

Research Article

The Environmental Effects Induced by a Metro Shield Tunnel Side-Crossing on Adjacent Pile Foundations and Its Impact Partition

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With the advantages of fast construction speed and small disturbance to stratum, shield tunneling is becoming the preferred construction scheme for metro construction. However, if the disturbance of stratum cannot be clearly understood, it will affect the safety of adjacent underground structures and aboveground buildings. Based on the construction of the interval tunnel between Shizishan station and Chuanshi station of Chengdu metro line 7 project, this paper proposed a simplified calculation method to analyze the impact of shield excavation on adjacent pile foundation. A two-stage method of modified Peck formula and Winkler elastic foundation model was used to investigate the additional displacement and internal force of pile. A three-dimensional numerical model validated with the field test data was performed to compare with the theoretical results. The impact factors and their impact degree were discussed. The results show that the additional displacement and internal force are at a reasonable and acceptable level. The permanent pile foundation should adopt the small diameter pile as far as possible. When the small diameter pile foundation cannot meet the stress requirements, the design scheme of the pile group should be adopted. The area around shield tunnel is divided into strong impact area of pile foundation ($S \leq 0.75D$), slight impact area ($0.75D \leq S \leq 1.5D$), and no impact area ($S > 1.5D$). The pile researched in this paper ($S = 0.83D$) is in the slight impact area.

1. Introduction

With the continuous progress of urbanization in China, the urban population is growing, and urban traffic pressure is also increasing. Metro has the advantages of large passenger capacity, fast speed, and full use of underground space, which is more and more popular in the modern society. However, the urban subway has strict requirements for its construction methods. Because of the high density of buildings in cities, the construction of the metro will cause stratum movement and bring adverse effects on ground buildings. Especially when passing through high-rise buildings, the disturbance caused by construction on stratum must be more strictly controlled. The construction period of metro is long, and the interruption of traffic should be avoided as far as possible when crossing the road. With

the advantages of fast construction speed and small disturbance to stratum, shield tunneling is becoming the preferred construction scheme for metro construction [1–6].

Although shield tunneling has many advantages as mentioned above, and the construction technology has made great progress after many years of development, due to the defects of geological conditions and construction technology, the advance of shield tunneling will inevitably make a disturbance on the stratum, change the stress state of soil, and cause additional internal force and deformation of adjacent buildings and pipelines. If the disturbance of stratum cannot be clearly understood, it will affect the safety of adjacent underground structures and aboveground buildings. Among these potential safety hazards, the problem of shield tunneling closely passing through bridges or building piles is one of the most frequently encountered

problems. Therefore, how to analyze the impact of tunnel construction on the adjacent pile foundation fast and accurately has always been a hot issue in the design and construction of urban metro [7–9].

Over the years, previous researchers have done numerous studies on methods to evaluate the effects of shield tunnel construction on the existing pile foundations and achieved some results [10–15]. These analysis methods can be divided into two categories: one is complete numerical simulation analysis, including simultaneous simulation of pile, soil, and tunnel excavation. The other is a simplified two-stage method, which involves the initial separation of soil and pile, so that the soil motion can be calculated first and then imposed to the pile. Complete numerical simulation analysis is usually based on a three-dimensional finite element model (FEM) or finite difference model (FDM) analysis, which provides a complete solution for tunnel-soil-pile interaction [16–21]. However, considering the nonlinear soil behavior and complex construction sequence, such analysis will become more uncertain due to the errors in the construction of three-dimensional model, the high sensitivity to the grid meshing, the distribution of mechanical properties of elements at the pile-soil interface, the interaction with adjacent structures, and the modeling of excavation sequence. Simulated costs can also become very high.

Because of the simplicity and accuracy of the calculation process, the two-stage analysis method can be used to make up for the shortcomings of the above-mentioned complete numerical simulation analysis, so it has been widely used and developed [22–25]. Basile researched the effects induced by tunneling on existing pile foundations through a two-stage procedure and a computer program PGROUPN for pile-group analysis based on a nonlinear boundary element solution [26]. Zhang et al. proposed a simplified solution based on Pasternak's foundation model to predict the lateral displacements and internal forces of a single-pile and group-piles induced by tunneling considering the effects of lateral soil displacements [27]. Wu et al. proposed a soil-tunnel interaction model based on the Timoshenko beam simplified model (TBSM) of the tunnel [28]. Franza et al. presented an elastic study of tunnel-pile-structure interaction through Winkler-based Two-Stage Analysis Methods (TSAMs), focusing on structural displacements resulting from tunnel excavation beneath piled frames or simple equivalent beams [29]. However, in the existing research, most of their solutions are complex, the mechanical state of pile foundation and its impact factors are not analyzed enough, and the connection with engineering project is not close enough, which is not conducive to practical application, so further improvement is needed.

Based on the two-stage method of modified Peck formula and Winkler elastic foundation model, this paper studied the impact of shield tunnel construction on adjacent pile foundation and analyzed the mechanical state of pile foundation and its impact factors by theoretical derivation and numerical simulation. And according to the variation degree of additional internal force of pile foundation, the stratum around the tunnel is divided into a strong impact zone, weak impact zone, and no impact zone, which can

provide guidance and reference for the design and construction of similar underground projects in the future.

2. Project Overview

Chengdu Metro Line 7 is an important rail transit line in Chengdu, China. It links up Chengdu Railway Station, Chengdu East Railway Station, and Chengdu South Railway Station and forms transfer relations with several urban rapid rail transit and regional rail transit radiation lines. Chengdu Metro Line 7 is a loop line, circling the city outside the Second Ring Road with a total length of 38.61 km, and the whole journey is underground. The project in this paper is an interval tunnel between Shizishan Station and Chuanshi Station. The tunnel consists of two single-track tunnels with a total length of 970.4 m, in a northeast-southwest direction. The location of the interval tunnel is shown in Figure 1.

The stratum that the tunnel passes through is mainly the Quaternary bottom cover, and the surface is mostly a small amount of artificial filling of the Quaternary miscellaneous fill. The lower part is the Quaternary lower Pleistocene glacial water deposit, the clay of impact origin, and the lower part is the weathered mudstone of the Cretaceous upper Guankou Formation. The tunnel is located on the third-grade terrace of Minjiang River in the western Sichuan plain. It is a Piedmont landform with surface water being gully water. Groundwater is not extremely abundant, and the groundwater level is below 15 m.

The location of the tunnel is extremely complex and sensitive. Around the tunnel are Wanke City Garden, Sichuan Normal University Hospital, Jiahe Garden, and other buildings. Furthermore, rainwater and gas pipelines are also densely distributed in this area. But one of the most serious problems is the left line tunnel closely passing through the East No. 1 Pile of Wanke City Garden Building 59. The distance between the pile foundation and the center of the left line of the tunnel is only 5 m. Therefore, how to evaluate the interaction between shield, soil, and existing building pile foundation in the construction process is extremely important to limit the risk of damage to existing buildings.

