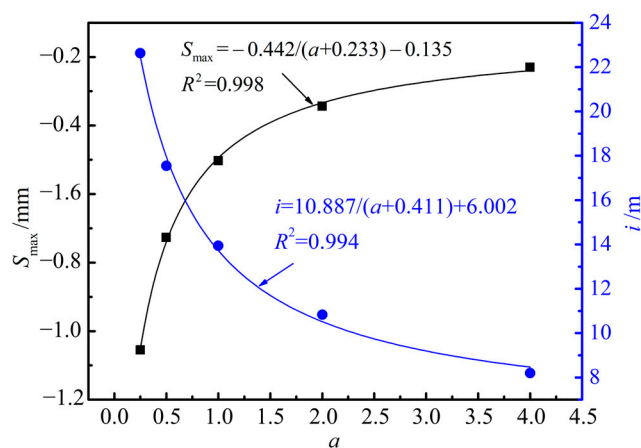


Figure 10. The variation rule of $S_{\max}$ and i with different strata elastic modulus (a is the ratio of the elastic modulus of each group of strata to the reference value).

5.2. Dimensions and Spatial Layout of the Tunnels

The diameter of the existing tunnel (D_1), the diameter of the new tunnel (D_2), the depth of the existing tunnel (H_1), and the spacing between the two tunnels (ΔH) are specified as variables to study the impact of tunnel dimensions and spatial layout on the engineering project. Table 3 lists the specific working condition groups, where Group 1 represents the actual situation. It is worth noting that tunnel support stresses are adjusted according to on-site stresses while maintaining a constant difference. The definitions of all parameters are shown in Figure 11.

Table 3. Working conditions.

Group Number	The Diameter of the Existing Tunnel D_1/m	The Diameter of the New Tunnel D_2/m	The Depth of the Existing Tunnel H_1/m	The Spacing Between Two Tunnels $\Delta H/\text{m}$
1	6	6	10	2
2, 3, 4, 5	4, 5, 7, 8	6	10	2
6, 7, 8, 9	6	4, 5, 7, 8	10	2
10, 11, 12, 13	6	6	12, 14, 16, 18	2
14, 15, 16, 17	6	6	10	1, 3, 4, 5

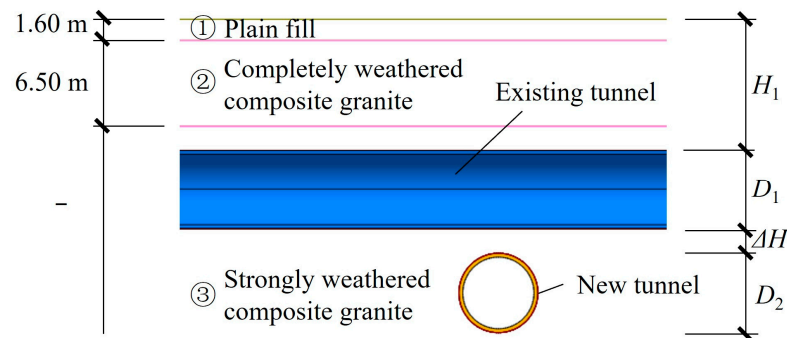
**Figure 11.** Analyzing diagram and meaning of each parameter.

Figure 12 illustrates the ground settlement curves for various tunnel dimensions and spatial layout conditions, along with corresponding numerical simulation results for comparison and analysis. The prediction model is highly consistent with the numerical simulation results. Different parameters have a significant impact on the ground settlement curves. The diameter of existing tunnels primarily affects settlement values within approximately 15 m of the new tunnel axis. When the diameter increases from 4.0 m to 8.0 m, the maximum settlement value decreases from approximately 0.75 mm to approximately 0.35 mm. In contrast, the diameter of the new tunnel affects the entire settlement curve. As the diameter increases from 4.0 m to 8.0 m, the maximum settlement value increases from approximately 0.20 mm to 0.90 mm. However, the burial depth and spacing of the two tunnels have negligible effects on ground settlement.

