Research Article

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Limit Support Pressure of Tunnel Face in Multi-Layer Soils Below River Considering Water Pressure

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Abstract: This paper presents a method to determine the limit support pressure of tunnel face in multi-layer soils below river considering the water pressure. The proposed method is based on the 3D Terzaghi earth pressure theory and the wedge theory considering the water pressure. The limit support pressures are investigated using the limit equilibrium method and compared to those calculated using a numerical method, such as FLAC3D. Four cases focusing different combinations of three layers are analyzed. The results obtained by the numerical method agree well with the predictions of the proposed limit equilibrium method. The limit support pressure obtained using the limit equilibrium method is greater than that obtained by the numerical method. The limit equilibrium method is safe and conservative in obtaining the limit support pressure. The proposed limit equilibrium method is expected to be easily adaptable and to enhance the reliability of tunnel design and construction in multi-layer soils below river.

Keywords: Tunnel face; Limit support pressure; Multi-layer soils; Limit equilibrium method; FLAC3D

1 Introduction

With a rapid increase in the construction of subways, shield tunnel technology has been widely used in the construction of tunnels [1–3]. Tunnel face stability is a key construction consideration in shield tunnel projects. Com-

plex geological conditions can pose serious technical challenges to shield tunnel excavation [4]. The support pressure counteracts the effective earth and water pressure. In order to ensure safety against the collapse of the tunnel face, it is essential to have a reliable analysis of tunnel face stability and predictions of the limit support pressure.

The limit support pressure of the tunnel face is the primary concern in stability analysis [5, 6]. The limit equilibrium method [7], numerical method [8, 9], and experimental method [10] are used to analyze the stability of the tunnel face and support pressure. For the experimental aspects on the stability of the tunnel face, Zhou et al. [11] used the in situ method to analyze the deformation that takes place close to the tunnel face. Messerli et al. [12] investigated the limit support pressure of the tunnel face in sand using experiments. Idinger et al. [13] used centrifuge model tests to investigate face stability on a small scale tunnel in cohesion-less soil, with the slip surface arising from a tunnel invert propagated at an angle of approximately $45^\circ + \varphi/2$ to the horizontal. Chambon et al. [14] also studied tunnel face stability using a centrifuge test, and analyzed the effect of tunnel depths on the collapse forms of tunnel faces.

Numerical models are increasingly being used to assess the stability of tunnel faces. Zhang et al. [15] used a 2D numerical approach to study face stability during the slurry shield tunneling process. Chen et al. [16] constructed a 3D discrete element method to analyze the face stability of shallow shield tunnels in dry sand. Zhang et al. [17] adopted 3D numerical simulations to analyze the effects of diameter-to-depth ratios and soil properties on the tunnel face stability. Hasanpour et al. [18] simulated the single shield, double shield, and universal double shield for excavation of deep long tunnels. Graziani et al. [19] considered the creep effects for the Brenner Base Tunnel. Liu et al. [20] conducted a numerical analysis to investigate the influence of different construction schemes on the tunnel heave.

The wedge limit equilibrium model has become very popular in calculating the stability of the tunnel face [21–24]. Lee et al. [25] revealed the influence of steel pipe-

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reinforced multi-step grouting on tunnel face stability using the limit equilibrium method. Xu et al. [26] calculated support pressure using the nonlinear Mohr-Coulomb failure criterion. Broere et al. [27] calculated the required limit support pressure using a time-dependent groundwater flow water model and the limit equilibrium wedge model. Leca et al. [28] assumed that the failure zone consisted of a series of conical bodies and derived the limit support pressure. Several researchers [29–31] derived the limit support pressure for tunnel face failure by assuming that the failure zone is a different shape. Zhang developed a horizontal slice method based on the limit equilibrium analysis theory to analyze tunnel face stability in soft surrounding rocks [32].

