Research Article

Face Stability Analysis of Shield Tunnels in Homogeneous Soil Overlaid by Multilayered Cohesive-Frictional Soils

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In order to better interpret failure features of the failure of soil in front of tunnel face, a new three-dimensional failure mechanism is proposed to analyze the limit support pressure of the tunnel face in multilayered cohesive-frictional soils. The new failure mechanism is composed of two truncated cones that represent the shear failure band and a distributed force acting on the truncated cones that represents the pressure arch effect. By introducing the concept of Terzaghi earth pressure theory, approximation of limit support pressures is calculated using the limit analysis methods. Then the limit support pressures obtained from the new failure mechanism and the existing approaches are compared, which show that the results obtained from the new mechanism in this paper provide relatively satisfactory results.

1. Introduction

Nowadays, tunnel constructions in urban shallow soft ground are more frequently being carried out using closed shields, which are basically divided into compressed-air shields, slurry shields, and earth pressure balanced shields. By supporting the excavation face and preventing the seepage flow to the face, closed shields control the surface settlement and limit the risk of tunnel face failure through the continuous support of the face during excavation. However, lack of sufficient face support leads to the instability of tunnel face. In extreme cases, the collapse propagates up to the ground surface creating a surface depression. Researchers have developed experimental, numerical, and analytical approaches to investigate the tunnel face stability and to determine the limit support pressure of the tunnel face in cohesive-frictional soils.

The stability of tunnel face in cohesive-frictional soils has been analyzed by using experimental tests (laboratory centrifuge tests [1–3] or 1g model tests [4–6]) and the numerical simulations (the Finite Element Method [7–9], the

Finite Difference Method [6, 10, 11], and the Discrete Element Method [12–14]). Both the limit equilibrium methods [15–23] and the limit analysis methods [9–11, 24–31] were used by other researchers to investigate the stability of tunnel face in cohesive-frictional soils. Actually, from the analytical approaches mentioned above, it can be concluded that either limit equilibrium methods or limit analysis methods have advantages and disadvantages. Sloan [32] presented detailed reviews and comparisons of the analytical approaches for assessing geotechnical stability.

From the results obtained from experimental tests and numerical simulations, it is shown that the failure of soil in front of tunnel face shows two main features, that is, shear failure band in the lower part and pressure arch effect in the upper part. Referring to the remarks claimed by Sloan [32] and considering the specific problems of tunnel face stability analysis, the main characteristics of the limit equilibrium and limit analysis to assess the stability of tunnel face in cohesive-frictional soils are summarized as follows. As a tradition stability analysis method, the limit equilibrium methods are widely used in the practical engineering to

FIGURE 1: The improved failure mechanism.

analyze the stability of tunnel face. Due to its simplicity, a multitude of prediction models have been developed to solve the complex situations of tunnel face, for example, seepage flow conditions and bolt reinforcement. To reflect the feature of pressure arch effect, most of failure mechanisms based on limit equilibrium methods adopted Terzaghi earth pressure theory. However, limit equilibrium methods neglect the stress-strain relationship of the soil, which is a condition that must be satisfied for a complete solution stated by Chen [33]. Compared to the limit equilibrium methods, the limit analysis methods have a strictly theoretic basis.

Based on the limit analysis methods, Zhang et al. [34] proposed a three-dimensional failure mechanism, which is composed of four truncated cones and a distributed force acting on those truncated cones. Their failure mechanism combines the generality of earth pressure theory with the rigour of limit analysis. And the results show that their improved treatments are effective. However, their works are limited to the case of homogeneous soils. Practical experience shows that the stability of the face is often a problem in heterogeneous soils (Ibrahim et al. [9]; Senent and Jimenez [11]). The shear strength parameters of soils in different strata are different from each other. In this paper, for multilayered cohesive-frictional soils, a new three-dimensional failure mechanism is proposed to determine the limit support pressure of the tunnel face using limit analysis methods. The new failure mechanism is composed of two truncated cones on which a distributed force acts. The distributed force could be calculated by using Terzaghi earth theory. Then, the limit support pressures obtained from the new failure mechanism and the existing approaches are compared.