3. Basic Theory and Calculation Method

Based on the two-stage method of modified Peck formula and Winkler elastic foundation model, the analysis is carried out in this part. The horizontal displacement coordinate system of stratum is $U_{(z)}$ - O - Z and that of pile foundation is $W_{(z)}$ - O - Z . The two coordinate systems coincide as shown in Figure 2.

The first stage is the deformation of stratum around the pile caused by tunnel excavation, and in this stage, the impact of the pile on stratum deformation is not considered at this time. The original Peck formula is derived in clay conditions, then it is improved to use in other kinds of rock and soil mass [30–33]. Han, through statistical analysis and practical investigation, found that Peck formula can be well applied in China [34]. The modified Peck empirical formula can solve the stratum settlement $S_{(x, z)}$ at any depth and the free horizontal displacement $U_{(x, z)}$ of stratum at pile

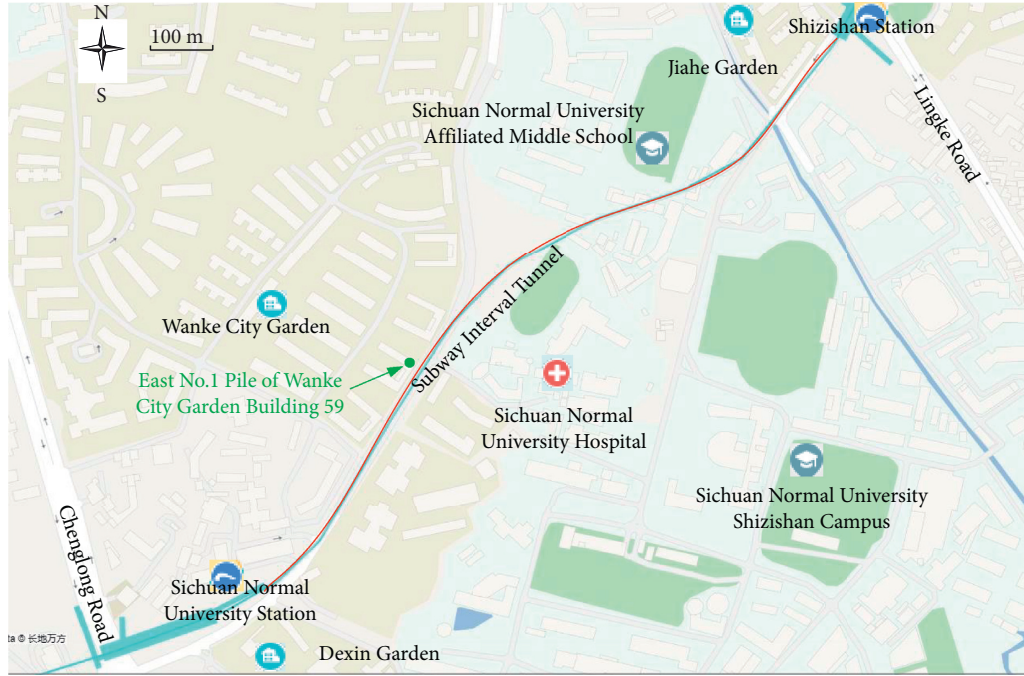


FIGURE 1: The location of the interval tunnel.

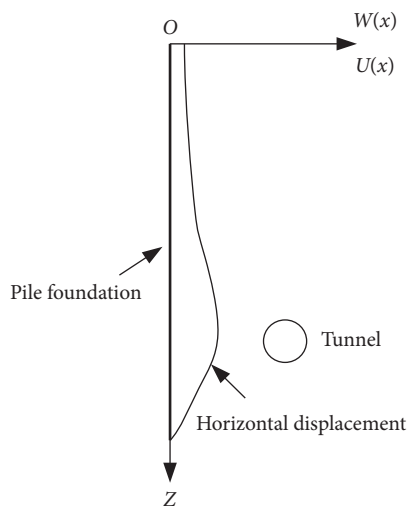


FIGURE 2: The coordinate system of horizontal displacement.

foundation caused by shield tunneling, and the $U_{(x, z)}$ is fitted into a polynomial $U_{(z)}$ for the convenience of the second stage solution [35, 36]. The diagram of Peck formula is shown in Figure 3.

$$\begin{aligned}
 S(x, z) &= \frac{V_s}{\sqrt{2\pi}i_z} \exp\left(-\frac{x^2}{2i_z^2}\right), \\
 U(x, z) &= \frac{-aKV_s x}{\sqrt{2\pi}i_z^2} \exp\left(-\frac{x^2}{2i_z^2}\right), \\
 i_z &= (H - az)K,
 \end{aligned}
 \tag{1}$$

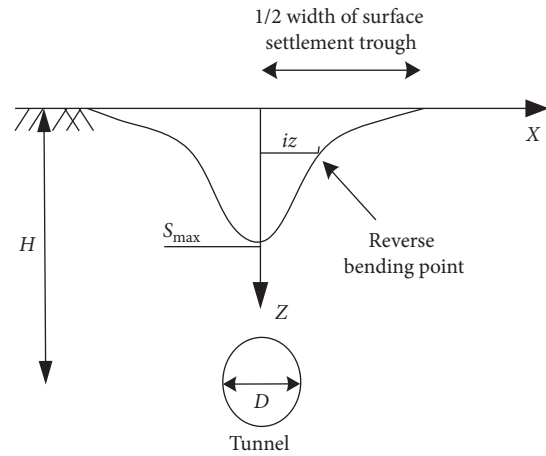


FIGURE 3: The diagram of Peck formula.

where x is the horizontal distance from the central axis of the tunnel (m); z is the vertical distance from the ground surface (m); i_z is the horizontal distance between the reverse bending point of settlement curve and the central axis of tunnel (m); V_s is the ground loss per unit length caused by tunnel excavation (m^3/m); H is the burial depth of tunnel center; K, a are statistical parameters, generally for cohesive soil layer, $K = 0.4 \sim 0.7, a = 0.65$, for sandy soil layer $K = 0.2 \sim 0.3, a = 0.5$.

In the second stage, the pile is regarded as an elastic foundation beam. The interaction between pile and stratum is simulated by Winkler foundation spring with a single parameter. The stratum displacement obtained in the first stage is applied to the structure, and the equilibrium differential equation is established. The additional displacement and internal force of pile foundation are analyzed by

solving the differential equation. The soil is idealized as a linear elastic body. It is assumed that the stress at any point on the side of the pile foundation is proportional to the horizontal displacement of the soil at that point, but not related to the stress and settlement at other points in the soil. That is to say, the soil is composed of a series of spring elements that are close to each other and independent of each other, and the soil is not subject to shear stress.

