Analysis of Limit Support Pressure on Tunnel Face Below the River Bed

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***Abstract* — Appropriate controlling of support pressure on the tunnel face can ensure tunnel excavation safety. This paper presents a limit equilibrium method to consider the water pressure in river for determine the limit support pressure on the tunnel face. The formula to calculate the limit support pressure on the tunnel face is based on the limit equilibrium theorem and Terzaghi earth pressure theory. The results obtained by proposed method considering water pressure in river are presented and compared with those based on the finite difference method FlAC^{3D}. A comparison shows close agreement between results with limit equilibrium method and numerical method. The proposed method is efficient to compute the limit support pressure on the tunnel face. The limit support pressure with the limit equilibrium method is greater than that with the numerical method. It is safety for the stability of tunnel face when the limit equilibrium method is used to calculate the limit support pressure.**

Keywords - limit support pressure; tunnel face; limit equilibrium method; water pressure; numerical modeling

I. INTRODUCTION

With the recent increase in underground space development as well as for transportation, shield tunneling method is usually used to excavate the tunnel in urban district. It is quite frequent that tunnel should be excavated below the river bed. Maintaining tunnel face stability is a key construction consideration in shield tunnel project, especially the tunnel below the river bed. Water pressure in river will influence the stress redistribution of tunnel face in the process of excavating the tunnel. The water in river will reduce the tunnel face stability, and need greater support pressure to keep face stable. The tunnel face happens easily collapse during the process of construction because of stress release especially blow the river bed. The balance between acting earth pressure and applying support pressure is important to ensure tunnel face safety against tunnel collapse.

The face stability of the tunnel was been investigated by theoretical approach [1-3], experimental tests [4-7] and numerical approach [8]. Lee et al. [9] analyzed the face stability of a tunnel employing the steel pipe-reinforced multi-step grouting by limit equilibrium method and the 3-dimensional finite element analysis with underwater condition. Li et al. [10] studied the face stability of shield tunnel by finite difference method FLAC^{3D}. Soranzo et al. [11] studied the face stability of shallow tunnels by means of centrifuge testing and numerical analysis. The methods to predict the required support pressure predicted by different method still vary significantly. Very little is known of the face stability or limit support pressure of a tunnel driven below the river bed. When the shield tunnel goes across the river, how to calculate the support pressure considering the effect of water in river is an important problem. The water level in river vary, the limit support pressure will change when the tunnel excavates below the river bed. The stability of the shield tunnel affected by variety of the water depth in river and will become difficult to control [12]. Thus, more attention should be paid to the face stability of the shield tunnels blow the river bed. The influence of water level in river on the stability and limit support pressure of the tunnel face needs further study.

This study focuses on limit support pressure on the tunnel face. Based on limit equilibrium method and Terzaghi earth pressure theory, the formula for three-dimension is deduced to estimate the limit support pressure as influenced by water pressure in river. Both the limit equilibrium method and three-dimensional numerical simulation are used to predict the limit support force for the safety of tunnel face. The same cases are simulated by the Finite Difference Method FLAC^{3D} in order to check the proposed mechanism correct. The relationship of deformation and support pressure of tunnel face is obtained by FLAC^{3D}.

II. Overlying Earth Pressure

The Terzaghi's formula [13] has generally been adopted as vertical earth pressure acting the tunnel. The face stability calculations are based on the limit equilibrium analysis of wedge shaped soil body [14]. The wedge stability model of three dimensional mode of tunnel face is shown in Figure 1, in which the collapsing soil in front of the tunnel is schematized as a triangular wedge, the wedge is assumed as a rigid body. B , L , H are the width, length, height of the prism, respectively. This three-dimensional model was first proposed by Horn, a right-angled prism extends from the tunnel crown to the surface. The earth pressure is being addressed by the 3D mechanism described. The soil is assumed as following the Mohr-Coulomb material.