2. Limit Analysis of the Tunnel Face Stability

2.1. The New Failure Mechanism. The tunnel face stability to be analyzed in this paper is idealized as shown in Figure 1. By considering that the tunnel is a rigid circular cylinder of diameter D driven under a depth of cover C , a surcharge σ_s is applied on the ground surface, and σ_T is the uniform support pressure on the tunnel face.

Referring to the concept proposed by Zhang et al. [34], a new prediction model is established to analyze face stability of shallow circular tunnels in multilayered cohesive-frictional soils. The new failure mechanism is composed of two truncated cones on which a distributed force acts. The two truncated cones were firstly proposed by Leca and Dormieux [26] to investigate the tunnel face stability in homogenous soil. Since this paper considers a multilayered cohesive-frictional soil (i.e., the crossed layer is assumed to be homogeneous whereas the cover soil is layered), two truncated cones proposed by Leca and Dormieux [26] are modified and only the part in crossed layer of the two truncated cones is preserved. The new failure mechanism considers the effect of the cover layers as a distributed force according to the Terzaghi earth theory. The same treatments were adopted in the works of Broere [18]; Kirsch and Kolymbas [19]; Anagnostou [20]; and Anagnostou and Perazzelli [21], which are based on the limit equilibrium method.

2.1.1. Geometric Properties. The intersections of two truncated cones, tunnel face, and horizontal plane crossing the top of the tunnel face are two ellipses and a circle and are called Σ_1 , $\Sigma_{1,2}$, and Σ_2 . The intersection of the first truncated cone

(adjacent to the tunnel face) with the circular tunnel face is an ellipse Σ_1 , with semiaxis lengths of a_1 and b_1 and with area of A_1 that are calculated as follows (cf. Figure 1):

$$\begin{aligned} a_1 &= \frac{D}{2}, \\ b_1 &= \frac{D}{2} \frac{\sqrt{\cos(\alpha - \varphi) \cos(\alpha + \varphi)}}{\cos \varphi}, \\ A_1 &= \frac{\pi D^2}{4} \frac{\sqrt{\cos(\alpha - \varphi) \cos(\alpha + \varphi)}}{\cos \varphi}, \end{aligned} \quad (1)$$

where φ defines the opening angles of the two truncated rigid cones that are equal to 2φ and α is the angle between the axis of the first truncated rigid cone which is adjacent and the horizontal.

The area of the contact elliptical surface $\Sigma_{1,2}$ between two truncated cones is ellipse with semiaxis lengths of a_{12} and b_{12} and with area of A_{12} is described as follows:

$$\begin{aligned} a_{12} &= \frac{D \cos(\alpha + \varphi)}{2 \sin(\beta + \varphi)}, \\ b_{12} &= \frac{D \cos(\alpha + \varphi)}{2 \cos \varphi} \left[\frac{\sin(\beta - \varphi)}{\sin(\beta + \varphi)} \right]^{1/2}, \\ A_{12} &= \frac{\pi D^2 \cos^2(\alpha + \varphi)}{4 \cos \varphi} \frac{\sqrt{\sin(\beta - \varphi)}}{[\sin(\beta + \varphi)]^{3/2}}. \end{aligned} \quad (2)$$

The intersection of the second truncated cone with the horizontal plane across the top of the tunnel face is a circle Σ_2 with radius of r_2 and with area of A_2 as follows:

$$\begin{aligned} r_2 &= \frac{D}{2} \left[\frac{\sin \beta \cos \alpha}{\sin \varphi \sin(\beta + \varphi)} - 1 \right] \tan \varphi, \\ A_2 &= \frac{\pi D^2}{4} \left[\frac{\sin \beta \cos \alpha}{\sin \varphi \sin(\beta + \varphi)} - 1 \right]^2 \tan^2 \varphi. \end{aligned} \quad (3)$$

In addition, the lateral surfaces and volumes of the two truncated cones are as follows:

$$\begin{aligned} S_{b1} &= \frac{\pi D^2}{4} \left[\frac{\cos \alpha}{\sin \varphi} R_A - \cos(\alpha + \varphi) R_C R_D \right], \\ S_{b2} &= \frac{\pi D^2}{4} \left[\cos(\alpha + \varphi) R_C R_D - \frac{\cos^2 \varphi}{\sin \varphi \cos^2(\alpha + \varphi)} R_C^4 \right], \\ V_{b1} &= \frac{\pi D^3}{12} \left(R_A R_B - \frac{\cos \varphi}{2 \sin \varphi} R_C^3 \right), \\ V_{b2} &= \frac{\pi D^3}{12} \left[\frac{\cos \varphi}{2 \sin \varphi} R_C^3 - \frac{\cos^2 \varphi}{2 \cos^2(\alpha + \varphi)} R_C^4 \right], \end{aligned} \quad (4)$$