3.1. The Fundamental Assumption. The initial horizontal load on the pile is triangular distribution:

$$\sigma(z) = \gamma \times z. \quad (2)$$

The interaction between pile foundation and stratum is simulated by Winkler foundation spring, and there is no separation between pile foundation and stratum [37–40]. When the tunnel has not been constructed and ground displacement has not occurred, the deformation of foundation spring is

$$\varepsilon_0(z) = \frac{\sigma(z)}{k_h}. \quad (3)$$

The reaction force of foundation spring is proportional to its deformation,

$$q(z) = (W(z) + \varepsilon_0(z) - U(z)) \times k_h, \quad (4)$$

where γ is the density of soil; z is the burial depth; k_h is the product of horizontal soil bed coefficient k'_h and pile diameter D .

3.2. The Equilibrium Differential Equation. The whole pile is divided into many microelements, and the additional stress mode of each microelement is shown in Figure 4. Based on the static equilibrium condition of microelement, it can be concluded that

$$-Q + (Q + dQ) - qdz + \sigma dz = 0. \quad (5)$$

After arrangement and replacement, it can be obtained that

$$\frac{dQ}{dz} = (w - u) \times k_h, \quad (6)$$

and

$$\begin{cases} \frac{dQ}{dz} = \frac{d^2 M}{dz^2}, \\ EI \frac{d^4 w}{dz^4} = -\frac{d^2 M}{dz^2}. \end{cases} \quad (7)$$

So, equation (6) can be transformed as

$$EI \frac{d^4 w}{dz^4} = (u - w) \times k_h. \quad (8)$$

That is, the equilibrium differential equation

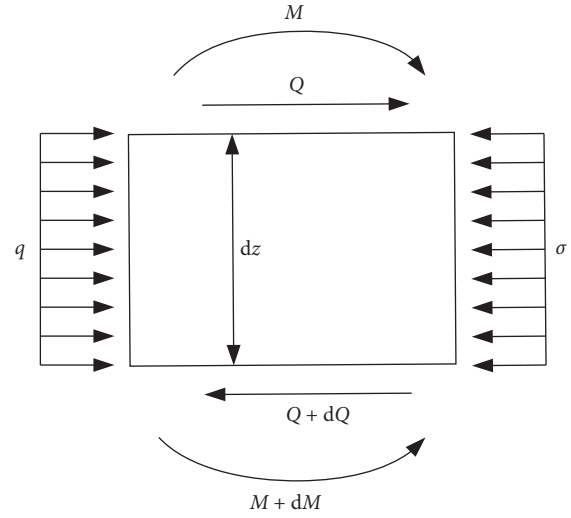


FIGURE 4: The microelement of pile.

$$EI \frac{d^4 w}{dz^4} + wk_h = uk_h, \quad (9)$$

where E is the elastic modulus of pile; I is the inertial moment of pile, $I = (\pi D^4/64)$; D is the pile diameter.

3.3. Boundary Conditions. The bottom of the pile is regarded as a free boundary. There is no concentrated horizontal load; the shear force of the pile bottom surface is zero. There is no bending moment load; the bending moment of the pile bottom surface is zero. That is, when $z = H$,

$$\begin{aligned} EI \frac{d^3 w}{dz^3} &= 0, \\ EI \frac{d^2 w}{dz^2} &= 0. \end{aligned} \quad (10)$$

Although the top of pile is constrained by building foundation, if the building is based on a small independent foundation, which is deformed together with the ground, therefore, the constraints are small and can also be regarded as a free boundary. That is, when $z = 0$,

$$\begin{aligned} EI \frac{d^3 w}{dz^3} &= 0, \\ EI \frac{d^2 w}{dz^2} &= 0. \end{aligned} \quad (11)$$

3.4. Solutions. The homogeneous differential equation of equation (9) is

$$EI \frac{d^4 w}{dz^4} + wk_h = 0. \quad (12)$$

Its general solution is

$$w^0 = e^{\lambda y} (c_1 \cos \lambda y + c_2 \sin \lambda y) + e^{-\lambda y} (c_3 \cos \lambda y + c_4 \sin \lambda y), \quad (13)$$

where $\lambda = \sqrt[4]{k_h/4EI}$, and c_1, c_2, c_3, c_4 are undetermined constants, which can be determined by boundary conditions,

$$\begin{cases} w_{z=0,H}^0(2) + w_{z=0,H}^*(2) = 0, \\ w_{z=0,H}^0(3) + w_{z=0,H}^*(3) = 0. \end{cases} \quad (14)$$

Bring the fitting polynomial $w^* = U(z)$ of the first stage into equation (14), and the final displacement solution of pile foundation can be obtained:

$$w = e^{\lambda y} (c_1 \cos \lambda y + c_2 \sin \lambda y) + e^{-\lambda y} (c_3 \cos \lambda y + c_4 \sin \lambda y) + \sum_{i=0}^n a_i z^i. \quad (15)$$

Furthermore, the additional bending moment and the additional shear force can be obtained from equation (15):

$$M = -EI \frac{d^2 w}{dz^2}, \quad (16)$$

$$Q = -EI \frac{d^3 w}{dz^3}.$$

4. Application of Engineering Project

Based on the typical engineering project of the left line tunnel closely passing through the East No. 1 Pile of Wanke City Garden Building 59 in the interval tunnel between Shizishan station and Chuanshi station of Chengdu metro line 7, the distance between the pile and the tunnel is 5 m. The length and diameter of pile are 20 m and 0.3 m, respectively. The Earth pressure balance shield is adopted with a burial depth of 15 m, and its external diameter is 6.28 m. Because the external diameter of segment is 6 m, there is a gap at the shield tail, which needs to be grouting-filled. The position relation of pile and tunnel is shown in Figure 5.

4.1. Parameters Validation. In order to ensure the adaptability of statistical parameters and models, focusing on the ground settlement, a three-dimension numerical model is built, as Figure 6 shows, to compare theory calculation. In this numerical calculation, finite difference software Flac^{3D} is adopted. The soil is simulated by Mohr Coulomb model and solid element. The pile and segment are simulated by pile element and shell element, respectively. The height of the model is 40 m, which is consistent with the thickness of the actual bearing soil layer. Avoiding the effect of boundary effect, the horizontal and longitudinal directions of the model are 120 m and 100 m, respectively. The physical and mechanical parameters of soil are mainly obtained by laboratory tests and point load tests. By using the standard soil test method [41], the test results are shown in Table 1.

