Research Article
Wedge Instability Model Improvement of Shield Tunnel Excavation Face under Inclined Surface

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In the process of the underground tunnel excavation, a kind of geological condition Necking Region is often encountered. The ground surface inclines very fast, which also leads to the increase of Earth pressure on the excavation face. The determination of the excavation face support pressure is essential to solve the active Earth pressure when the shield passes through the Necking Region. In this paper, based on Horn’s wedge model, considering the influence of surface dip angle on excavation face support pressure, the traditional wedge model was improved. The analytical solution of the ultimate support pressure for the active failure of shield excavation face was derived. To evaluate the quality of the model, the theoretical model was compared with the ultimate bearing pressure of the horizontal surface test. The influence of the ultimate support pressure on the parameters of $N_c$, $N_r$, and $N_q$ was consistent with the results of finite element simulation and existing theories, which verified the rationality of the model. The stability of the excavation face of the Heyan road river crossing tunnel was analyzed by using the improved wedge model. The results show that the mud support pressure considering the slope angle was 36 kPa higher than that without considering the slope angle.

1. Introduction

The stability of the excavation face is very important for tunnel excavation. When the tunnel crosses the river and sea, almost all the tunnels are constructed by shield method. While the shield tunneling underground, to ensure the safety of the project, one of the key issues is to ensure the stability of the excavation face. The stability of shield tunnel excavation face is developed from the classical slope stability theory, and there are many research results. At present, two methods are used to analyze the stability of the tunneling surface. One is the limit analysis method [1–5], which is divided into the upper limit analysis method and lower limit analysis method. Due to the complexity of calculation, the limit analysis method is seldom used in engineering. The other method is a limit equilibrium method [6–8], which is widely used in engineering because of its simplicity and practicality.

The stability analysis of excavation face based on wedge instability model is the most common method for tunnel excavation under the horizontal surface. The wedge model proposed by Horn [9] is the earliest and most widely used excavation face support pressure solution mode, which is based on the equilibrium method. The essence of the theory is to use Coulomb’s static equilibrium theory for reference, assume the sliding surface of the tunneling surface, establish the mechanical balance equation of the excavation face based on Janssen’s [10] silo theory, and solve the excavation face support pressure. Anagnostou and Kovár [6] and Anagnostou and Kovár [11] systematically deduced Horn’s wedge model based on the Mohr–Coulomb failure criterion and modified the side length of the prism above the wedge. Broere [12], Zhang et al. [13], and Chen et al. [8] proposed the wedge-shaped instability model of excavation face under the horizontal plane and improved the applicability of...
Horn’s model in different strata. However, these methods assume that the overburden above the excavation face is horizontal, and Horn’s model under the inclined surface is not modified. In the process of underwater tunnel excavation, there is often a geological condition, namely, a necking area. The surface of the necking area section of the river crossing tunnel inclines rapidly, and the thickness of the soil on the top of the tunnel changes. The change of overburden thickness of the tunnel has a great influence on the support pressure of the excavation face.

The influence of overburden thickness on the support pressure of the excavation face has been studied. Among them, Chen et al. [7] considered that when the ratio of overburden thickness to tunnel diameter is less than 1.0, the support pressure increases significantly with the increase of overburden thickness, which is consistent with the conclusion revealed by Anagnostou and Kovář [6]. When the ratio of overburden thickness to tunnel diameter is greater than 1.0, the support pressure increases slowly with the increase of overburden thickness, which is slightly different from the results of Anagnostou and Kovář [6], Anagnostou and Kovář [11], Vermeer et al. [14], and Li et al. [1]. But it is consistent with the results of the centrifugal model test [15]. He et al. [16] also obtained through theoretical derivation that the support force of tunnel excavation face increases nonlinearly with the increase of surface inclination angle. All these showed that it is necessary to adjust the support pressure of the excavation face by considering the surface slope when the shield tunnel passes through the necking area.

In addition, the study of the influence of surface dip angle on slope stability also has a certain reference value. Based on the wedge-shaped static equilibrium equation, Wu et al. [17], Xu et al. [18], and Xiong et al. [19] all believe that the Earth pressure of inclined surface is greater than that of the horizontal surface. We can see from here. The influence of surface dip angle on the ultimate pressure of the excavation face cannot be ignored. On the basis of the limit equilibrium method considering the surface dip angle, it is urgent to improve the existing wedge prism model.