FIGURE 2: Distributed force calculated by Terzaghi earth pressure theory.

where

$$\begin{aligned} R_A &= \frac{\sqrt{\cos(\alpha - \varphi) \cos(\alpha + \varphi)}}{\cos \varphi}, \\ R_B &= \frac{\cos(\alpha - \varphi) \cos(\alpha + \varphi)}{\sin(2\varphi)}, \\ R_C &= \frac{\cos(\alpha + \varphi)}{\cos \varphi} \left[\frac{\sin(\beta - \varphi)}{\sin(\beta + \varphi)} \right]^{1/2}, \\ R_D &= \frac{\sin \beta}{\sin \varphi \sin(\beta + \varphi)}. \end{aligned} \quad (5)$$

2.1.2. The Distributed Force Acting on the Two Truncated Cones. For cohesive-frictional soils, the results obtained from experimental tests and numerical simulations show that the failure of soil in front of tunnel face demonstrates two main features, that is, shear failure band in the lower part and pressure arch effect in the upper part. The existing researches [18–21] considered the effect of the cover layers as a distributed force and achieved satisfactory results, which is calculated using the Terzaghi earth pressure theory. In this paper, the distributed force acting on the two truncated cones is also calculated by Terzaghi earth pressure theory as shown in Figure 2.

Assuming that the cover layer above the tunnel is homogeneous and the soil in cover layer satisfies the Mohr-Coulomb failure criteria, the vertical equilibrium of a microthin layer dz at z depth and the boundary condition at the ground surface are as follows:

$$\begin{aligned} \pi r_2^2 \sigma_v + \pi r_2^2 \gamma dz &= \pi r_2^2 (\sigma_v + d\sigma_v) + 2\pi r_2 \tau dz, \\ \tau &= c + K_0 \tan \varphi, \\ z = 0 \mid \sigma_v &= \sigma_s, \end{aligned} \quad (6)$$

where the lateral pressure coefficient is defined as $K_0 = (\cos^2 \theta_0 + K_a \sin^2 \theta_0) / ((1/3)(1 - K_a) \sin^2 \theta_0 + K_a)$, $\theta_0 = \pi/4 + \varphi/2$, $K_a = \tan^2(\pi/4 - \varphi/2)$ [35].

Equation (6) is a differential equation for the vertical stress $\sigma_v(z)$. To solve the differential equation and consider

the boundary condition, the result of vertical stress at any depth is determined as follows:

$$\begin{aligned}\sigma_v = & \frac{\gamma r}{2K_0 \tan \varphi} \left(1 - e^{-(2K_0 \tan \varphi/r)C}\right) \\ & - \frac{c}{K_0 \tan \varphi} \left(1 - e^{-(2K_0 \tan \varphi/r)C}\right) \\ & + \sigma_s e^{-(2K_0 \tan \varphi/r)C}.\end{aligned}\quad (7)$$

Since the ground above the tunnel is heterogeneous and consists of n horizontal layers (cf. Figure 2), the vertical stress should be applied recursively for every layer from the top to down by treating the pressures of every overlying layer as a surface loading as follows:

$$\begin{aligned}P_i = & \frac{\gamma_i r}{2K_{0i} \tan \varphi_i} \left(1 - e^{-(2K_{0i} \tan \varphi_i/r)C_i}\right) \\ & - \frac{c_i}{K_{0i} \tan \varphi_i} \left(1 - e^{-(2K_{0i} \tan \varphi_i/r)C_i}\right) \\ & + P_{i-1} e^{-(2K_{0i} \tan \varphi_i/r)C_i} \quad (i = 1, \dots, n), \\ P_0 = & \sigma_s, \\ P_n = & \sigma_v.\end{aligned}\quad (8)$$