When the shield tunnel excavates below the river bed, high water pressure is encountered in the site [33]. Water pressure plays an important role in stability of tunnel excavation, and it may cause the stability of the tunnel face to become difficult to control [34]. If the applied face pressure is less than the limit support pressure, the tunnel face will collapse. The shield tunnel is inevitably constructed below the river, so the research on the stability of the tunnel face in multi-layer soils below river is significant. In this paper, the limit equilibrium method which takes water pressure into account is proposed for the calculation of the limit support pressure, and the solutions are compared with the numerical results for verification. The numerical analysis is carried out using FLAC3D [35].

2 Methods

The limit equilibrium method first proposed by Horn [36] is used to analyze the tunnel face stability. The tunnel face is supposed to be three-dimensional failure mode. The simple mechanism model of face stability is illustrated in Figure 1. The tunnel face mechanism approximates the circular face by means of a square and consists of a wedge and a right-angled prism. The wedge is assumed as a rigid body. B, L, H are the width, length, and height of the prism, respectively. The soil is assumed to obey the Mohr-Coulomb failure condition with cohesion represented as c and friction angle represented as φ .

The side length B of the square is approximated by setting its area equal to that of the circular tunnel face [36, 37].

$$B^2 = \frac{1}{4}\pi D^2 \quad (1)$$

where D is the tunnel diameter.

Figure 1: Wedge stability model

The inclination of wedge α is

$$\alpha = 45^\circ + \frac{\varphi}{2} \quad (2)$$

The other side length of prism is

$$L = \frac{B}{\tan \alpha} \quad (3)$$

Consider an element having a dimension dz within a prism of height H as shown in Figure 1. The upper and lower vertical force of element are, respectively,

$$P_1 = A\sigma_v \quad (4)$$

$$P_2 = A(\sigma_v + d\sigma_v) \quad (5)$$

where σ_v is the effective earth pressure from the overlying prism, A is the cross-sectional area of prism.

The vertical friction applied to the lateral side of the element is

$$f = k_0 U \sigma_v \tan \varphi dz + Ucdz \quad (6)$$

where U is the girth of prism, k_0 is coefficient of earth pressure at rest, and $k_0 = 1 - \sin \varphi$.

The vertical equilibrium equation of dz reads as follows

$$k_0 U \sigma_v \tan \varphi dz + Ucdz + A(\sigma_v + d\sigma_v) - A\sigma_v = \gamma Adz \quad (7)$$

where γ is the gravity of the soil.

From the equilibrium equation of the forces acting on the wedge Eq. (7), with boundary conditions $z = 0, \sigma_v = p_0$, p_0 is the surcharge on the surface. The effective earth pressure acting upon the wedge can be calculated using the following equation.

$$\sigma_v = \frac{A\gamma - Uc}{Uk_0 \tan \varphi} \left(1 - e^{\frac{-zUk_0 \tan \varphi}{A}} \right) + p_0 e^{\frac{-zUk_0 \tan \varphi}{A}} \quad (8)$$

In practical engineering, the overburden layer of a tunnel is often the composite layers. In this paper, the effect of a multi-layered overburden on the overlying earth pressure also can be taken into account. A model of earth pressure calculation is shown in Figure 2.

Figure 2: Model of earth pressure calculation

It is assumed that there are n layers in the composite soils. σ_{v1} and σ_{vi} are the earth pressures acting upon the wedge of the first and i -th layer, respectively. They can be respectively expressed as

$$\sigma_{v1} = \frac{A\gamma_1 - Uc_1}{Uk_{01} \tan \varphi_1} \left(1 - e^{\frac{-z_1 Uk_{01} \tan \varphi_1}{A}} \right) + p_0 e^{\frac{-z_1 Uk_{01} \tan \varphi_1}{A}} \quad (9)$$

$$\sigma_{vi} = \frac{A\gamma_i - Uc_i}{Uk_{0i} \tan \varphi_i} \left(1 - e^{\frac{-z_i Uk_{0i} \tan \varphi_i}{A}} \right) + \sigma_{vi-1} e^{\frac{-z_i Uk_{0i} \tan \varphi_i}{A}} \quad (10)$$

The earth pressure of the No. n layer can be obtained using a layer to layer method.