Considering the traditional wedge model, this paper proposes an improved model considering the slope angle of the ground surface. The effect of surface inclination on the ultimate pressure of tunneling surface was researched, and the ultimate pressure of tunneling surface under the inclined surface was calculated. Based on the improved model, the theoretical solution of the limit ultimate pressure for the active failure of the excavation face was derived. For the purpose of verifying the rationality of the improved model, the ultimate bearing capacity of the theoretical model is compared with that of the existing tests under the horizontal surface. The influence of ultimate bearing pressure on the parameters of \(N_p\), \(N_q\), and \(N_s\) was basically consistent with the experimental results of available references, which verified the validity of the improved model. The stability of the tunneling surface of Heyan road river crossing tunnel was analyzed by using the improved wedge model, and the influence of surface dip angle on the ultimate bearing pressure was obtained. The ultimate bearing capacity calculated by the improved wedge model was basically consistent with the finite element results.

### 2. Finite Element Simulation of Shallow Shield Working Face under Inclined Ground Surface

For the intention of analyzing the effect of the slope angle on the stability of excavation face, this section used the elastic-plastic model finite element of Mohr–Coulomb failure criterion to simulate the ultimate support pressure of shield tunnel face, and the instability form of excavation face under the condition of ground surface tilt is analyzed. In order to compare with the existing research, in addition to the surface dip angle, other parameters are also selected according to the tunnel parameters studied in Vermeer et al. [14] and Lu et al. [20].

As shown in Figure 1, among them, \(D = 10\ m\) was selected as the tunnel diameter. \(C = 10\ m\) was selected as the buried depth. \(H = 30\ m\) was selected as the distance between the river water level and the top of the tunnel. The density of soil \(\gamma = 17\ kN/m^3\), Young’s Modulus \(E = 30\ MPa\), the cohesion \(c = 30\ kPa\), internal friction angle \(\phi = 35^\circ\), and Poisson’s \(\nu = 0.3\). The parameters of the finite element model were shown in Table 1. The model is meshed using an 8-node hexahedron element with pore water pressure. The model’s OYZ face is horizontal restraint in the X direction. The OXZ plane is horizontal restraint in the Y direction, and the half-plane of tunnel excavation is symmetrically constrained. The bottom OXY is fixed. The influence of tunnel excavation settlement on the stability of the excavation face is not considered here, and the internal of the excavated tunnel is fixed and constrained.

The numerical simulation is divided into three steps as follows:

1. **In situ stress balance:** the initial stress caused by soil weight and load is calculated, and then the in situ stress balance is carried out.

2. **Tunnel and excavation support:** remove the stratum unit in the excavation area and apply the initial support pressure \(\sigma_{	ext{so}}\) evenly distributed on the face of the tunnel. Generally, the static Earth pressure is selected as the upper limit of excavation face support pressure. Therefore, the static Earth pressure is used as the initial support force \(\sigma_{	ext{so}}\) to maintain the stability of the tunneling surface. \(\sigma_{	ext{so}} = K_0[q + \gamma(C + (D/2))] K_0 = 1 - \sin \phi\) [14, 21]

3. **Using dichotomy method to reduce excavation support pressure:** \(\sigma_{	ext{so}}\) [14, 21]: the displacement changes in front of the excavation face are recorded until the face collapses. The limit support pressure value of the tunneling surface and the displacement nephogram of the tunneling surface are obtained, and the instability mode of the tunneling surface is determined by the displacement nephogram when the excavation face is unstable.

As shown in Figure 2, in order to obtain the effect of the slope angle on the constancy of the excavation face, the slope angle is set to 10°, 20°, and 30° in the numerical simulation. Figure 2(a) shows the numerical simulation results of the volatile patterns of the excavation face when the ground is
Table 1: The parameters of the finite element model.

| The parameters | Density \( \gamma \) (kN/m\(^3\)) | Young’s modulus \( E \) (MPa) | Cohesion \( c \) (kPa) | Internal friction angle \( f \) (°) | Poisson’s \( \nu \) |
|----------------|---------------------------------|--------------------------------|-----------------|----------------|----------------|
| The value      | 17                              | 30                            | 30              | 35             | 0.3            |

Figure 1: The 3D finite element mesh for the stability analysis of shield tunnel.