Then the final result of vertical stress distribution acting on the two truncated cones is obtained as follows:

$$\begin{aligned}\sigma_v = & \frac{\gamma_n r}{2K_{0n} \tan \varphi_n} \left(1 - e^{-(2K_{0n} \tan \varphi_n/r)C_n}\right) \\ & + \sum_{k=1}^{n-1} \left[\frac{\gamma_k r}{2K_{0k} \tan \varphi_k} \left(1 - e^{-(2K_{0k} \tan \varphi_k/r)C_k}\right) \right. \\ & \cdot \left. \prod_{m=k+1}^n e^{-(2K_{0m} \tan \varphi_m/r)C_m} \right] - \frac{c_n}{K_{0n} \tan \varphi_n} \left(1 - e^{-(2K_{0n} \tan \varphi_n/r)C_n}\right) \\ & - \sum_{k=1}^{n-1} \left[\frac{c_k}{K_{0k} \tan \varphi_k} \left(1 - e^{-(2K_{0k} \tan \varphi_k/r)C_k}\right) \right. \\ & \cdot \left. \prod_{m=k+1}^n e^{-(2K_{0m} \tan \varphi_m/r)C_m} \right] + \sigma_s \prod_{k=1}^n e^{-(2K_{0k} \tan \varphi_k/r)C_k},\end{aligned}\quad (9)$$

where C_k is the thickness of layer k and c_k , φ_k , K_{0k} , and γ_k denote its unit weight and shear strength parameters.

2.1.3. Velocity Field. The two truncated rigid cones are translated with velocities with different directions, which are collinear with the cones' axes and are at an angle φ to the discontinuity surface. The velocity of first cone and the

relative velocity between the two cones are described by the following equations:

$$\begin{aligned}V_1 = & \frac{\sin(\beta + \varphi)}{\sin(\beta - \varphi)} V_2, \\ V_{12} = & \frac{\cos \alpha}{\sin(\beta - \varphi)} V_2.\end{aligned}\quad (10)$$

2.2. Limit Support Pressure. To satisfy the stability conditions of the tunnel face according to the upper bound theorem, the following relation is considered:

$$P_e \leq P_v, \quad (11)$$

where P_e represents the power of the external loads and P_v denotes the dissipation power. The power of the external loads, P_e , is the sum of three components: P_T , the power of the support pressure σ_T , P_s , the power of the vertical stress σ_v , and P_γ , the power of the soil unit weight γ :

$$\begin{aligned}P_e = & P_T + P_{\sigma_v} + P_\gamma, \\ P_v = & P_{1v} + P_{2v} + P_{3v},\end{aligned}\quad (12)$$

where

$$\begin{aligned}P_T = & \left[\frac{\pi D^2 \cos^2(\alpha + \varphi)}{4 \cos^2 \varphi} \right] \left[-\frac{\cos \alpha}{R_C^2} R_A \sigma_T \right] V_2, \\ P_{\sigma_v} = & \left[\frac{\pi D^2 \cos^2(\alpha + \varphi)}{4 \cos^2 \varphi} \right] \left[\frac{R_E^2}{\cos^2(\alpha + \varphi)} \sigma_v \right] V_2, \\ P_\gamma = & \left[\frac{\pi D^2 \cos^2(\alpha + \varphi)}{4 \cos^2 \varphi} \right] \left[\left(\sin \alpha \frac{R_A R_B}{R_C^2} \right. \right. \\ & + \frac{\cos \alpha \cos \varphi \cos(\beta + \varphi)}{2 \sin \varphi \sin(\beta + \varphi)} R_C \\ & \left. \left. - \frac{R_E^3}{2 \sin \varphi \cos^2(\alpha + \varphi)} \right) \frac{\gamma D}{3} \right] V_2, \\ P_{1v} = & \left[\frac{\pi D^2 \cos^2(\alpha + \varphi)}{4 \cos^2 \varphi} \right] \left[\frac{\cos \alpha}{\sin \varphi} \frac{R_A}{R_C^2} - \cos(\alpha + \varphi) \right. \\ & \cdot \left. \frac{R_D}{R_C} \right] [c \cos \varphi] V_2, \\ P_{2v} = & \left[\frac{\pi D^2 \cos^2(\alpha + \varphi)}{4 \cos^2 \varphi} \right] \left[\frac{\cos^2 \varphi}{\cos^2(\alpha + \varphi)} R_C R_D \right. \\ & \left. - \frac{R_E^2}{\sin \varphi \cos^2(\alpha + \varphi)} \right] [c \cos \varphi] V_2, \\ P_{3v} = & \left[\frac{\pi D^2 \cos^2(\alpha + \varphi)}{4 \cos^2 \varphi} \right] \left[\frac{2 \cos \beta \sin^2 \varphi \cos(\alpha + \varphi)}{\sin \beta} \right. \\ & \left. \cdot \frac{R_D^2}{R_C} \right] [c \cos \varphi] V_2.\end{aligned}\quad (13)$$