The ground settlement obtained by numerical simulation and theory calculation is fitted in Figure 7. Most parts of

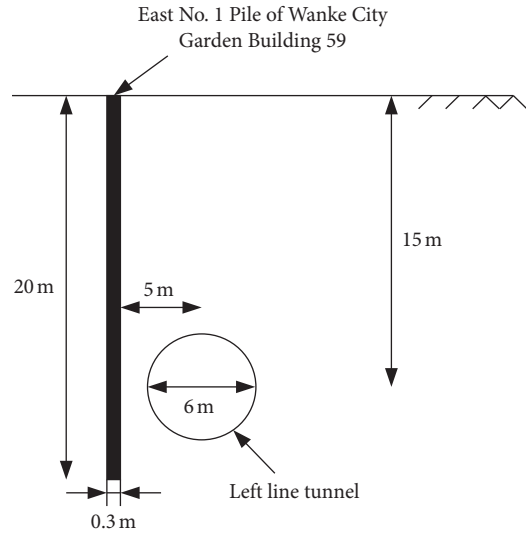


FIGURE 5: The position relation of pile and tunnel.

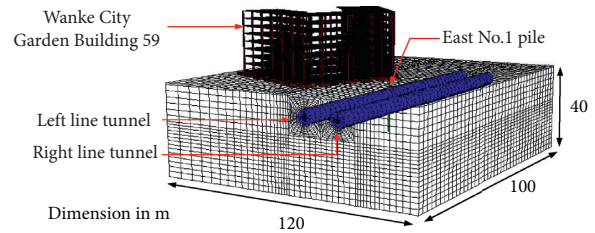


FIGURE 6: Three-dimension numerical model.

the two curves are in good agreement, and until the horizontal distance from the central axis of the tunnel reaches 14 m, there is a slight deviation between the two. And more importantly, after the field monitoring data is written in, the difference among them is obvious, but acceptable. With the decrease of the horizontal distance from the central axis of the tunnel, the difference tends to increase and reaches a maximum at the center of the settlement trough. The maximum of ground settlement S_{\max} calculated by modified Peck formula is -8.6 mm, S_{\max} for field monitoring is -9.7 mm, the difference is 1.1 mm, and the deviation rate is 12.79%. This shows that the settlement troughs obtained by numerical simulation and theoretical calculation are acceptable, and the selected parameters and models are reasonable. If the difference is too large, parameters can be modified through a back-analysis method based on the field monitoring data.

4.2. Calculation Result of Pile. After parameters validation, the two-stage calculations of pile are carried out based on the process introduced in Section 3. The horizontal displacement of pile is shown in Figure 8. The results of the two-stage method and numerical simulation presented are generally close, but there are relatively large differences at the bottom of the pile and at the peak displacement. Specifically, the maximum displacement of the two-stage method is larger, and the bottom displacement is smaller, which leads to a

TABLE 1: Mechanical parameters of various materials.

Material	Unit weight (kN/m ³)	Height/thickness (m)	Passion's ratio	Cohesion (kPa)	Friction angle (°)	Elastic modulus (MPa)
Topsoil	20	2.1	0.3	45	14	5.94
Intermediary weathered mudstone	23.4	37.9	0.17	300	18	180
Segment	25	0.3	0.3	—	—	3.0×10^4
Grout	25	0.14	0.2	—	—	1.20×10^3
Pile	25	0.3	0.2	—	—	3.0×10^4

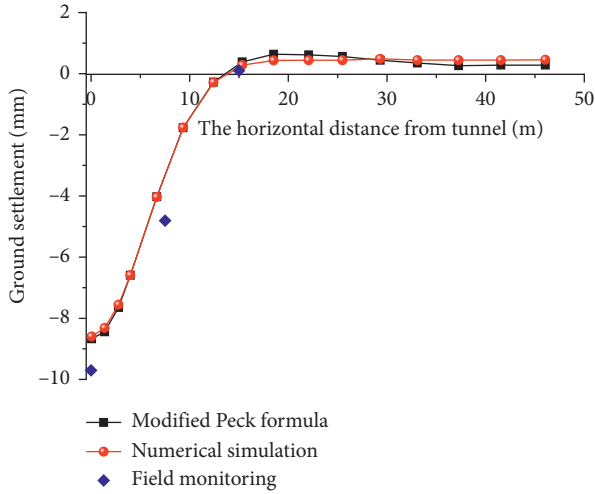


FIGURE 7: The ground settlement trough curve.

larger curvature. The displacement difference maximum is at the bottom, and the deviation rate is 38.71%. This is mainly because of two reasons. On one hand, in order to simplify the calculation, the boundary conditions assume that there are no moment and horizontal constraints at both ends of the pile, but there are a few constraints. On the other hand, the shear stiffness between stratum spring elements is not considered in the single-parameter Winkler model, which will bring some errors to the deformation calculation. Two-parameter Pasternak model with shear stiffness G_p between stratum spring elements can be introduced to replace the single-parameter Winkler model adopted in this paper [42–44].

$$EI \frac{d^4 w}{dz^4} - DG_p \frac{d^2 w}{dz^2} + w Dk_h = uk_h, \quad (17)$$

where the meaning of parameters is consistent with Winkler model's. It is worth mentioning that this will also lead to a more complex solution process and parameter validation. A small amount of sacrifice in accuracy is in exchange for the convenience. The latter is of great significance for practical application in engineering. Therefore, the single-parameter Winkler model is chosen in this paper.

Because the horizontal displacement of the stratum is limited by the pile, the stratum will bring reaction force on the pile, resulting in additional internal force. While the horizontal soil bed coefficient k'_h is 135 MN/m^3 , the shear force and bending moment of pile are shown in Figures 9 and 10, respectively. The maximum bending moment occurs