Figure 2: The Instability model of excavation face based on numerical simulation: (a) numerical simulation in the literature [21], (b) ground tilt angle 10°, (c) ground tilt angle 20°, and (d) ground tilt angle 30°.
horizontal. In Figure 2(a), the volatile area ahead of the excavation face is wedge-shaped regions, and the instability regions above the wedge-shaped regions are prism area.

Figures 2(b)–2(d) show a triangular failure pattern in front of the excavation as shown in the side view of the instability pattern at three different ground inclinations. In space, it is an unstable mode with a wedge shape, as shown in Figure 3(a). The instability mode of the excavation surface is similar to the traditional wedge model, except that the upper boundary of the wedge is almost the same as the slope angle. According to the Horn wedge model, once the shape of the boundary of the wedge is almost the same as the slope angle, the upper boundary of the wedge is almost the same as the slope angle. This provides a new idea for the improvement of the wedge-shaped model. Referring to the results of the numerical model, an improved wedge-based model is proposed in the third section. In addition, this paper also uses the numerical method to analyze the influence of different cohesion $c$, internal friction angle $\phi$, and the ratio of overburden thickness to tunnel diameter $C/D$ on the support pressure, which will be compared with the theoretical model in the fourth section.

3. Minimum Limit Support Pressure of Excavation Face Based on Improved Wedge Instability Model

Therefore, for the intention of estimating the ultimate bearing pressure of the tunneling surface under the inclined surface more accurately, a modified three-dimensional wedge prism model was proposed. As shown in Figure 4, the black area is the original model (Horn’s model). The yellow area and the blue area are the modified wedge model. In the modified model, the instability model area also includes two parts: one is the wedge $abcdefg$ in front of the tunneling surface, and the other is the prism $edfgnmkl$ above the wedge. The prism acts on the wedge and is affected by gravity (see Figure 3(a)). The fracture angle $\theta$ of the improved model with a different surface dip angle is basically the same. The upper boundary of the wedge is almost the same as the surface tilt angle.

$$\sigma_v = \frac{Ay - 2c(B + C)}{2K_0(B + C)\tan\phi} \left[1 - e^{-2K_0(B+C)\tan\phi/A}z\right] + qe^{-2K_0(B+C)\tan\phi/A}z. \quad (2)$$

Then, the force $V$ above the wedge is

$$V = A\sigma_v. \quad (3)$$

The following is the mechanical balance analysis of the wedge slider. The force analysis diagram of the wedge slider is shown in Figure 5.

It is assumed that the formation material is homogeneous and the formation tallies with the Mohr–Coulomb fatigue damage criterion. The ultimate bearing pressure is considered to be the maximum required bearing pressure for wedges corresponding to different ground inclination angles $\alpha$ or $\beta$. It should be noted that the effect of seepage is not considered in this paper, but the water head is applied to the formation surface as a stress boundary condition. The unit weight of the soil below the water surface is expressed as effective gravity. Then, the improved model is analyzed theoretically.

3.1. Establishment of Improved Active Wedge Instability Model

The computational analysis of the model is shown in Figures 3 and 5. In Figure 3(a), $B$ is the width of the prism, $C$ is the length of the prism, $H$ is the depth of the tunnel, $D$ is the diameter of the tunnel, $\alpha$ is the inclination of the ground surface, and $\sigma_v$ is the loose Earth pressure. By establishing the force balance of the slide in front of the tunneling surface, the calculation formula of ultimate bearing pressure is established.

When analyzing the static equilibrium of the wedge-shaped slide block, the Earth pressure $\sigma_v$ above the tunnel needs to be calculated. In order to coordinate with the cross-section shape of the wedge-shaped slide block in the wedge-shaped body model, the loose soil area above the sliding area of the excavation face was assumed to be a cylinder; it was also called the prime region. As shown in Figure 3(a), $c$ is the depth of the analysis point. According to the force balance of the soil column element, the following results can be obtained:

$$A\sigma_v + Aydz = A(\sigma_v + d\sigma_v) + 2(B + C)(c + K_0\sigma_v \tan\phi)dz.$$ \hspace{1cm} (1)

In the formula, $y$ is the weight of the soil; $A = B \times C$ is the cross-sectional area, $B = (\sqrt{3}/2)D$ is the width of the prism, $C = (D \sin(90 - \theta)/\sin\theta) \sin(\theta - \alpha) = (\cos\theta/\sin(\theta - \alpha))D$ is the length of the prism, $c$ is the cohesion of the soil, $\phi$ is the angle of internal friction of the soil, and $K_0$ is the coefficient of lateral Earth pressure. According to Vermeer et al. [14], $K_0' = 1 - \sin\phi$, and $\alpha$ is the angle of the Earth’s surface.