By equating the total rate of external work to the total rate of internal energy dissipation, as shown in (11), the pressure σ_T at the tunnel face is obtained by (14) with (15) as follows:

$$\sigma_T = N_S \cdot \sigma_S + N_\gamma \cdot \gamma D + N_c \cdot c + \sum_{k=1}^n N_k \cdot \gamma_k D + N_{ck} \quad (14)$$

$\cdot c_k$,

$$N_S = \left[\frac{1}{\cos \alpha \cos^2 \varphi \sin(\beta + \varphi)} \frac{R_E^2}{R_A} \right]$$

$$\cdot \left[\prod_{k=1}^n e^{-(2K_{0k} \tan \varphi_k / r) C_k} \right],$$

$$N_\gamma = \left\{ \frac{1}{3} \left[\tan \alpha R_B + \frac{\cos \varphi \cos(\beta + \varphi)}{2 \sin \varphi \sin(\beta + \varphi)} \frac{R_C^3}{R_A} \right. \right. \\ \left. \left. - \frac{1}{2 \sin \varphi \cos \alpha \cos^2 \varphi \sin(\beta + \varphi)} \frac{R_E^3}{R_A} \right] \right. \\ \left. + \left[\frac{1}{\cos \alpha \cos^2 \varphi \sin(\beta + \varphi)} \frac{R_E^2}{R_A} \right] \left[\frac{r}{2K_0 \tan \varphi D} (1 - e^{-(2K_0 \tan \varphi / r) C}) \right] \right\},$$

$$N_c = \left\{ \frac{1}{\tan \varphi} \left[\frac{1}{\cos \alpha \cos^2 \varphi \sin(\beta + \varphi)} \frac{R_E^2}{R_A} - 1 \right] \right. \\ \left. + \left[\frac{1}{\cos \alpha \cos^2 \varphi \sin(\beta + \varphi)} \frac{R_E^2}{R_A} \right] \left[-\frac{1}{K_0 \tan \varphi} (1 - e^{-(2K_0 \tan \varphi / r) C}) \right] \right\}, \quad (15)$$

$$\sum_{k=1}^n N_{\gamma k} \cdot \gamma_k D + N_{ck} \cdot c_k = \left[\frac{1}{\cos \alpha \cos^2 \varphi \sin(\beta + \varphi)} \right. \\ \cdot \frac{R_E^2}{R_A} \left[\frac{\gamma_n r}{2K_{0n} \tan \varphi_n} (1 - e^{-(2K_{0n} \tan \varphi_n / r) C_n}) \right. \\ \left. + \sum_{k=1}^{n-1} \left[\frac{\gamma_k r}{2K_{0k} \tan \varphi_k} (1 - e^{-(2K_{0k} \tan \varphi_k / r) C_k}) \right. \right. \\ \left. \cdot \prod_{m=k+1}^n e^{-(2K_{0m} \tan \varphi_m / r) C_m} \right] - \frac{c_n}{K_{0n} \tan \varphi_n} (1 - e^{-(2K_{0n} \tan \varphi_n / r) C_n}) \\ \left. - \sum_{k=1}^{n-1} \left[\frac{c_k}{K_{0k} \tan \varphi_k} (1 - e^{-(2K_{0k} \tan \varphi_k / r) C_k}) \right. \right. \\ \left. \cdot \prod_{m=k+1}^n e^{-(2K_{0m} \tan \varphi_m / r) C_m} \right] \left. \right].$$

FIGURE 3: Influence of $\Delta\varphi_0$ on the limit support pressures ($\Delta c_0 = 0$ kPa).