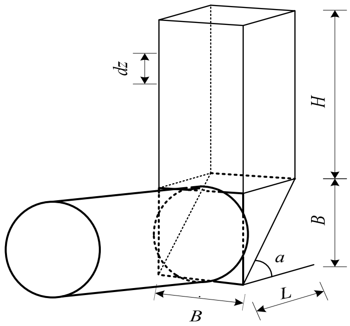

Figure 1. Wedge stability model

The circular cross-section of the tunnel is approximated by a square whose sides are as long as the tunnel diameter and has the same area.

$$B^2 = \frac{1}{4}\pi D^2 \tag{1}$$

where D is the tunnel diameter, B is the side length of the square.

The inclination of wedge α is

$$\alpha = 45^\circ + \frac{\varphi}{2} \tag{2}$$

where φ is friction angel of soil.

The other side length L of prism is

$$L = \frac{B}{\tan\alpha} \tag{3}$$

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

$$P_1 = A\sigma_v\,, \quad P_2 = A\left(\sigma_v + d\sigma_v\right) \tag{4}$$

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

Vertical friction applied to lateral of the element is

$$f = k_0 U\sigma_v \tan\varphi dz + Ucdz \tag{5}$$

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

The vertical equilibrium equation of dz reads as follows

$$k_0 U\sigma_v \tan\varphi dz + Ucdz + A\left(\sigma_v + d\sigma_v\right) - A\sigma_v = \gamma Adz \tag{6}$$

where γ is the volumetric weight of the soil.

From the (6) of the forces acting on the wedge, 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 obtained as

$$\sigma_v = \frac{A\gamma - Uc}{Uk_0\tan\varphi}\left(1 - e^{\frac{-zUk_0\tan\varphi}{A}}\right) + p_0 e^{\frac{-zUk_0\tan\varphi}{A}} \tag{7}$$

III. Support Pressure on Tunnel Face

A. *Without Water Pressure*

Wedge analysis based on the limit equilibrium theory is adopted to obtain the support pressure for the tunnel face stability. For the sake of simplicity, the soil of wedge is considered to be homogeneous. A static equilibrium equation can be set up, assuming the failure criterion holds along the failure face. The forces acting upon the wedge at the face illustrates in Figure 2. There is 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 surface of the wedge.

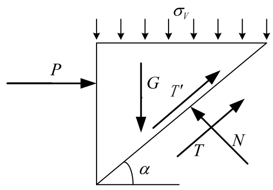

Figure 2. Forces acting upon the wedge without water pressure

The self-weight of wedge G

$$G = \frac{\gamma B^3}{2\tan\alpha} \tag{8}$$

The vertical mean stress σ_z' on the inclined sliding surface

$$\sigma_z' = \frac{2\sigma_v}{3} + \frac{B\gamma}{3} \tag{9}$$

The shear forces T on the inclined sliding surface

$$T = \frac{cB^2}{\sin\alpha} + N\tan\varphi \tag{10}$$

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

$$T' = \frac{B^2}{2\tan\alpha}\left(c + k_0\sigma_z'\tan\varphi\right) \tag{11}$$

Vertical earth pressure P_v on the top of wedge

$$P_v = A\sigma_v = BL\sigma_v \tag{12}$$

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By equating force in the vertical and horizontal direction, the equations of equilibrium in two dimensions are obtained as

$$P + T\cos\alpha + 2T'\cos\alpha = N\sin\alpha \tag{13}$$

$$P_v + G = T\sin\alpha + 2T'\sin\alpha + N\cos\alpha \tag{14}$$

Resolving the forces vertically and horizontally (13) and (14), one obtains the following expression for the minimize support pressure at the tunnel face

$$P = \varepsilon\left(\sigma_v BL + G\right) - \left(\frac{cB^2}{\sin\alpha} + 2T'\right)\left(\varepsilon\sin\alpha + \cos\alpha\right) \tag{15}$$

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

B. Considering the Water Pressure

When a shield tunnel is located below the river, the stability analysis of the shield tunnels face needs to consider the influence of water pressure. The water pressure usually is considered an external force [15] and illustrates in Figure 3. A hydrostatic distribution of water pressures along the slip surface is assumed.