By introducing the boundary condition $\sigma_v|_{z=0} = q$, the loose Earth pressure at any depth is

$$P \sin\theta - N + (G + V)\cos\theta = 0,$$

$$P \cos\theta + T + 2T_v - (G + V)\sin\theta = 0. \quad (4)$$
Then,

\[ P = \frac{(G + V)(\sin \theta - \cos \theta \tan \varphi) - (2T_s + c_{abfg})}{\sin \theta \tan \varphi + \cos \theta} \]  \quad (5)

In the formula, \( G \) is the weight of the wedge, \( P \) is the limit support force, \( N \) and \( T \) are the normal force and the friction forces between the wedge side \( abfg \) and the stratum, respectively, \( T_s \) is the friction force between wedge slope \( adf \) or \( beg \) and stratum, and \( N_{eg} = S_{beg}K_S \left( \sigma_y + \left( \frac{B}{3}\right) \gamma \right) \tan \phi \) is the normal force on lateral wedge \( adf \) or \( beg \).

The gravity force of the wedge \( G \) is

\[ G = \frac{1}{2} \gamma B S_{beg} \]  \quad (6)

\( T \) and \( T_s \) are the friction force between the wedge slope and the stratum, which are obtained by the product of the area \( abfg \) and \( adf \) between the slope and the stratum and the shear strength.
By solving equation (5), the ultimate bearing pressure can be obtained:

\[
T_s = \left[ c + K_0 \left( \sigma_v + \frac{B}{3} \gamma \right) \right] S_{beg},
\]

(7)

\[
T = cS_{abfg} + N \tan \varphi.
\]

When the surface inclination angle \( \alpha = 0 \), the derived formula (8) can also be obtained from the previous study [6,8], which verifies the feasibility of the model-derived results.

3.2. Superposition of the Limit Support Pressures. By substituting equation (2) into equation (8), the formula for calculating the ultimate bearing pressure of the excavation face was obtained. According to the superposition method widely used in support pressure analysis and the superposition method recently used to calculate the lateral Earth pressure of retaining wall [22], the obtained ultimate bearing pressure could be realigned as cohesion, overload load, and soil weight multiplied by its effect parameters:

\[
\sigma_s = cN_c + qN_q + \gamma DN_r.
\]

The coefficient of the cohesion is

\[
N_c = \left( \frac{4 \cos \theta + \pi}{\pi (\sin \theta \tan \varphi + \cos \theta)} \right) \left( \frac{1 - \exp \left( -\left( \frac{2}{K_0 \tan \varphi + \cos \theta} \right) \right)}{1 - e^{-\left( 2K_0 (B+C) \tan \varphi \gamma / \alpha \right)}} \right),
\]

(10)

where \( C \) is the thickness of the soil above the tunnel top.

The coefficient of the surcharge load is
The numerical results of ultimate bearing capacity were obtained. For the purpose of obtaining reasonable calculation results, this section used the numerical model in Section 2 to establish three series of numerical simulations.

In the purpose of verifying the rationality of the theoretical analysis, it is essential to obtain the influence of the above coefficients on the ultimate support pressure. Some scholars studied the influence of the friction angle under the horizontal surface on the correlation parameters by numerical simulation. Section 4.1 verified the rationality of the theoretical model by comparing the results of parameter analysis under the condition that the inclination angle $\alpha$ was 0 and the existing horizontal surface ($\alpha = 0$). In the purpose of analyzing the influence of other formation parameters on the support pressure of excavation face, Section 4.2 took the formation dip into account, and the relationship between soil mass $\gamma$, the cohesion $c$, the cover soil thickness and shield diameter ratio $C/D$, and the excavation face support pressure was analyzed.