Figure 13 shows the variation patterns of $S_{\max}$ and i under different tunnel dimensions and spatial layouts. Overall, although various parameters have different effects on $S_{\max}$ and i , they consistently exhibit an apparent regularity. As D_1 increases, $S_{\max}$ shows a quadratic trend with an upward concavity, and the amplitude gradually decreases, while i shows a linear increase. As D_2 increases, $S_{\max}$ also exhibits an upward concave quadratic decrease trend, but with a steeper gradient compared to D_1 . In contrast, i decreases linearly, albeit with a significantly smaller decrease. As H_1 increases, $S_{\max}$ exhibits a linear decreasing trend, but the amplitude is significantly smaller than in the previous two cases, while i increases linearly at a moderate rate. Finally, as ΔH increases, $S_{\max}$ exhibits a concave-up quadratic trajectory with gradually decreasing amplitude similar to H_1 , while i increases linearly at a rate comparable to that of H_1 . In actual engineering, when a large-diameter new tunnel crosses an existing small-diameter tunnel, special attention should be paid to the ground settlement values above the crossing section. Conversely, if both sides are large-diameter tunnels, the crossing scenario may expand the overall range of ground settlement influence.

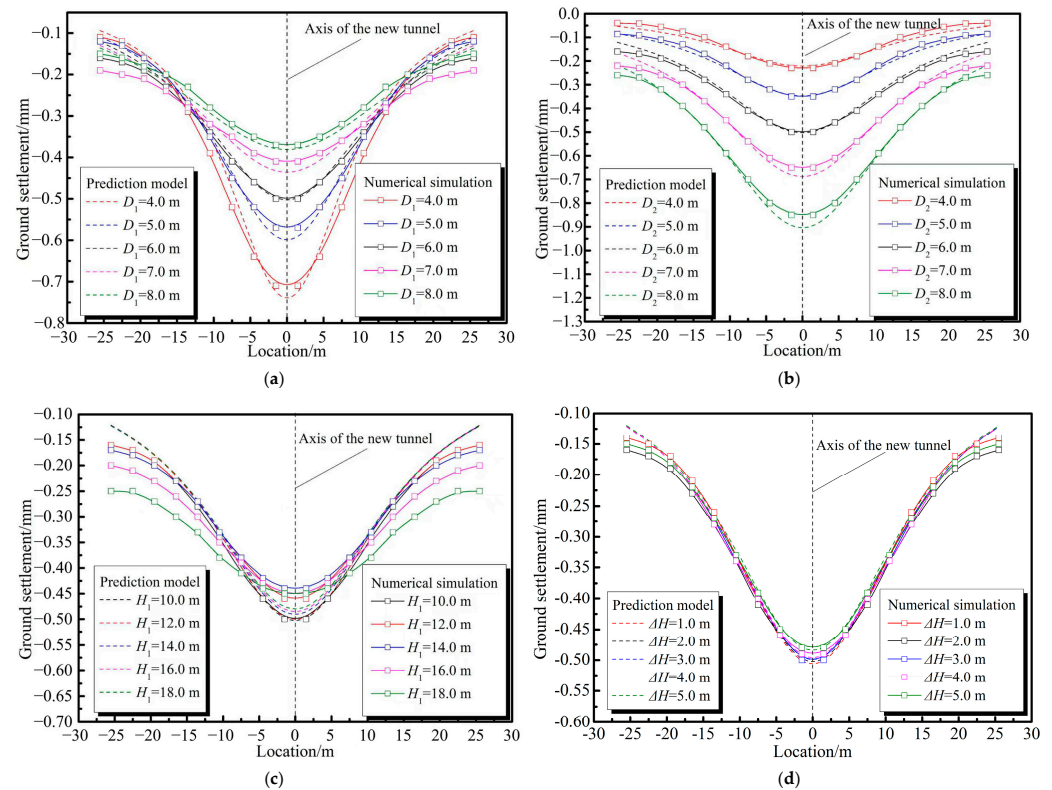


Figure 12. Ground settlement curve with different tunnel sizes and locations: (a) The diameter of the existing tunnel D_1 . (b) The diameter of the new tunnel D_2 . (c) The buried depth of the existing tunnel H_1 . (d) The spacing between two tunnels ΔH .