$$\sigma_{vn} = \sum_{i=2}^n \frac{A\gamma_i - Uc_i}{Uk_{0i} \tan \varphi_i} \left(1 - e^{\frac{-z_i Uk_{0i} \tan \varphi_i}{A}} \right) + \frac{A\gamma_n - Uc_n}{Uk_{0n} \tan \varphi_n} \left(1 - e^{\frac{-z_n Uk_{0n} \tan \varphi_n}{A}} \right) + p_0 e^{\frac{-z_n Uk_{0n} \tan \varphi_n}{A}}$$

$$+ p_0 \prod_{i=1}^n e^{\frac{-z_i Uk_{0i} \tan \varphi_i}{A}} \quad (11)$$

Wedge analysis based on the limit equilibrium theory is adopted to obtain the support pressure for the tunnel face stability. Assuming the failure criterion holds along the failure face, a static equilibrium equation can be set up. When a shield tunnel is located below the river, the stability analyses of the shield tunnels face needs to consider the influence of water pressure. Water pressure is usually considered an external force [33], and the forces acting upon the wedge at the face are illustrated in Figure 3. A hydrostatic distribution of water pressures along the slip surface is assumed. There is an effective earth stress σ_v at the wedge-prism-interface, the self-weight of wedge G , the support force P at the tunnel face, the normal force N on the inclined sliding surface, the shear forces T on the inclined as well as on the sliding surface, the symmetric normal force N' and the shear force T' on the two lateral surfaces of the wedge.

Figure 3: Forces acting upon the wedge

The self-weight of the wedge G

$$G = \frac{\gamma B^3}{2 \tan \alpha} \quad (12)$$

where B is the height of the wedge.

The vertical mean stress σ'_z on the inclined sliding surface

$$\sigma'_z = \frac{2\sigma_{vn}}{3} + \frac{B\gamma}{3} \quad (13)$$

The shear forces T on the inclined sliding surface

$$T = \frac{cB^2}{\sin \alpha} + N \tan \varphi \quad (14)$$

The shear force T' on the two sides of the wedge

$$T' = \frac{B^2}{2 \tan \alpha} (c + k_0 \sigma'_z \tan \varphi) \quad (15)$$

The vertical earth pressure P_v acting on the top of wedge is presented as

$$P_v = A\sigma_v = BL\sigma_v \quad (16)$$

When not taking the infiltration in the excavation face into account, the overburden strata are assumed to be permeable with high permeability, such as sand. Thus, a complete hydraulic connection exists between the river water and the groundwater [38]. The water table can be assumed to be at the surface of the river water for the computation of water pressure acting on the tunnel. In this sense, the water pressure generated by river water can be expressed as

$$P_w = B^2 \left(\sum \gamma_w z_i + \gamma_w h \right) \quad (17)$$

where γ_w is the unit weight of water, z_i is the thickness of the i -th stratum, which is located above the center point of the tunnel face.

According to the limit equilibrium principles, force equilibrium is realized horizontally and vertically. By considering the water pressure and equating force in the vertical and horizontal directions, the equations of equilibrium in the horizontal and vertical dimensions are obtained as

$$P - P_w + T \cos \alpha + 2T' \cos \alpha = N \sin \alpha \quad (18)$$

$$P_v + G = T \sin \alpha + 2T' \sin \alpha + N \cos \alpha \quad (19)$$

By solving the equilibrium Equations (18) and (19), the limit support force on the tunnel face is calculated as

$$P = \varepsilon (\sigma_v BL + G) - \left(\frac{cB^2}{\sin \alpha} + 2T' \right) (\varepsilon \sin \alpha + \cos \alpha) + P_w \quad (20)$$

where $\varepsilon = \frac{\sin \alpha - \tan \varphi \cos \alpha}{\cos \alpha + \tan \varphi \sin \alpha}$.