### 4.1. Coefficient Analysis of the Limit Support Pressures

The influence of surface dip angle on the stability parameters of tunneling surface was analyzed in this section. It could be seen from equation (9) that if any of the values of $c$, $q$, and $\gamma$ are set to zero, the coefficient of the other nonzero term could be obtained. For the purpose of obtaining reasonable results, this section used the numerical model in Section 2 to establish three series of numerical simulations.

In the first group of numerical simulation, when $c$ and $q$ were set to 0, only soil cohesion changes. By substituting the ultimate bearing capacity into equation (9), the relationship between $N_c$ and formation dip was obtained. The results were shown in Figure 6(a). In the latter two groups of numerical simulation, the other two parameters were set to 0. The numerical results of ultimate bearing capacity were given. $N_q$ was obtained in the same way. The relationship between $N_y$, $N_q$ and formation dip was obtained. The results were shown in Figures 6(b) and 6(c).

The relationship between $N_c$, friction angle, and formation dip was shown in Figure 6(a). In general, with the ascend of the friction angle $\phi$, $N_c$ increases. With the ascend of the ground surface inclination angle $\alpha$, $N_c$ decreases. When the value of $\phi$ was small, the absolute value of $N_c$ was larger, which indicated that the surface dip angle $\alpha$ had a great influence on $N_c$. With the ascend of $\phi$, the absolute value of $N_c$ descends, which indicates that the influence of the ground surface inclination angle $\alpha$ on $N_c$ decreases. When the surface dip angle is 0, the result is basically consistent with that in Vermeer et al. [14]. The rationality of the assumed model is explained. When the friction angle is between (15, 20), the numerical results given by the wedge model are basically consistent with the numerical simulation results. When the friction angle is greater than 20°, it can be seen from the calculation results that the calculation results based on the improved model are slightly different from the finite element simulation results, and the deviation ascends with the increase of friction angle.

The relationship between $N_q$, friction angle, and formation dip was shown in Figure 6(b). In general, $N_q$ decreased with the increase of friction angle and increased with the increases of the ground surface inclination angle $\alpha$. When the value of $\phi$ was small, the value of $N_q$ was larger, which indicated that the influence of surface dip angle $\alpha$ on $N_q$ is greater. With the increase of $\phi$, the value of $N_q$ decreases, which indicated that the influence of surface inclination $\alpha$ on $N_q$ decreases. When the surface dip angle was 0, the result was basically consistent with that in Vermeer et al. [14]. The rationality of the assumed model was explained. When $\phi$ reached a certain value (about 30°), $N_q$ tended to zero. When the friction angle was between (30, 40), the numerical results given by the assumed wedge model were basically consistent with the element simulation results. While the friction angle was less than 30°, the difference between the calculated results and the element simulation results is large, and the deviation increases with the decrease of the friction angle.

The relationship between $N_y$, friction angle, and formation dip angle was shown in Figure 6(c). In general, $N_y$ decreases with the increase of friction angle and increases with the increase of ground surface inclination angle. When the value of $\phi$ was small, the value of $N_y$ was larger, which indicated that the
influence of surface dip angle \( \alpha \) on \( N_c \) was greater. With the increase of \( \varphi \), the value of \( N_c \) decreased, which indicated that the influence of surface dip angle \( \alpha \) on \( N_c \) decreases. When the surface dip angle was 0, the result was basically consistent with that in Vermeer et al. [14]. The rationality of the assumed model was explained. When \( \varphi \) reached a certain value (about 35°), \( N_q \) tends to zero. When the friction angle was between (35, 40), the results given by the assumed wedge model were basically consistent with the numerical simulation results. When the friction angle was less than 35°, the difference between the calculated results and the numerical results was large, and the deviation increases with the decrease of the friction angle.

4.2. The Relationship between the Active Pressure and the Ground Inclination. With the purpose of analyzing the effects of other coefficients on the support pressure of the excavation face, the numerical model of Section 2 was adopted in this section. Three groups of analysis parameters were selected: the soil density \( c \), cohesion \( c \), and the ratio of overburden thickness to shield diameter \( C/D \). Nine series of numerical simulations were used to analyze the relationship between them and the abutment pressure of the tunneling surface.

The relationship between the slope angle and the support pressure of the tunneling surface was considered only by changing the soil gravity. As shown in Figure 7. In general, the

![Graph](image-url)
active Earth pressure increased with the increase of formation density. The increased range of soil weight $c$ is 5 kN/m$^3$, and the abutment pressure of the tunneling surface increases about 50 kPa. When the density is higher ($c > 25$ kN/m$^3$), the slope of support pressure increases slightly; that is, the increased range of active Earth pressure becomes larger.