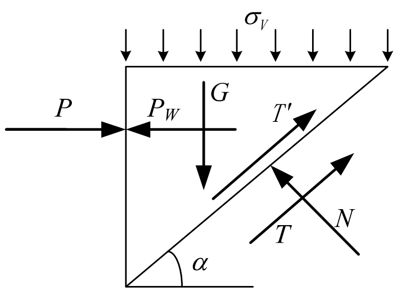


Figure 3. Forces acting upon the wedge with water pressure

Not taking the infiltration in excavation face into account, the overburden strata are assumed to be permeable with high permeability, such as sand, gravel and cobble, so a complete hydraulic connection exists between the river water and groundwater. In this sense, the water pressure generated by river water can be expressed as

$$P_w = B^2\left(\sum \gamma_w z_i + \gamma_w h\right) \tag{16}$$

where γ_w is the unit weight of water, z_i is the thickness of stratum No.i, which is located above the center of tunnel face.

By considering the pore water pressure and equating force in the vertical and horizontal direction, the equations of equilibrium in two dimensions are obtained as

$$P - P_w + T\cos\alpha + 2T'\cos\alpha = N\sin\alpha \tag{17}$$

$$P_v + G = T\sin\alpha + 2T'\sin\alpha + N\cos\alpha \tag{18}$$

The power of the limit support pressure on the tunnel face is

$$P = \varepsilon\left(\sigma_v BL + G\right) - \left(\frac{cB^2}{\sin\alpha} + 2T'\right)\left(\varepsilon\sin\alpha + \cos\alpha\right) + P_w \tag{19}$$

The support pressure is simplified to be uniformly, the minimize support pressure termed as limit support pressure which keeps the tunnel face stable is given as

$$\sigma_T = \frac{P}{B^2} \tag{20}$$

In the theory, this would provide a simple design method for limit face support pressure. The proposed method can be applied to calculate the limit support pressure on the face of the tunnel in multiple strata but only results for soil of homogeneity are presented due to page length limitations.

IV. NUMERICAL ANALYSIS

FLAC^{3D} code has been used for numerical investigation of stability of tunnel face in order to verify the method obtained in section Ⅲ. FLAC^{3D} is a three-dimensional explicit finite different program for engineering mechanics computation. Three-dimensional numerical model are shown in Figure 4, and only half ground and tunnel is simulated.

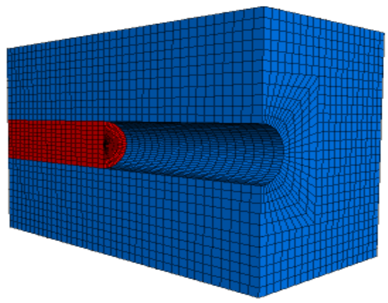


Figure 4 Three-dimensional numerical model

The parameters of tunnel and soils come from the Metro Line 1 across the Ganjiang River. The tunnel is excavated at a depth of 12 m below the river bed. The tunnel diameter D is 6 m. The water depth in river is 10 m. Consider the symmetry of the shield tunnel, only one half is included, a model considering a height of 30 m and a width of 21 m, is adopted. The mesh length of the model is 40 m. All the soil behaviors are simulated by using a linearly elastic and perfect plastic model with the Mohr-Coulomb failure criterion. The tunnel support parameters: segment is C50 concrete, segment thickness is 30 cm, and Young modulus of concrete segment is 34.5GPa, Poisson' ratio of concrete segment is 0.2, density is 2.45g/cm^3. Examples tunnel chosen in this study are assumed to be on the variable ground, and the elastic modulus E , Poisson' ratio ν , density ρ , friction angle φ and coefficient of earth pressure k_0 in Ganjiang River summarize in TABLE 1, which are those of coarse sand, gravel sand and cobble, respectively. The cohesion of coarse sand, gravel sand and cobble is zero. Water above the river bed surface is considered as a surcharge.