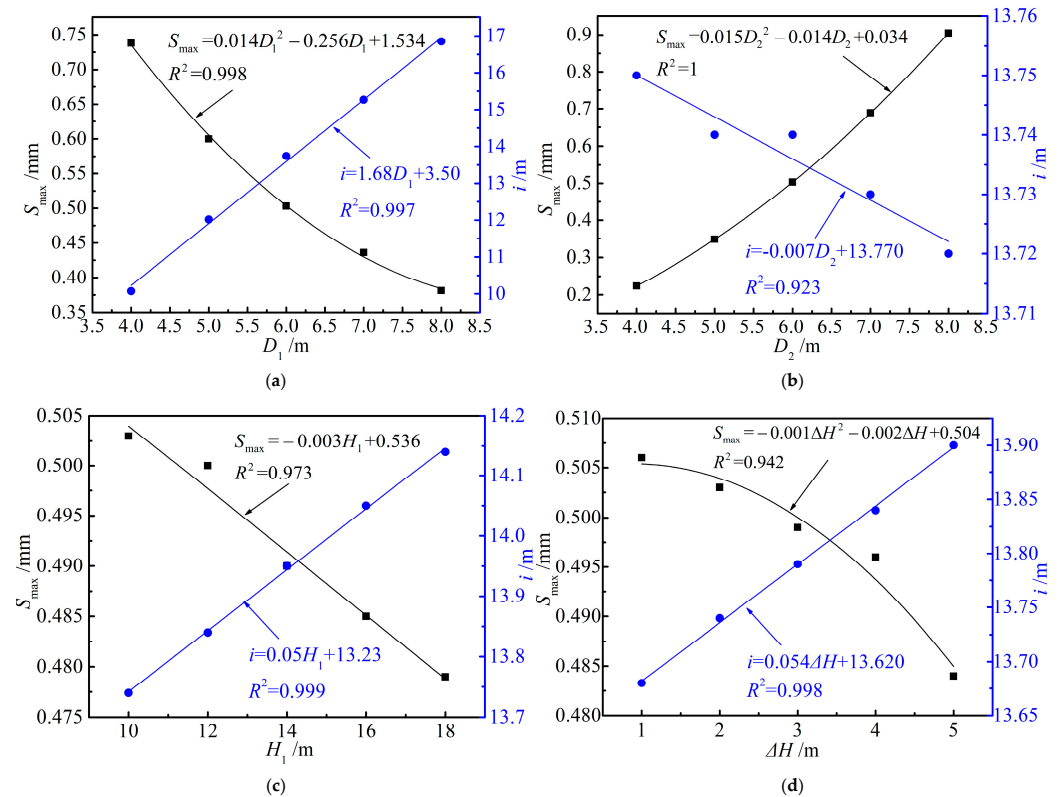


Figure 13. The variation rule of S_{max} and i with different tunnel sizes and locations: (a) The diameter of the existing tunnel D_1 . (b) The diameter of the new tunnel D_2 . (c) The buried depth of the existing tunnel H_1 . (d) The spacing between two tunnels ΔH .

5.3. Adaptability of the Prediction Model

To quantitatively evaluate the consistency between the prediction model and numerical simulation results, the following evaluation method was adopted: For two curves with the same functional form, calculate the integral of the absolute difference between them, divide this value by the integral of one of the curves, and subtract the resulting ratio from 1. The resulting metric is referred to as the consistency evaluation index (CI), which quantifies consistency—the higher the CI value, the stronger the consistency; conversely, the lower the CI value, the weaker the consistency. Figure 14 illustrates the evaluation method.

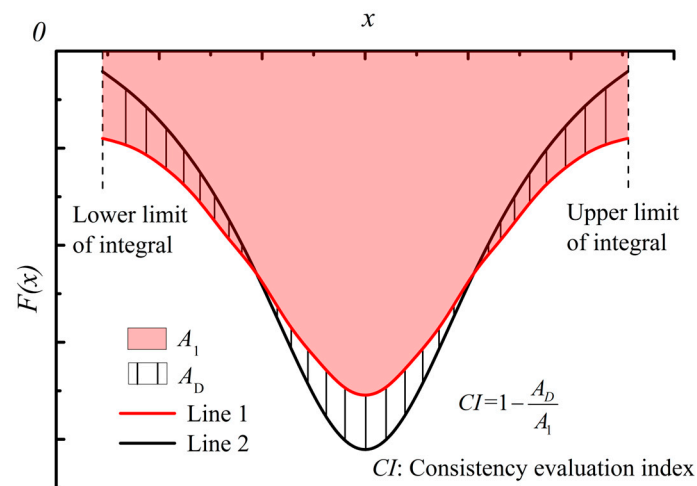


Figure 14. Schematic diagram of consistency evaluation method (Lines 1 and 2 are two curves with the same equation form, A_1 is the integral area of Line 1, A_D is the integral area of the absolute value of the difference between Lines 1 and 2, CI is the consistency evaluation index).