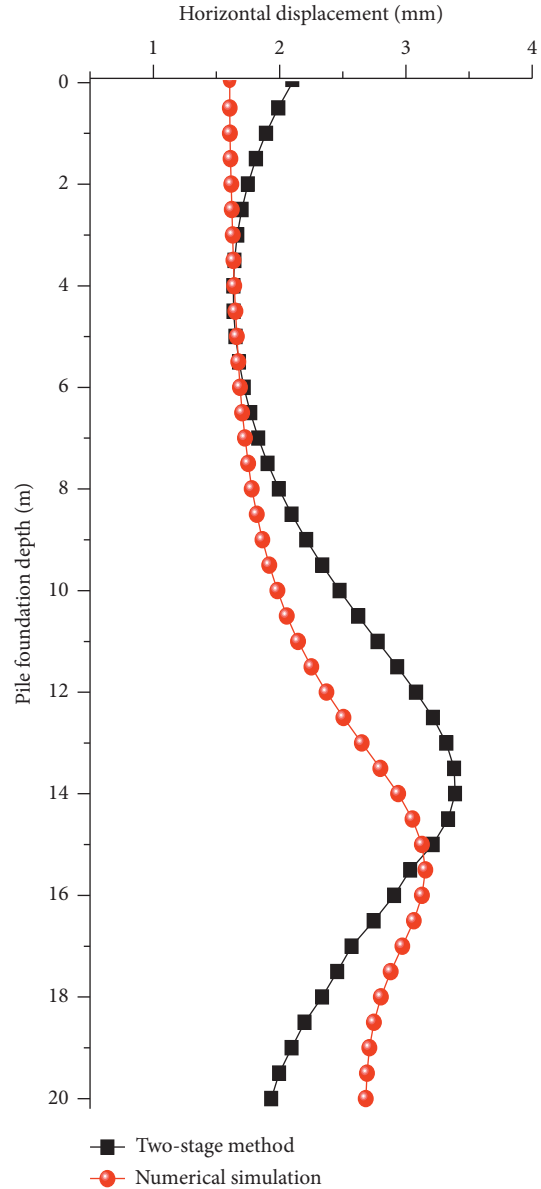


FIGURE 8: Horizontal displacement of pile.

near the burial depth of tunnel, and the tunnel side of pile is in tension, while the shear force reaches the maximum at the top and bottom of tunnel. The results of the two-stage method and numerical simulation are very close. The largest difference is at the peak value, the deviation rate of shear

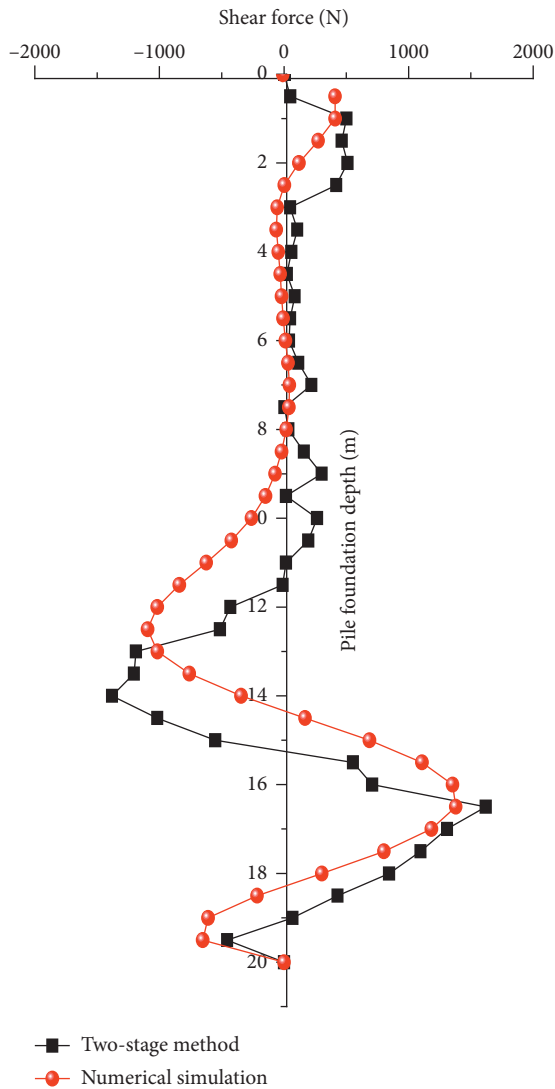


FIGURE 9: Shear force of pile.

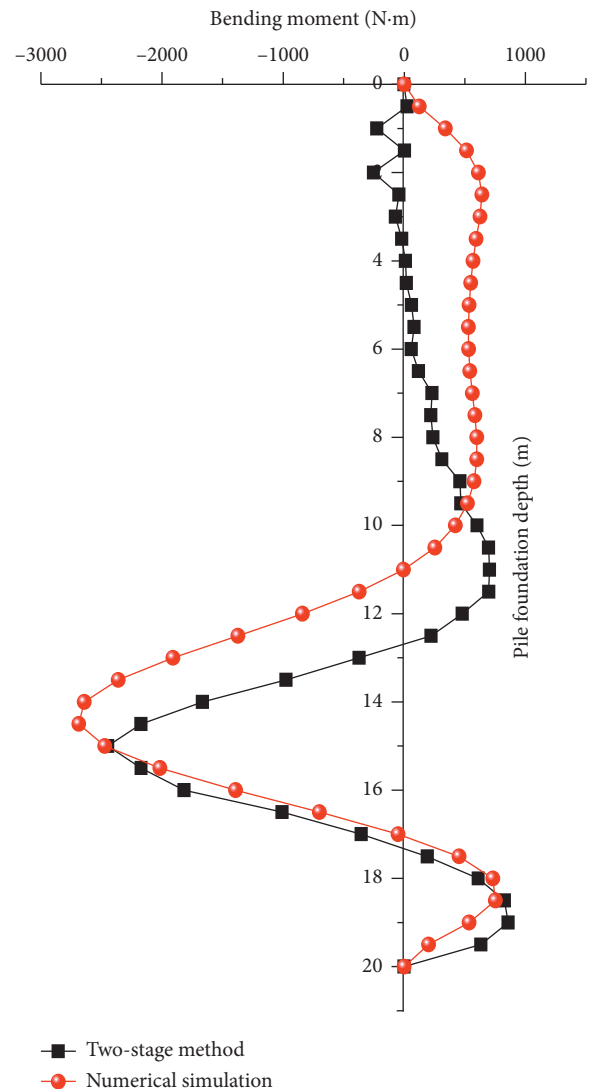


FIGURE 10: Bending moment of pile.

force is 26.14%, and the deviation rate of bending moment is 9.71%. In general, the additional displacement and internal force are at a reasonable and acceptable level.

4.3. Impact Factor Analysis of Pile. The impact degree of shield tunnel construction on adjacent pile foundation is closely related to the horizontal soil bed coefficient k'_h , the pile diameter D , and the distance between pile and tunnel S . Therefore, the analysis of relevant factors can provide effective guidance for design and construction.

The horizontal displacement difference between pile and tunnel Δs_x is an important indicator of impact degree. As shown in Figures 11 and 12, Δs_x increases with the increase of k'_h and D , and the increase rate of Δs_x is also increasing. When k'_h increases by 66.7% from baseline value 75 MN/m^3 , Δs_x increases by 40.56%. However, when D increases by 66.7% from baseline value 0.3 m, Δs_x increases by 216.53%. The impact of the pile diameter D on the horizontal displacement difference Δs_x is much greater than that of the horizontal soil bed coefficient k'_h . This shows that it is more

efficient to optimize the pile diameter than to improve the stratum. The permanent pile foundation should adopt the small diameter pile as far as possible. When the small diameter pile foundation cannot meet the stress requirements, the design scheme of pile group should be adopted. This will reduce the displacement difference between pile and stratum, reduce the additional internal force, and improve the stability of pile foundation.