The slope of active Earth pressure increased when the friction angle was less than 15° significantly higher than that when the friction angle greater than 15°, and there was a rapid increase trend when the slope angle and friction angle were the same. It could be explained that the closer the surface inclination angle is to the formation friction angle, the greater the ultimate pressure of the tunneling surface is, and the accurate support control is more difficult. In the wake of the increase of friction angle, the ultimate pressure becomes smaller and smaller. Because of the upper limit of formation friction angle, the effects of friction angle on support pressure were limited.

When the internal friction angle was the same, with the increase of formation density, there was little difference between the results of finite element simulation and improved theoretical analysis, which showed that the error of numerical simulation and theoretical analysis is determined by the method itself and had little relationship with formation density and friction angle.

As shown in Figure 8, the base of cohesion was 3.5 kPa, and then the cohesion was increased by 10 times (35 kPa), 30 times (105 kPa), 60 times (210 kPa), and 100 times (350 kPa), respectively. Five groups of experimental data are analyzed. Five groups of numerical simulation results indicated that the active Earth pressure decreases with the increase of
Figure 8: The relationship between active pressure and surface inclination angle under different cohesive forces: (a) the cohesion of soil $c = 3.5 \, \text{kPa}$ ($\gamma = 20 \, \text{kN/m}^3$), (b) the cohesion of soil $c = 35 \, \text{kPa}$ ($\gamma = 20 \, \text{kN/m}^3$), (c) the cohesion of soil $c = 105 \, \text{kPa}$ ($\gamma = 20 \, \text{kN/m}^3$), (d) the cohesion of soil $c = 210 \, \text{kPa}$ ($\gamma = 20 \, \text{kN/m}^3$), and (e) the cohesion of soil $c = 350 \, \text{kPa}$ ($\gamma = 20 \, \text{kN/m}^3$).
As shown in the first three groups of data, when the cohesion was small, the active Earth pressure was negatively correlated with the angle of internal friction. In the last two groups of data, when the cohesion was large, the active Earth pressure was proportional to the internal friction angle. The calculation method of active Earth pressure in slope stability analysis was used for reference. When Rankine Earth pressure was used to calculate active Earth pressure, 

$$\sigma_a = \gamma z K_a - 2\sqrt{K_a}$$  

$$\sigma_a = \gamma z K_a - 2c$$  

When $\gamma$ is fixed, the coefficient of active Earth pressure is closely related to cohesion $c$. When $\sqrt{K_a}$ is less than $c/\gamma z$, $\sigma_a$ decreases with the increase of $\sqrt{K_a}$. When $\sqrt{K_a}$ is greater than $c/\gamma z$, $\sigma_a$ increases with the increase of $\sqrt{K_a}$. When the cohesion $c$ increases, $\sqrt{K_a}$ will gradually be less than $c/\gamma z$, which also explains why the active Earth pressure of the latter series decreases with the decrease of internal friction angle. Of course, if the active Earth pressure coefficient could be expressed by cohesion and internal friction angle, an extreme value of cohesion could also be obtained.

It could be seen from the first two groups of figures that the influence of cohesion on the support pressure of the excavation face is similar to that of gravity. As the friction angle was less than $25^\circ$, the increased slope of active Earth pressure was obviously higher than that of friction angle greater than $25^\circ$, and the increase of active Earth pressure had a rapidly increasing trend. It showed that the closer the surface dip angle is to the formation friction angle, the greater the ultimate pressure of the tunneling surface and the greater the support difficulty. With the increase of friction angle, the support pressure became smaller and smaller.

When the formation density was the same, with the increase of cohesion, the finite element simulation results and the improved theoretical analysis results showed a trend of first increase and then decrease, which showed that
cohesion has an obvious influence on both numerical simulation and theoretical analysis.

As shown in Figure 9, the influence of tunnel diameter ratio on the limit pressure of tunnel surface is similar to that of gravity. Three groups of test results show that the active Earth pressure is positively correlated with the tunnel diameter ratio. When the hole diameter ratio is constant, the active Earth pressure is negatively correlated with the internal friction angle and positively correlated with the surface dip angle.