Table 4 presents the CI of ground settlement curves obtained using the prediction model and numerical simulation under different operating conditions. Since the purpose of this consistency evaluation is to assess the agreement between the proposed prediction model and numerical simulation results, it is more conventional to designate A_1 as the numerical simulation results and A_2 as the prediction model results. In fact, as intuitively reflected in Figure 12, the prediction model's results are very close to those of the numerical simulation. Therefore, whether A_1 or A_2 is used as the reference has little impact on the results.

Table 4. Consistency evaluation between the prediction model results and numerical simulation results under different parameters.

a	0.25	0.5	1	2	4	Average
CI	0.873	0.928	0.959	0.939	0.768	0.895
D_1/m	4	5	6	7	8	Average
CI	0.931	0.971	0.959	0.923	0.954	0.948
D_2/m	4	5	6	7	8	Average
CI	0.895	0.960	0.959	0.956	0.967	0.947
H_1/m	10	12	14	16	18	Average
CI	0.959	0.940	0.920	0.885	0.825	0.906
$\Delta H/m$	1	2	3	4	5	Average
CI	0.980	0.959	0.978	0.972	0.976	0.973

Clearly, the prediction model and numerical simulation results exhibit good consistency under all operating conditions, with CI values primarily concentrated around 0.95 and nearly all exceeding 0.9. This indicates that the model shows high robustness and accuracy in predicting ground settlement resulting from the orthogonal crossing of the new and existing tunnels. However, there are slight differences in CI values under different conditions, especially in overly soft or hard rock layers, where CI values tend to be lower. As shown in Figure 9, the primary source of these differences lies in minor deviations in curve shapes: the prediction model exhibits a smoother settlement curve in soft rock layers and a steeper curve in hard rock layers. To enhance the prediction accuracy of ground settlement patterns, the correction coefficient A can be redefined as a function dependent on rock layer conditions. The model demonstrates excellent accuracy in predicting settlement behavior under different tunnel dimensions and spatial layouts. However, when the existing tunnel burial depth is deep ($H_1 \geq 16$ m), the CI value is below 0.9. As shown in Figure 12c, this is primarily due to significant deviations in the wing regions of the curve edges, indicating that the prediction model may introduce errors at locations farther from the crossing point in deep burial crossing scenarios.

6. Engineering Case Verification

In this section, a model test was designed based on actual engineering cases to verify the applicability of the prediction model.

6.1. Engineering Background

A tunnel in Chongqing crosses the Jialing River and the Yangtze River, connecting the Yuzhong Peninsula with Jiangbei and Nan'an Districts in a planar “Y” configuration (Figure 15). The project includes six parallel lines and five closely spaced interchanges. This study focuses on a specific section where the newly constructed tunnel intersects the existing tunnel at right angles. The new tunnel is buried at a depth of 43 m, with a clear distance of 8.25 m between the two tunnels. Both tunnels feature a three-center arch cross-section, measuring 11 m in width, 8.6 m in height, and 77.84 m² in cross-sectional area. Excavation was carried out using the drill-and-blast method, with composite lining. The geological strata in the study section primarily consist of Grade IV sandy mudstone (approximately 95%), with localized interlayers of Grade III sandstone (approximately 5%).