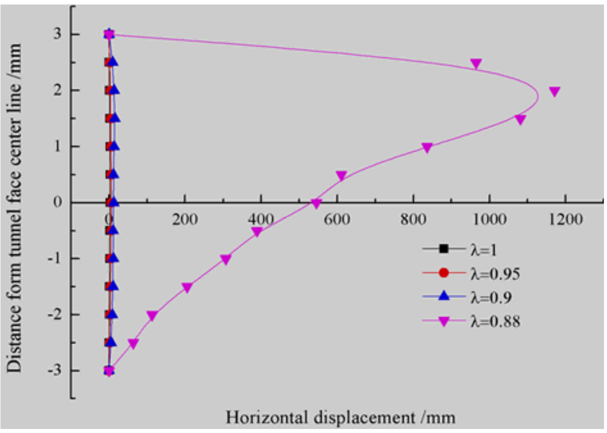
TABLE 1 PROPERTIES OF SOILS IN GANJIANG RIVER

stratum	E	ν	ρ	φ	k_0
	MPa	—	(g/cm^3)	(°)	—
coarse sand	35	0.32	1.90	32	0.27
gravel sand	40	0.30	1.95	35	0.27
cobble	42	0.28	2.10	42	0.24

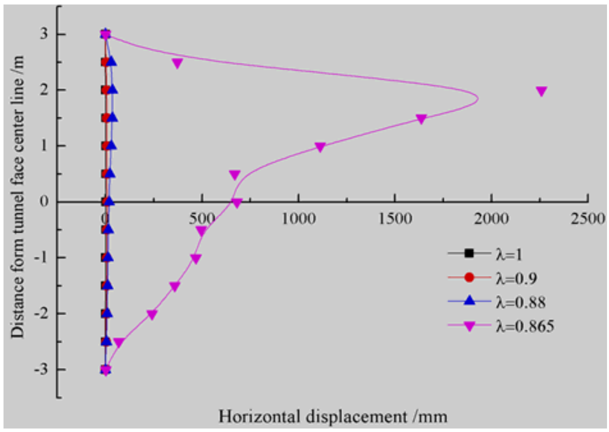
Numerical modeling for tunnel excavation using FLAC^{3D} is preformed to investigate the behavior of tunnel face during failure. The initial support pressure on the face is set as earth pressure, and is reduced until face collapse. The applied support pressure is normalized by the initial horizontal stress and termed the support pressure ratio λ.

The support pressure ratio is equal to the applied support pressure divided the initial horizontal stress at the center of the tunnel face. The initial support pressure is equal to the earth pressure at rest at center of tunnel face.

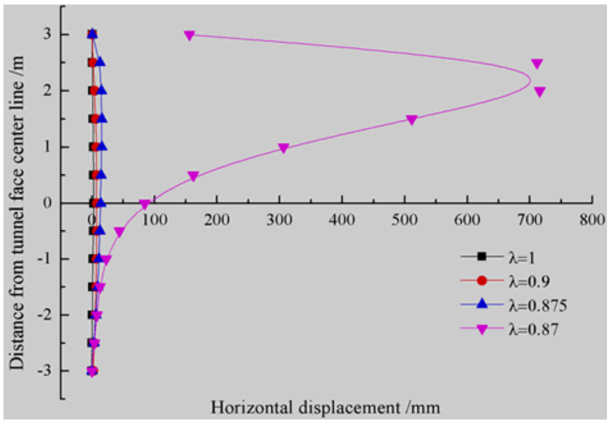
Figure 5 shows horizontal displacement along the center vertical axis of the tunnel face with variation of support pressure ratio for coarse sand, gravel sand and cobble. Large displacement is expected to be occurred with small support pressure ratio. The tunnel face kept stabilized when the support pressure ratio is large than 0.9, 0.88, 0.875 for coarse sand, gravel sand and cobble, respectively. The results of numerical analysis indicate the support pressure ratio have a great impact on displacement. It is found that the decreasing of support pressure ratio cause an increase in displacement. The tunnel face instability is assessed by sequentially decrease the support pressure ratio.