With the increase of k'_h , the additional internal force of pile foundation increases gradually, but the increase rate decreases slightly, as shown in Figures 13 and 14. This shows that improving stratum is also an effective measure, but considering the high cost, it has a lower priority in practice. As we can see from Figures 15 and 16, the additional internal force of pile foundation is sensitive to the distance between pile and tunnel S . The additional moment of pile at $S=0.75 D$ is about 47% when $S=0.5 D$, and that of the additional shear is 42%. While S increases to $1.5 D$, the additional internal force decreases to about 5% when $S=0.5 D$. When $S \leq 0.75 D$, the change rate of the additional

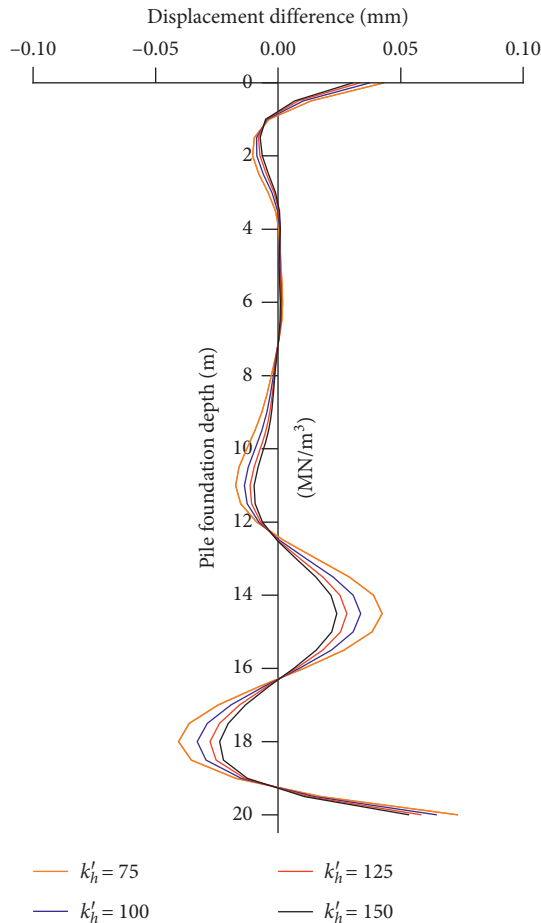


FIGURE 11: Relation between k'_h and Δs_x .

internal force is high. The change rate is relatively low at $0.75 D \leq S \leq 1.5 D$. While $S > 1.5 D$, the additional internal force stays at a lower level. Based on this rule of pile, the zone around shield tunnel is divided into strong impact zone of pile foundation ($S \leq 0.75 D$), slight impact zone ($0.75 D \leq S \leq 1.5 D$), and no impact zone ($S > 1.5 D$). The pile researched in this paper ($S = 0.83 D$) is in slight impact zone. When designing and constructing shield tunnel, the relative position of pile foundation should be avoided from the strong impact zone and should be in the slight and no impact zone as far as possible. If the pile foundation is unable to be in the strong impact zone, pre-reinforcement measures should be taken before construction. Stratum movement should be strictly controlled and monitored, reducing the additional internal force of pile foundation.

4.4. Case of Soft-Clay Pile. Due to the pile being surrounded by rock, the pile responses (Deflection, shear force, and bending moment) are not extremely dangerous. Therefore, a case of soft-clay in this section can give readers a more comprehensive understanding about the key mechanistic framework and, besides, expand the applicable range of this method. The physical and mechanical parameters of soft clay are shown in Table 2 [45]. The horizontal displacement,

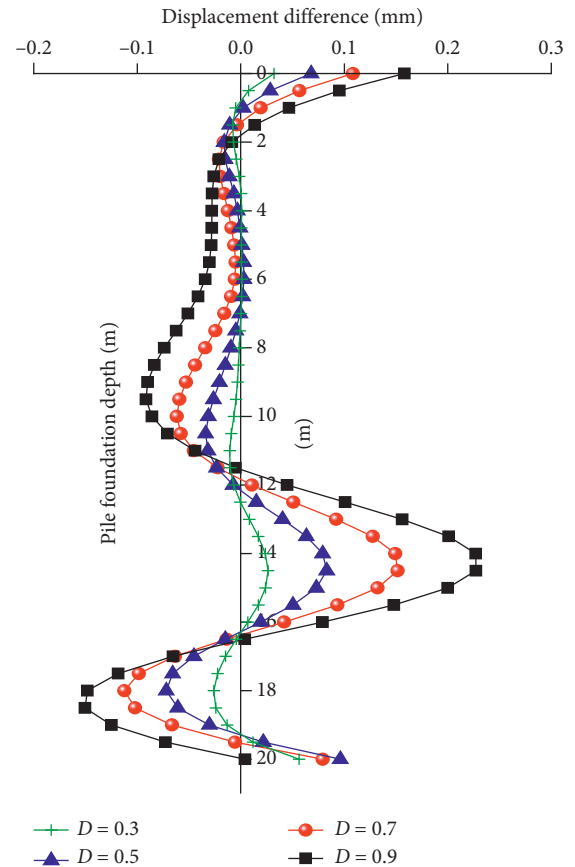


FIGURE 12: Relation between D and Δs_x .

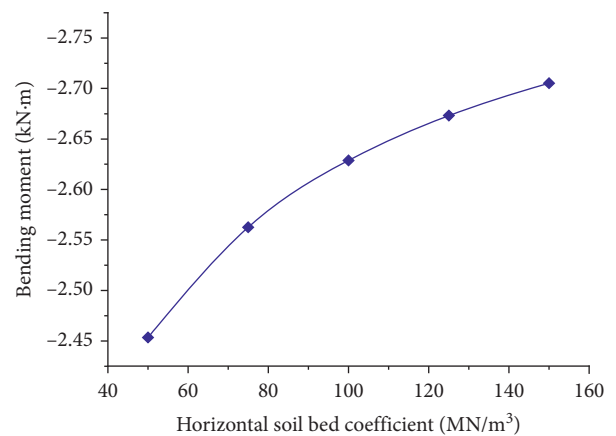


FIGURE 13: Relation between k'_h and M .

shear force, and bending moment of pile are shown in Figures 17–19, respectively.