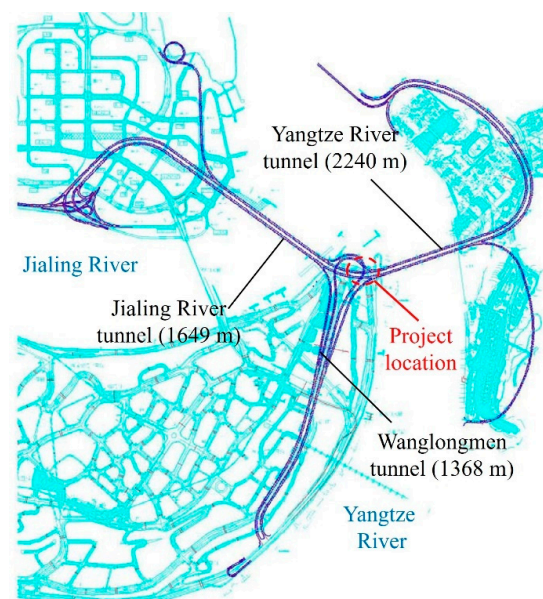


Figure 15. Plan layout of the cross-river tunnels in Chongqing.

6.2. Model Test Process

Based on rigorous derivations from similarity theory, carefully designed test protocols, and strict program execution, indoor similarity model tests can accurately reproduce the unloading effects and ground disturbance caused by tunnel excavation [39,40]. Due to the lack of field monitoring data, similar model tests were designed to obtain the deformation and mechanical response patterns of the tunnel and surrounding rock when a new tunnel undercrosses an existing tunnel. Additionally, considering the test effectiveness, an appropriate weakening treatment was applied to the relatively hard Grade III–IV surrounding rock.

The geometric similarity ratio $C_L = 85$ was determined after careful consideration of the test conditions and expected results. Other similarity ratios were derived strictly based on the principles of similarity theory. However, due to space limitations, the detailed derivation process is omitted here. According to similarity theory, the remaining ratios are determined as follows: unit weight similarity ratio $C_\gamma = 1.27$; Poisson's ratio, strain, and friction angle similarity ratios $C_\mu = C_\varepsilon = C_\varphi = 1$; Strength, stress, cohesion, and elastic modulus similarity ratios $C_R = C_\sigma = C_c = C_E = 107.95$; and lining bending stiffness similarity ratio $C_D = 614,125$. A similar material was prepared using gypsum, quartz sand, barite powder, and water. Approximately 120 test specimens were prepared to calibrate the material parameters (Figure 16). The optimized material ratio was gypsum: quartz sand: barite powder: water = 0.03:0.67:0.24:0.06. The physical and mechanical properties of the similar materials prepared according to this ratio, as well as the properties of the original material, are shown in Table 5.



Figure 16. Samples for the determination of similar material parameters.

Table 5. The physical and mechanical parameters of similar materials and prototype materials.

	Unit Weight $\gamma/\text{kN/m}^3$	Elastic Modulus E/GPa	Poisson's Ratio $\mu/-$	Cohesion c/MPa	Friction Angle $\varphi/^\circ$
Prototype materials	24.60	0.3	0.33	0.13	31–33
Similar materials	19.40	0.0028	0.31	0.0011	26.52

The dimensions of the model test apparatus are 2000 mm × 900 mm × 1400 mm (length × width × height). The left, right, rear, and bottom sides of the test chamber are constructed from 5 mm thick steel plates, reinforced with angle iron. A viewing window measuring 1400 mm × 1000 mm (length × height) is installed on the front side, fitted

with a 10 mm thick acrylic glass panel for visual monitoring. In the model test, the tunnel cross-section adopts a three-center arch structure, with an arch width of 130 mm, an arch height of 100 mm, a clear distance of 100 mm between the two tunnels, and a new tunnel burial depth of 500 mm. The original tunnel length is 2000 mm, and the new tunnel length is 900 mm. Only the primary support structures were simulated in the test, with the secondary lining serving as a safety reserve. Based on the similarity ratio of the bending stiffness of the lining, 0.9 mm-thick stainless steel plates were selected to simulate the lining. The model test chamber and its dimensions are shown in Figure 17, which also indicates the layout of the measurement points, spaced 300 mm apart and located directly above the existing tunnel.