(a) Coarse sand



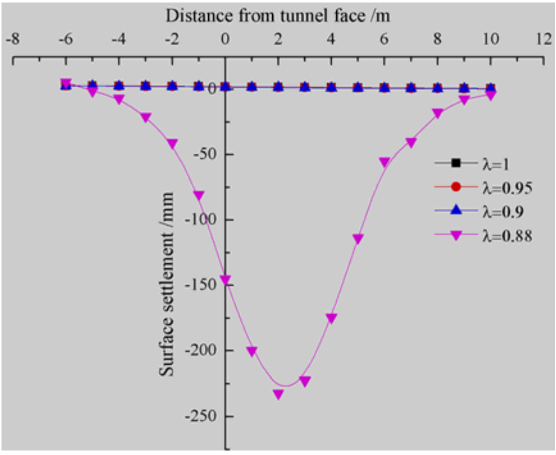
(b) Gravel sand



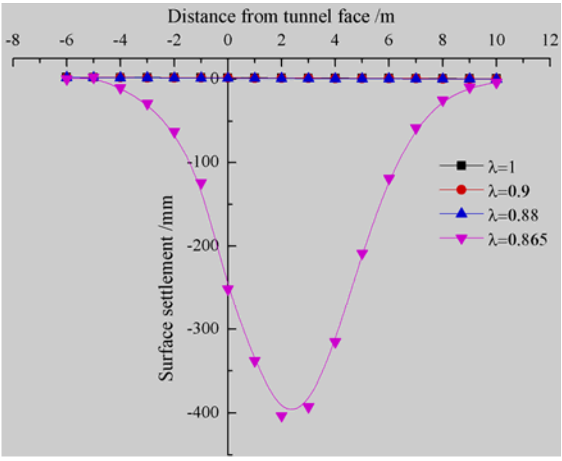
(c) Cobble

Figure 5 Relationship between horizontal displacements with support pressure ratio.

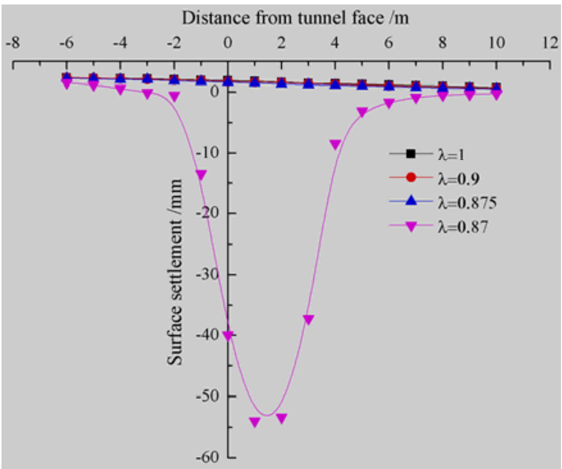
The developments of surface settlements for different support pressure ratio are compared in Figure 6. The maximum vertical settlement experienced is at almost 2 m distance from tunnel face. The failure mechanism of different stratum could be obviously observed by the plot in Figure 6. Surface settlement at initial stage is no obvious change, but it increases rapidly when the support pressure is less than a certain value.



(a) Coarse sand



(b) Gravel sand



(c) Cobble

Figure 6 Relationship between surface settlements with support pressure ratio.