In (14), σ_T , N_γ , N_c , and N_S depend on the mechanical and geometrical characteristics c , φ , and C/D and on the angular parameters of the failure mechanism α . These parameters were obtained by maximizing σ_T in (14) with respect to the angles α . An upper bound solution can be found by numerically optimizing (14) with respect to the angle α .

3. Comparisons

To validate the results obtained from the limit analysis developed in this paper, comparisons between the results of this work and existing approaches (Broere [18]; Tang et al. [30]; Senent and Jimenez [11]) were performed. Overall, the model proposed by Broere [18] is based on limit equilibrium methods while the models proposed by Tang et al. [30], Senent and Jimenez [11], and this paper are based on limit analysis methods, respectively.

Three sets of analyses were carried out by referring to the researches recently reported by Tang et al. [30] and Senent and Jimenez [11]. And the detailed soil parameters of layers are shown in Table 1. The unit weight γ is equal to 18 kN/m³ for the layers in all cases ($C/D = 1.5$).

3.1. Influence of the Variables of the Crossed Soil on Limit Support Pressures. The first set of analyses described the cases of a single cover layer with constant strength parameters (c_1 and φ_1) and a single crossed layer with different strength parameters (c_0 and φ_0 and increments Δc_0 and $\Delta\varphi_0$). Figures 3 and 4 show the curves of the limit support pressures with the variation of Δc_0 and $\Delta\varphi_0$, respectively.

In Figure 3, the limit support pressures declined nonlinearly with the increase of $\Delta\varphi_0$ from -5° to 15° . The decrease of the limit support pressures with $\Delta\varphi_0$ from -5° and 0° was steeper than those with $\Delta\varphi_0$ in the range between 0° and 15° , which shows that $\Delta\varphi_0$ has a higher influence on the limit support pressures if the strength parameters of the crossed

TABLE 1: Soil parameters of layers.

Sets' number	Layers' name	Layers' number	c (kPa)	φ (°)	C (m)
1	Single crossed layer	Variable	$c_0 = 2.5$ $\Delta c_0 = \pm 2$	$\varphi_0 = 20^\circ$ $\Delta \varphi_0 = -5^\circ \sim 15^\circ$	$C_0 = D = 6$
	Single cover layer	Constant	$c_1 = 2.5$	$\varphi_1 = 20^\circ$	$C_1 = 9$
2	Single crossed layer	Constant	$c_0 = 2.5$	$\varphi_0 = 20^\circ$	$C_0 = D = 6$
	Single cover layer	Variable	$c_1 = 2.5$ $\Delta c_1 = \pm 2$	$\varphi_1 = 20^\circ$ $\Delta \varphi_1 = -5^\circ \sim 15^\circ$	$C_1 = 9$
3	Single crossed layer	Constant	$c_0 = 2.5$	$\varphi_0 = 20^\circ$	$C_0 = D = 6$
	Two cover layers	Variable	$c_1 = 2.5$	$\varphi_1 = 15^\circ$	$C_1 = 0 \sim 9$
			$c_2 = 2.5$	$\varphi_2 = 10^\circ$	$C_2 = 9 - C_1$

FIGURE 4: Influence of Δc_0 on the limit support pressures ($\Delta \varphi_0 = 0^\circ$).

soil were weaker than strength parameters of the cover soil. Moreover, the results from this paper and Senent and Jimenez [11] were between the results from Broere [18] (the highest solutions) and from Tang et al. [30] (the lowest solutions). The approach proposed by Broere [18] overestimated the limit support pressure compared to the results from other approaches.

Figure 4 shows that the limit support pressures decreased linearly with the increase of Δc_0 . Similarly, the results from this paper and Senent and Jimenez [11] were almost equal and located between the results from Broere [18] (the highest solutions) and from Tang et al. [30] (the lowest solutions). It is noteworthy that rotational face collapse mechanism proposed by Senent and Jimenez [11] considers the whole face of the tunnel by employing a spatial discretization technique, which provides the results that outperform previous upper bound solutions. As mentioned above, the approximations of limit support pressures obtained from the new failure mechanism are not strictly an upper bound solution and the results from our paper are slightly less than the results from Senent and Jimenez [11] in most cases. The comparisons show that the results in this paper provide relatively satisfactory

FIGURE 5: Influence of $\Delta \varphi_1$ on the limit support pressures ($\Delta c_1 = 0$ kPa).

results. To compare Figures 3 and 4, it has also been found that the friction angle affected the support pressure effectively compared to the cohesion.