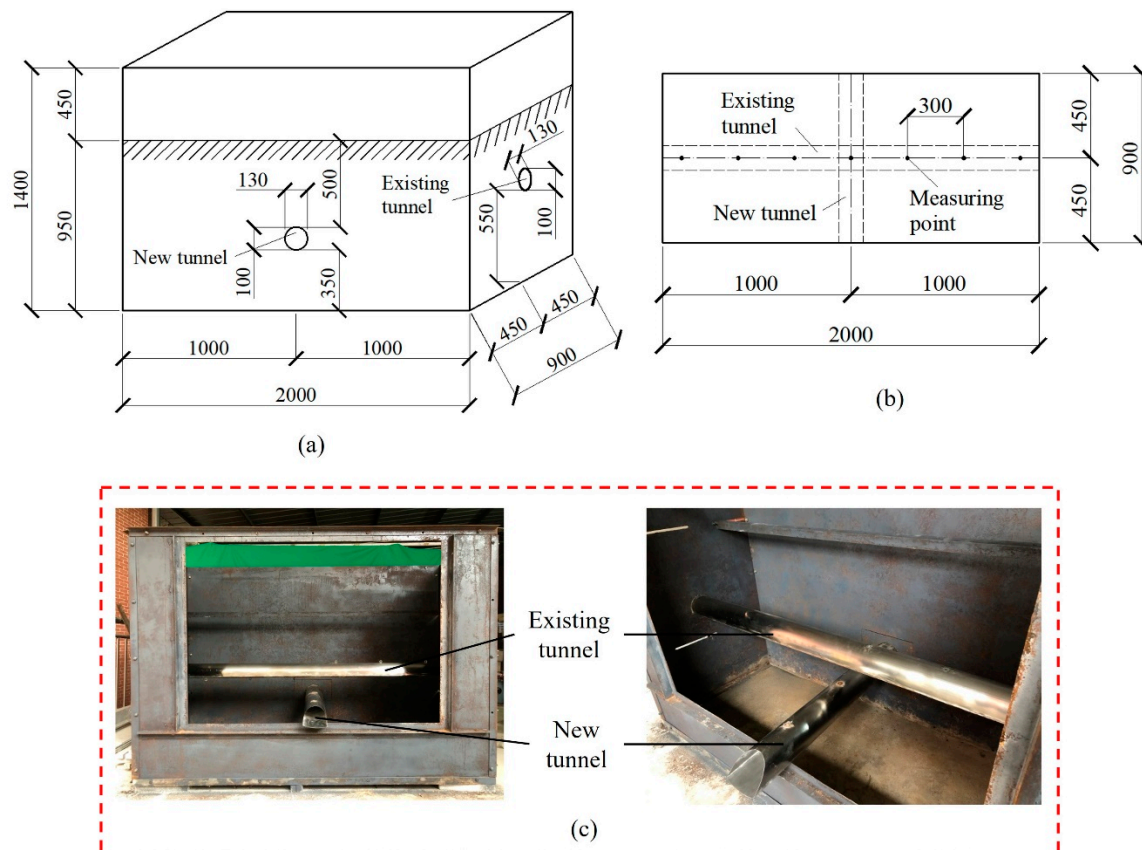


Figure 17. The model test box and dimensions (unit: mm): (a) Model test dimensions. (b) Measuring point arrangement. (c) Model test box.

The model test processes are shown in Figure 18. During the test, the same material was continuously filled in 12 layers, with compaction performed after each layer to ensure uniform density. When the filling reached the tunnel bottom elevation, the tunnel was inserted through the opening reserved in the model box wall and extended to the other side, thereby exposing it and completing the embedding. After the material was filled, a 2-day curing period followed, after which excavation operations commenced. Digital dial gauges fixed to the monitoring frame via a magnetic base were used to measure ground settlement. Self-made excavation equipment was used to excavate the new tunnel, including two special long-handle shovels and a laser rangefinder, which was used to measure the excavation depth of each step. The tunnel was advanced in increments of 5 cm in depth. After each excavation increment, work was halted for 10 min to allow the tunnel and surrounding strata to deform fully. The dial gauge must be reset to zero before testing begins, and readings were recorded before each subsequent excavation step.



Figure 18. Model test processes: (a) Similarity materials production. (b) Materials filling. (c) Tunnels burying. (d) Continue filling. (e) Similarity materials curing. (f) Dial gauges arranging. (g) New tunnel excavation. (h) Excavation complete.