From the figure, in soft soil, good consistency is shown between the two methods, which proves the large applicability of formulas. On the other hand, the maximum of additional displacement is bigger than 2 cm, and its internal forces significantly increase to a dangerous level, 210 kN·m and 250 kN. Generally, the designed bending moment and shear strength of pile like this size are about 200 kN·m and 200 kN, respectively, and the crack resistance strength is only

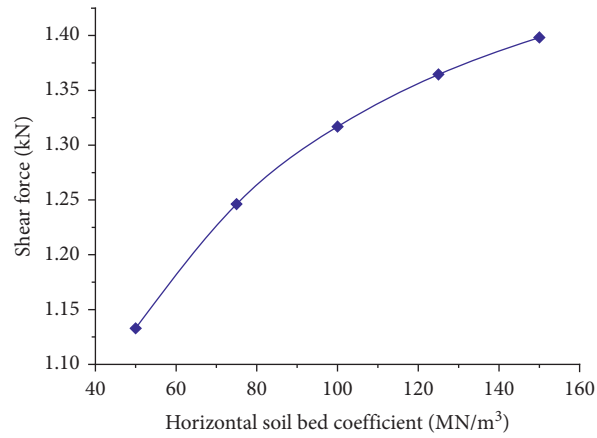


FIGURE 14: Relation between k'_h and Q .

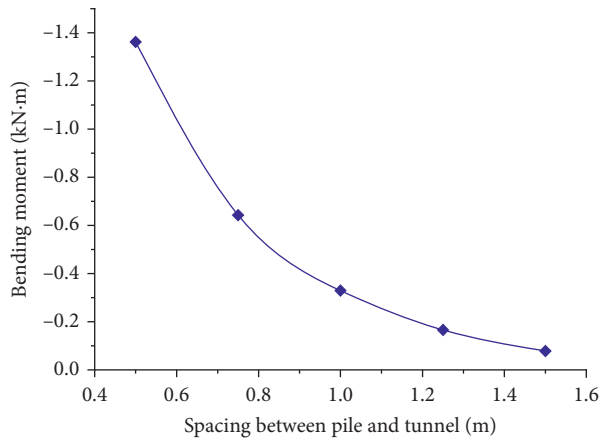


FIGURE 15: Relation between S and M .

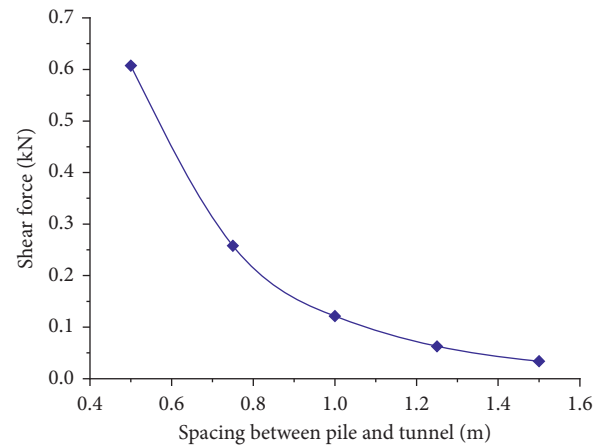


FIGURE 16: Relation between S and Q .

TABLE 2: Mechanical parameters of soft clay [45].

Material	Unit weight (kN/m ³)	Passion's ratio	Cohesion (kPa)	Friction angle (°)	Elastic modules (MPa)	Bed coefficient (kN/m ³)
Soft clay	18	0.3	3	20	5	5000

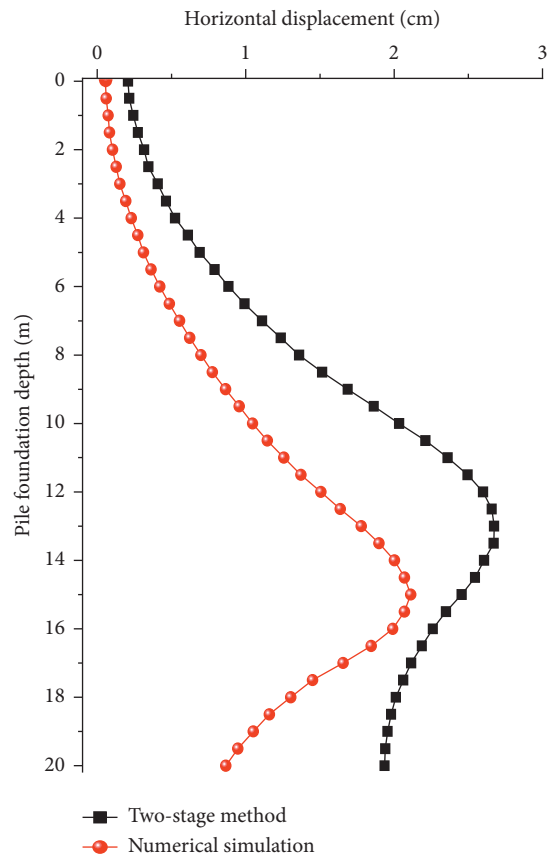


FIGURE 17: Horizontal displacement of pile in clay.

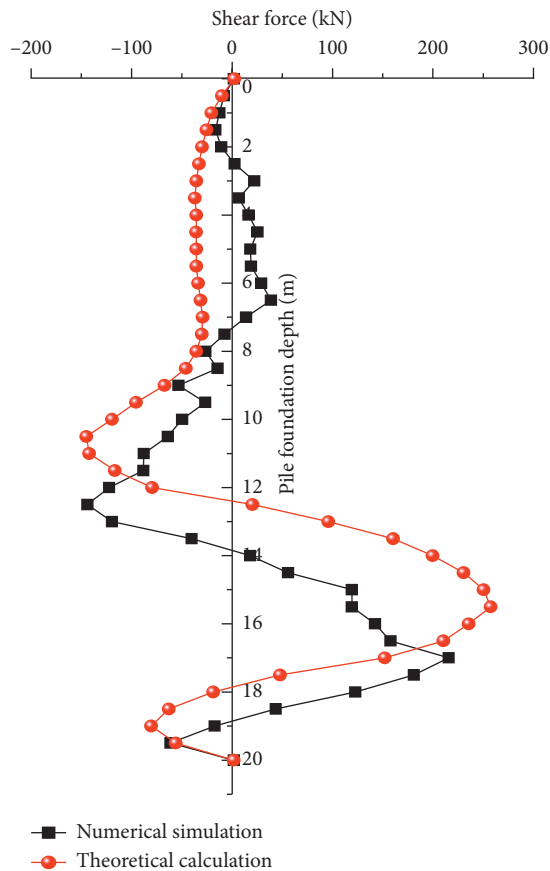


FIGURE 18: Shear force of pile in clay.