In order to estimate the limit support pressure or limit support pressure ratio for tunnel face stability, horizontal displacements at the center node of tunnel face are plotted against the support pressure ratio in Figure 7. The face stability can be investigated by evaluate the relation between displacement of the center of face and the applied support pressure normalized by the initial horizontal stress [16].The maximum settlement at the surface of river bed against the support pressure ratio is shown in Figure 8. The results in Figure 8 are the same trend as the results obtained in Figure 7, the tunnel face become instability almost at the same limit support pressure ratio. The displacement increases rapidly when the support pressure ratio is less than the limit support pressure ratio. The settlement value of 20 mm can be used to determine the limit support pressure ratio and estimate the stability of tunnel face in practice engineering. The simulation method predicts that the limit support pressure 257.81kPa, 253.86kPa and 253.40kPa for coarse sand, gravel sand and cobble, respectively. The results of limit support pressure at the tunnel face are summarized by the FLAC^{3D} considering the water in river at TABLE 2.

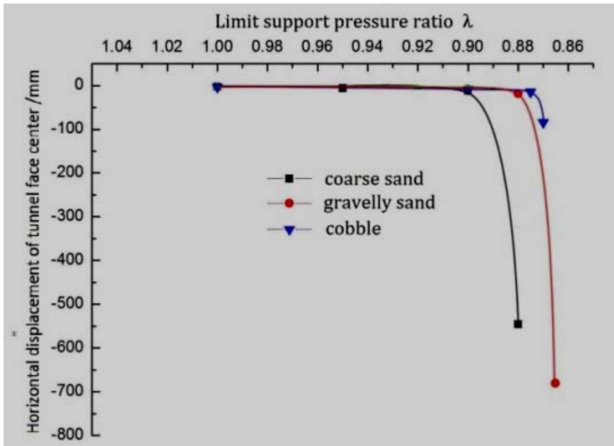


Figure 7 Horizontal displacement of the central point of tunnel face

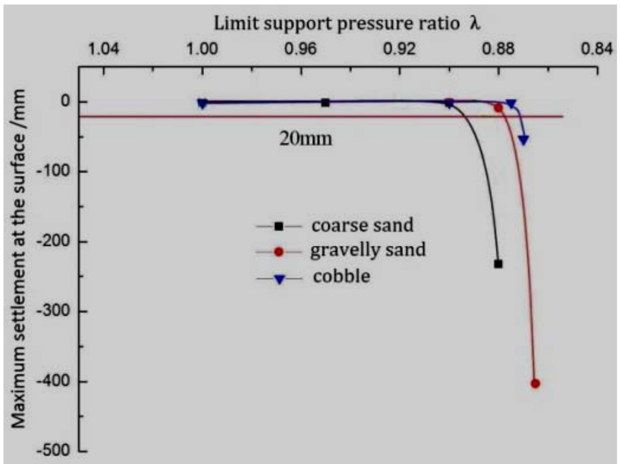


Figure 8 Maximum settlement at the surface.

TABLE 2 LIMIT SUPPORT PRESSURE BY FLAC

stratum	Initial earth pressure	Limit support pressure ratio	Limit support pressure
	KPa	—	*KPa*
coarse sand	286.45	0.9	257.80
gravelly sand	288.48	0.88	253.85
cobble	289.60	0.875	253.40

V. RESULTS

The limit support pressure at the tunnel face can be obtained by formulae previously presented with considering the water pressure in river. Limit support pressure acting on tunnel face by numerical analysis and theoretical analysis is shown in TABLE 3. The results of limit equilibrium method can be compared to the numerical method. The results of limit equilibrium method and numerical method appear to be in good agreement. The error between two methods is only about 5%. The limit support pressure of the limit equilibrium method leads to greater limit support pressure than when the numerical method is used. It is safe and efficient when the limit equilibrium method is used to design the support pressure on tunnel face.

TABLE 3 RESULTS OF LIMIT SUPPORT PRESSURE

	coarse sand	gravelly sand	cobble
numerical method	257.80	253.85	253.40
limit equilibrium method	269.04	267.97	263.99
error	4.18%	5.27%	4.01%

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