3.2. Influence of the Variables of the Cover Soil on Limit Support Pressures

3.2.1. Single Cover Layer. The second set of analyses presented a reverse situation, which described the cases of a single cover layer with different strength parameters (c_1 and φ_1 and increments Δc_1 and $\Delta \varphi_1$) and a single crossed layer with constant strength parameters (c_0 and φ_0). Figures 5 and 6 compared change rules of the limit support pressures with the variation of Δc_1 and $\Delta \varphi_1$, respectively.

Figure 5 shows that the limit support pressures decreased nearly linearly with the increase of $\Delta \varphi_1$ from -5° to 15° . And the results from this paper were slightly less than the results from Senent and Jimenez [11] and were much higher than the results from Tang et al. [30].

In Figure 6, the limit support pressures dropped linearly as Δc_1 increased. Similarly, the results from this paper were

FIGURE 6: Influence of Δc_1 on the limit support pressures ($\Delta \varphi_1 = 0^\circ$).

equal to the results from Senent and Jimenez [11] and were much higher than the results from Tang et al. [30].

Since this section was a reverse situation with the last section, it is necessary to compare Figures 3 and 5 (Figures 4 and 6) to analyze their influence degree on the limit support pressures. For the variation of friction angle ($\Delta \varphi_0$ or $\Delta \varphi_1$), the limit support pressures decreased from 40.09 kPa to 5.80 kPa (decrement 34.29 kPa) in Figure 3 while they decreased from 24.86 kPa to 19.44 kPa (decrement 5.42 kPa) in Figure 5. And for the variation of cohesion (Δc_0 or Δc_1), the limit support pressures decreased from 27.11 kPa to 18.11 kPa (decrement 9.00 kPa) in Figure 4 while they decreased from 23.46 kPa to 21.76 kPa (decrement 1.7 kPa) in Figure 6. Those data indicated that strength parameters' variation in crossed layer plays bigger role in limit support pressures than in cover layers with the same variable range of strength parameters.

3.2.2. Two Cover Layers. The third set of analyses described the cases of two cover layers with constant strength parameters (c_1 and φ_1 , c_2 and φ_2 , and relative thickness $\mu = C_1/C_2$) and a single crossed layer with constant strength parameters (c_0 and φ_0). Figure 7 shows the curves of the limit support pressures with the relative thickness μ . It can be seen that the limit support pressures drop dramatically with the increase of μ from 0 to 0.5 while the limit support pressures remain constant when μ is bigger than 0.5. Moreover, the results obtained from this paper are slightly higher than the results obtained from Senent and Jimenez [11] and much higher than those of Tang et al. [30].

4. Conclusions

In order to better interpret the failure features, a new three-dimensional failure mechanism is proposed to analyze the limit support pressure of the tunnel face in multilayered cohesive-frictional soils based on limit analysis methods. The new failure mechanism is composed of two truncated cones

FIGURE 7: Influence of μ on the limit support pressures.

that represent the shear failure band and a distributed force acting on the truncated cones that represents the pressure arch effect. The distributed force could be calculated by using Terzaghi earth pressure theory. Moreover, the comparisons between the results of the present study and those of existing approaches were provided. The main conclusions are presented as follows.

(1) Results show that influence rules obtained from different models are similar. More importantly, our mechanism provides limit pressures that are almost equal to the results from Senent and Jimenez [11] and are higher than those computed by Tang et al. [30], which show that the results obtained from the new mechanism in this paper provide relatively satisfactory results.

(2) For the strength parameters variations of soil in the crossed soil or in the cover soil, the results show that $\Delta \varphi_0$ has a higher influence on the limit support pressures if the strength parameters of the crossed soil were weaker than strength parameters of the cover soil (i.e., $\Delta \varphi_1 \leq 0$).

(3) No matter strength parameters variations of soil in the crossed soil or in the cover soil, it has also been found that the friction angle affected the support pressure effectively compared to the cohesion.

(4) The results indicated that strength parameters' variation in crossed layer plays bigger role in limit support pressures than in cover layers with the same variable range of strength parameters.

Competing Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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