In addition to ground settlement, the test also monitored the deformation of the existing tunnel, the earth pressure around the two tunnels, and the strain in the surrounding areas. However, since this paper focuses only on ground settlement directly above the existing tunnel, the analysis of other monitoring parameters is omitted here.

6.3. Results and Verification

Figure 19 shows the ground settlement data monitored directly above the existing tunnel during the model test. Based on the excavation progress, the test phase is divided into Stages 1 to 5, with the distance values reflecting the actual engineering proportions.

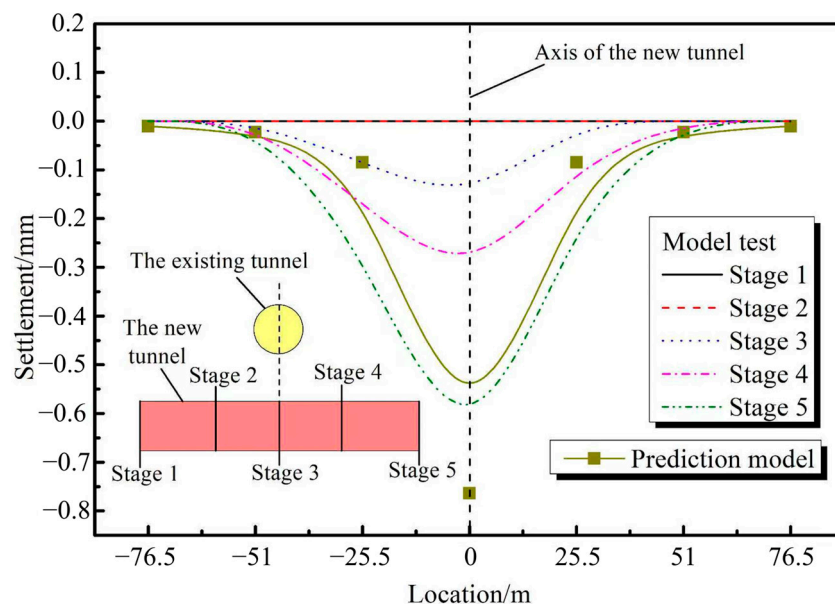


Figure 19. Comparison of model test results and prediction model.

Stage 1: The new tunnel excavation begins (excavation depth in the model test is 0 cm).

Stage 2: The new tunnel is located 17 m ahead of the existing tunnel axis (excavation depth in the model test is 25 cm).

Stage 3: The new tunnel is directly below the existing tunnel (excavation depth in the model test is 45 cm).

Stage 4: The new tunnel advances to a distance of 17 m from the axis of the existing tunnel (excavation depth in the model test is 65 cm).

Stage 5: Excavation of the new tunnel is complete (the excavation depth in the model test is 90 cm).

Meanwhile, after the new tunnel was excavated, the prediction model was used to calculate the settlement of the ground above the existing tunnel, as shown in Figure 19. The parameter calculation process is the same as described earlier and will not be repeated here. For the sake of direct comparison, the ground settlement values obtained from the model test were converted to actual engineering quantities using geometric similarity ratios.

As shown in the observations, no ground settlement occurred directly above the existing tunnel during Stages 1 and 2. When the new tunnel reached directly below the existing tunnel (Stage 3), settlement began to occur, with a maximum value of 0.17 mm, immediately appearing directly above the axis of the new tunnel. After the new tunnel passes through the existing tunnel, settlement of the poor foundation within approximately 50 m on both sides of the new tunnel axis continues to worsen, eventually reaching approximately 0.765 mm. The final ground settlement curve derived from the prediction model is highly consistent with the model test results, with a consistency evaluation index (*CI*) of 0.800. This confirms the model's accuracy in predicting ground settlement above the existing tunnel caused by the new tunnel's orthogonal undercrossing.

7. Discussion

7.1. Applicability of the Double-Track Tunnel Undercrossing Problems

The prediction model proposed in this paper was primarily researched and validated for scenarios where a new single-track tunnel undercrosses an existing single-track tunnel. However, its applicability can be extended to double-track tunnel undercrossing situations. Double-track tunnel undercrossing can be categorized into three types: (1) a new double-track tunnel undercrossing an existing single-track tunnel; (2) a new single-track tunnel undercrossing an existing double-track tunnel; (3) A new double-track tunnel crossing an existing double-track tunnel. For case (1), the new double-track tunnel can be regarded as the superposition of two single-track tunnels. The settlement curve may change from a "U" shape to a 'W' shape, but the influence of the existing tunnel may weaken the "W" trend. By adjusting parameters of the equivalent layer method and the mirroring method (such as equivalent stiffness and geometric relationships), the model can theoretically be applied to this scenario.