The face stability analysis relevant to a circular rigid tunnel could be idealized. The support pressure is simplified to be uniform [17]. The minimal support pressure termed as the limit support pressure σ_T can be calculated with consideration of water pressure in river and is given as

$$\sigma_T = \frac{P}{B^2} \quad (21)$$

3 Results

3.1 Results using the numerical method

The validity of the proposed method is evaluated in this section through a comparison with the numerical model

using FLAC3D. The FLAC3D is adopted to predict the limit state and the development of deformation. An explicit Lagrangian calculation scheme is used in the FLAC3D. This is a forward scheme for a nonlinear problem which does not require iteration [18]. This code can deal with the large deformation problem very well and can avoid the problem of numerical instabilities during analysis [39–41]. Therefore, the FLAC3D is adopted to simulate the limit support pressure. The FLAC3D model is shown in Figure 4. The three-dimensional numerical model is 40 m long, 21 m wide, and 30 m high. A circular tunnel under the three layers soils is considered in evaluating the limit support pressure on the tunnel face. The tunnel support parameters are as follows: the segment is C50 concrete, the segment thickness is 30 cm, the Young modulus of the concrete segment is 34.5 GPa, Poisson's ratio of the concrete segment is 0.2, and the density is 2.45 g/cm³. The parameters of the tunnel and soils are selected from the case of subway Line No.1 across the Ganjiang River. A circular tunnel with a diameter D of 6 m, a tunnel depth H of 12 m, and a water depth h of 10 m in river is assumed. The analysis is conducted using a linear Mohr-Coulomb yield criterion. Analyses are performed for the four cases, and the mechanical parameters of friction angle φ and cohesion c of the soil stratum are summarized in Table 1. The four cases are used to analyze the situations with the excavation layer as cohesive soil or cohesive soil, and the cases of overlying soil as cohesive soil or cohesive soil. The soil thicknesses of 1-1, 1-2, 1-3 are 6 m, 6 m, 18 m, respectively. For stratum 1-1, 1-2, 1-3, an elastic modulus E of 40 MPa, Poisson's ratio ν of 0.33, and gravity γ of 20 kN/m³ of are selected.

Table 1: Mechanical parameters of soil

soil	Case A		Case B		Case C		Case D	
	c /kPa	φ /°	c /kPa	φ /°	c /kPa	φ /°	c /kPa	φ /°
1-1	0	25	5	35	0	25	5	35
1-2	5	35	0	25	5	35	0	25
1-3	0	25	0	25	5	35	5	35

The analyses of tunnel face stability and supporting pressure are performed with FLAC3D. The applied support pressure can be normalized by the initial earth pressure [42]. The support pressure ratio is equal to the applied support pressure divided by the initial horizontal earth stress at the center of the tunnel face. The initial support pressure on the face is set as the initial horizontal earth pressure, and is gradually reduced until the tunnel face collapses. The displacements in the surrounding ground have been highlighted. Displacement contours

Figure 4: Numerical model of calculation

around tunnel face and the displacement fields for case A, case B, case C, and case D are plotted in Figure 5. The monitoring displacements in Figure 5 are in the horizontal direction (excavation direction), and the unit of the displacement is in meters. The displacements of the tunnel face are calculated according to the different cases with a variation in soil layers. For case A and case B, the tunnel faces are in cohesionless soils. Case C and case D are in cohesive-frictional soils.

The failure zones in case A and case B are obviously larger than that in case C and case D, and mainly concentrate in the upper region of tunnel face. By comparing Figure 5(a) and Figure 5(b), it is evident that the case A has the larger zone of sliding, but is blocked by the cohesive soil layer 1-2 and is mainly concentrate in the soil layer 1-3. The sliding zone in case B extends gradually to the river bed surface, the displacement decreases drastically. For case C and case D, when the tunnel excavates in the cohesive soil layer, the sliding zone shrinks and concentrates in front of the tunnel face. It is obvious that cohesion has an important influence on the failure form.