As shown in Figure 9(a), when the tunnel diameter ratio is 0.5 and the surface inclination angle changes from 0 to 25°, the maximum support pressure increases by 30 kPa. As shown in Figure 9(b), when the tunnel diameter ratio is 1 and the surface inclination angle changes from 0 to 25°, the maximum support pressure increases by 45 kPa. As shown in Figure 9(c), when the tunnel diameter ratio is 2 and the surface inclination angle changes from 0 to 25°, the maximum support pressure increases by 40 kPa.

It shows that when the tunnel diameter ratio is small, the influence of the surface tilt angle on the support pressure of the excavation face is large, and when the tunnel diameter ratio is large, the influence of the surface tilt angle on the support pressure of the excavation face will be reduced.

5. Application in HeYanlu Cross-River Shield Tunnel

In this paper, the improved wedge considering the surface inclination was used to reckon the ultimate bearing pressure of the HeYanlu river crossing the tunnel. Nanjing Heyan road cross-river tunnel is located in Qixia District, Nanjing City, Jiangsu Province, China. The tunnel starts at BaGuaZhou of PuYi and ends at the intersection of Heyan road and YanHeng road. The tunnel is 2965 m long, with an outer diameter of 14.5 m. The tunnel mileage is ZK1 + 726 ~ ZK4 + 691. The lowest position of the tunnel is 11 m away from the water surface, the highest water level is 50 m, the excavation footage is 2 m/ring, the lithology of soil layer is a strongly weathered breccia, elastic model $E = 60$ MPa, internal friction angle is 30°, density is 2.0 g/cm³, and cohesion is 30 kPa.

Along the excavation direction, two typical sections (as shown in Figure 10) were selected to study the variation of ultimate bearing pressure. For the purpose of research, the effects of the ground dip angle on the stability of the face, the ultimate pressure under the horizontal stratum of section I and the inclined stratum of section II was calculated. The statistics of formation parameters of the two sections were shown in Table 1. The ultimate pressure of the excavation face was calculated by using the improved wedge model method, the finite element numerical simulation method, the Rankine Earth pressure method, and the Coulomb Earth pressure method.

It could be seen from Table 2 that the analytical solution of the ultimate bearing pressure was consistent with the numerical results. When the surface was horizontal, Rankine overestimated the ultimate bearing pressure of the tunneling surface. When the ground surface was inclined, Rankine underestimated the support pressure of the tunneling surface and overestimated the support pressure of the excavation face by the method of Coulomb Earth pressure plus additional pressure. The reason was that Coulomb Earth pressure was mainly used in the calculation of slope Earth pressure. When solving the slope Earth pressure, all the soil gravity above the fracture angle was assumed to be unstable soil, which was inconsistent with the assumption of shield tunnel. Therefore, the effects of the slope of the surface on the support pressure of the excavation surface could not be ignored.

6. Conclusions

In this paper, the relationship between the slope angle of the ground and the ultimate bearing pressure of the shallow tunnel face was simulated by the three-dimensional elastoplastic finite element method. The failure model of the excavation face considering the stratum dip angle was established. Based on the improved three-dimensional wedge instability model, the calculation method of the
ultimate bearing pressure of the excavation face was proposed. The following four conclusions are obtained.

(1) An improved three-dimensional wedge model was proposed to calculate the ultimate bearing pressure of the excavation face under an inclined ground surface, and the analytical solution of the ultimate bearing pressure for the active failure of shield excavation face is derived. To evaluate the quality of the improved model, the theoretical model was compared with the ultimate bearing pressure of the horizontal surface test. The influence of ultimate support pressure on \(N_c, N_p, \) and \(N_q\) parameters was consistent with the existing literature, which verified the rationality of the model.

(2) The stability of the tunneling surface of Heyan road cross-river tunnel was analyzed by the improved wedge model, and the influence of surface inclination on the ultimate bearing pressure was obtained. The ultimate bearing pressure calculated by the improved wedge model was basically consistent with the numerical calculation results.

(3) In practical projects, the inclined surface in the middle of the river was mostly gully section, and the thickness of the overburden is small. Considering the influence of surface slope on support pressure could help to reduce the impact of the inaccurate setting of support pressure on the overlying soil splitting failure.

(4) This paper did not consider the effect of seepage on support pressure, which would be considered in the future research.

Data Availability

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

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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