However, for scenarios (2) and (3), the presence of the existing double-track tunnel introduces complex spatial interactions. The superposition effects of settlement curves along the axis of the existing tunnel and in the perpendicular direction must be considered. The current model simplifies tunnels into strata using the equivalent layer method, failing to adequately account for the transition between the double-track tunnel and intermediate strata. This limitation prevents direct resolution of Scenarios (2) and (3).

Future research will focus on validating the model's applicability in case (1) and exploring improvement strategies for cases (2) and (3).

7.2. Applicability in Non-Vertical Undercrossing Problems

The prediction model proposed in this paper was primarily developed and validated for vertical undercrossing scenarios, employing a combination of the equivalent layer method and the mirroring method to calculate ground settlement. However, non-vertical undercrossing introduces complex spatial and geometric relationships, as well as asymmet-

ric stress distributions. Existing models lack cross-angle parameters, making them difficult to apply directly.

To address this issue, introducing an angle-dependent correction function can adjust the asymmetry in axial deformation and settlement curves of existing tunnels. This correction function can be derived through regression analysis of extensive field monitoring data or combined with numerical simulations to capture the impact of angular variations on settlement.

Future research will explore the incorporation of angle-dependent terms into the mirroring method and validate the model's adaptability through practical non-vertical underpass cases, thereby expanding its application scope.

8. Conclusions

By combining the equivalent layer method with the mirror method, a prediction model was preliminarily proposed for directly estimating ground settlement caused by a new tunnel crossing beneath an existing tunnel at right angles. The model was subsequently modified in light of the significant discrepancy with reality. Parameter analysis was conducted using the modified prediction model, and its applicability was evaluated through comparison with corresponding numerical simulations. Based on an actual engineering case, indoor similarity model tests were designed to validate the practicality of the prediction model in engineering applications. The main conclusions are as follows:

1. In cases where a new tunnel undercrosses an existing tunnel, the significant stiffness of the existing tunnel interacts with the foundation settlement, resulting in a flatter settlement curve and a reduction in the maximum settlement value directly above it. The classic Peck formula does not account for the presence of existing tunnels when calculating foundation settlement, leading to significant errors in the results.
2. Based on the equivalent layer method and the mirror method, the preliminary proposed ground settlement prediction model has the advantages of simple formulas, clear parameters, and high computational efficiency. Compared to the Peck formula, this model considers the influence of existing tunnels on ground settlement; however, it overestimates the stiffness difference between the tunnel and the surrounding strata, as well as the overall deformation capacity of the tunnel lining.
3. The modified prediction model retains the advantages of the original model while introducing a correction coefficient A to optimize the calculation results, thereby enabling a more reasonable assessment of the impact of existing tunnels on ground settlement. Compared with the Peck formula and the preliminary prediction model, this model has significantly improved prediction accuracy.
4. The elastic modulus of the strata has a minor influence on the shape of the ground settlement curve but a significant impact on the magnitude of settlement. Softer strata result in greater settlement at all locations. The diameter of existing tunnels primarily affects settlement within approximately 15 m of the new tunnel axis, with settlement values decreasing gradually as the diameter increases. Conversely, the diameter of the new tunnel determines the overall shape of the settlement curve; the larger the tunnel diameter, the more pronounced the settlement amplitude will be. The burial depth of existing tunnels and the spacing between tunnels have negligible effects on settlement. Consistency assessments confirm that the prediction model exhibits strong adaptability under various conditions ($CI > 0.9$).
5. When the new tunnel is directly located below the existing tunnel, ground settlement begins to occur, with a maximum settlement of 0.17 mm. After the new tunnel passes the existing tunnel, ground settlement continues to increase within approximately 50 m on either side of the new tunnel axis, eventually reaching approximately

0.765 mm. The consistency evaluation results indicate that the prediction model demonstrates good application effectiveness in actual engineering projects.

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