Tunnel face stability and the behavior of surrounding soils are analyzed with varying support pressure ratios at the tunnel face. Evaluating the relationship between the displacement of a control point (the face center) and the applied support pressure ratio (normalized by initial stress) is a common way to analyze the tunnel face stability [43]. Figure 6(a) and Figure 6(b) show horizontal and vertical displacement of the face center with

a variation of support pressure ratios for the case A, case B, case C, and case D, respectively. Large displacement is expected to occur when there is small support pressure ratio. The displacement increases significantly when the support pressure ratio decreases to a certain value. This value is the limit support pressure ratio. From the results of the FLAC3D analysis for the four cases previously mentioned, the calculation results of the limit support pressure on the tunnel face are shown in Table 2.

Table 2: Results with the FLAC3D

Case	Initial earth pressure /kPa	Limit support pressure ratio	Limit support pressure /kPa
A	275	0.755	207.63
B	275	0.78	214.50
C	275	0.715	196.63
D	275	0.720	198.00

3.2 Results using the limit equilibrium method

Comparative calculations are carried out. The same cases are analyzed using the proposed limit equilibrium method and numerical method using FLAC3D. The analyses of the tunnel face supporting pressure with the proposed limit equilibrium method are preformed. Table 3 compares the limit support pressure obtained using the limit equilibrium method with that obtained from the numerical method.

Table 3: Results of limit support pressure

	A	B	C	D
numerical method	207.63	214.50	196.63	198.00
limit equilibrium method	219.01	229.27	197.72	204.39
error	5.19%	6.44%	0.55%	3.12%

4 Discussion

The results of case A and case B, which were excavated in cohesionless soil, are different. The error of them is nearly 10%, although the tunnel excavates at the same soil layer. The reason is that the upper soil layer 1-2 in case A is comprised of cohesive soils, which are conducive to the stability of the tunnel face and prevent the development of a fail-

Figure 5: Displacement contours around the tunnel face

ure zone. Thus, the required limit support pressure to keep the tunnel face stable in case A is small compared to that in case B. When the excavation is in the cohesive soil for case C and case D, their required limit support pressures are similar, although the cover soil layers are different. The limit support pressure in case A and case B are obviously greater than that in case C and case D. The main reason is that the excavation stratum of case A and case B is cohesionless soil.

The limit support pressures calculated by limit equilibrium method are 219.01 kPa, 229.27 kPa, 197.72 kPa, 204.39 kPa for the cases A, B, C, and D, respectively. The results of the limit equilibrium method agree well with numerical calculations with respect to the limit support pressure, as shown in Table 3. The obtained results suggest that

the approach proposed herein could be applied for fast estimations of the pressure needed for face support in multi-layer soils below river, with consideration to water pressure. These values are about 5% higher than the prediction from the numerical method. Mollon et al. [44] got a similar result for a 2D stability analysis of a pressurized tunnel face in sand. The proposed limit equilibrium method is also an effective method to calculate the limit support pressure on the tunnel face for permeable overburden strata that has a high permeability. It cannot be used for impermeable overburden strata that has a low permeability. The proposed method is suitable for cohesionless soil. The proposed method is reliable and effective when it is used to design the tunnel excavation process.

(a) Horizontal displacement

(b) Vertical displacement

Figure 6: Displacement of the central point of face

5 Conclusions

Managing suitable support pressures acting on a tunnel face is very important in stabilizing the tunnel face. In this paper, a limit equilibrium method with considerations for water pressure is proposed to determine the limit support pressure of tunnel faces in multi-layer soils below river. Based on the proposed limit equilibrium methods, the limit support pressures required for tunnel face stability below the river bed are estimated on variable ground. The face stability is also investigated using the FLAC3D. The limit support pressures have been calculated to compare the results of the limit equilibrium method with those from a three-dimension numerical method FLAC3D. There is good agreement between the limit equilibrium results and the numerical results. The limit support pressure using the limit equilibrium method is greater than that obtained by the numerical method. It is safe and efficient when the limit equilibrium method is used to design the

support pressure on tunnel face with permeable overburden strata that has a high permeability. This research can provide a theoretical foundation for tunnel engineering practices in multi-layer soils below the river bed. The next stage of the work is studying the limit support pressure for impermeable overburden strata that has a low permeability.

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