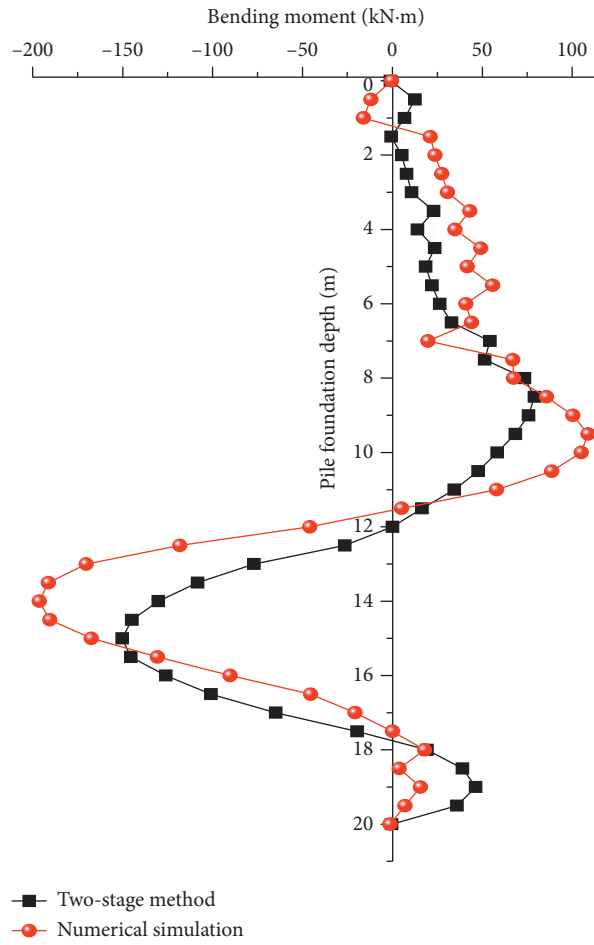


FIGURE 19: Bending moment of pile in clay.

TABLE 3: Burgers parameters of saturated clay.

Material	Elastic modulus E_M (MPa)	Viscoelastic modulus E_K (MPa)	Maxwell η_M (MPa·s)	Kelvin η_K (MPa·s)
Saturated clay	15	2.5	4.52×10^8	4.52×10^{13}

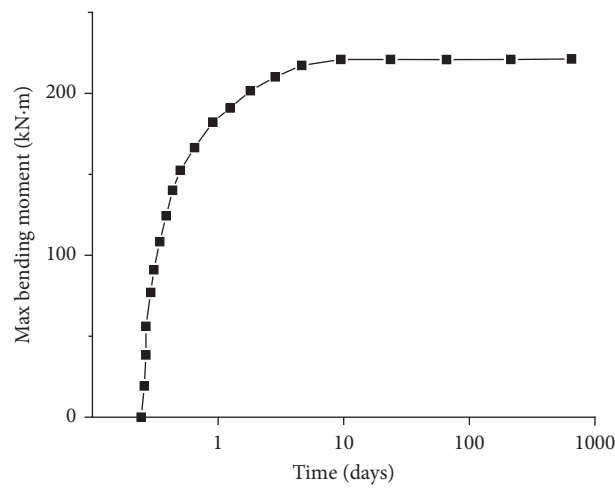


FIGURE 20: The maximum bending moment over time.

about half of that. This indicates that the pile is insecure under the impact of lateral soil movements, which will make it susceptible to future damages.

And it is noteworthy that excavation in saturated clay will always induce development of excess pore water pressure in the soil, resulting in the time-dependent behavior of pile-soil. The max BM may happen after the excavation and not necessarily during the excavation, which requires long-term analysis and observations always [46–49]. In this paper, Burgers creep model is used to study this process, and the parameters are shown in Table 3.

As shown in Figure 20, the maximum bending moment of pile in saturated clay shows a great time-dependent behavior, and the BM value increases significantly first, then gradually slows down, and finally stabilizes at 220 kN·m; it takes about 10 days. This is because the disturbance of shield excavation leads to the redistribution of pore water pressure in the surrounding soil, resulting in soil reconsolidation.

5. Conclusions

This paper presents a new method for impact analysis of shield excavation on an adjacent pile foundation. A modified Peck formula and a three-dimensional numerical model are firstly validated with the field monitoring data in ground settlement. Then, based on the validated parameters, the additional displacement and internal force of the pile foundation are evaluated through a two-stage method and numerical model. Finally, the impact degree of the horizontal soil bed coefficient, the pile diameter, and the distance between pile and tunnel are discussed. The following conclusions are drawn:

- (1) Not only the vertical settlement of soil caused by shield construction, but the horizontal displacement can also be calculated by Modified Peck formula. It can be used for many purposes, and the empirical parameters of formula are also convenient to use field monitoring data to back-analysis modification. In this paper, the good consistency between numerical simulation and theoretical calculation and monitoring data of ground settlement verifies the rationality of parameter selection.
- (2) In general, the additional displacement and internal force are at a reasonable and acceptable level. The results of the two-stage method and numerical simulation presented are generally close, but there are relatively large differences at bottom of the pile and at the peak displacement. This is because the shear stiffness between stratum spring elements is not considered in the single-parameter Winkler model, which will bring some errors to the deformation calculation. Two-parameter Pasternak model can be introduced to replace the single-parameter Winkler model adopted in this paper. It is worth mentioning that this will also lead to a more complex solution process and parameter validation. Therefore, the single-parameter Winkler model is chosen in this paper.

- (3) The impact of the pile diameter D on the horizontal displacement difference Δs_x is much greater than that of the horizontal soil bed coefficient k'_h . This shows that it is more efficient to optimize the pile diameter than to improve the stratum. The permanent pile foundation should adopt the small diameter pile as far as possible. When the small diameter pile foundation cannot meet the stress requirements, the design scheme of pile group should be adopted.
- (4) The additional internal force of pile foundation is sensitive to the distance between pile and tunnel S . Based on the change rule of pile internal force, the zone around shield tunnel is divided into strong impact zone of pile foundation ($S \leq 0.75 D$), slight impact zone ($0.75 D \leq S \leq 1.5 D$), and no impact zone ($S > 1.5 D$). The pile researched in this paper ($S = 0.83 D$) is in the slight impact zone. When designing and constructing shield tunnel, the relative position of pile foundation should be avoided from the strong impact zone and should be in the slight and no impact zone as far as possible.
- (5) In order to give a more comprehensive mechanistic framework and the applicable range of this method, a case in soft-clay is applied. The results show that the maximum of additional displacement is bigger than 2 cm, and internal forces significantly increase and even exceed the design strength. This indicates that the pile is insecure under the impact of lateral soil movements, which will make it susceptible to future damages. In saturated clay, this process shows obvious time-dependent behavior, and the maximum bending moment happens after the excavation rather than during the excavation. The final value is slightly larger than that of the elastoplastic method.

Data Availability

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

Conflicts of Interest

The authors declare no conflicts of interest.

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