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Stability Analysis of Shield Tunnel Face Considering Spatial Variability of Shear Strength of Argillaceous Siltstone

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Abstract: The traditional research methods used to analyze the stability of shield tunnel face are mostly based on the assumption that the rock mass is homogeneous and isotropic material, which ignores the influence of heterogeneity and discontinuity of rock mass. Therefore, the influence of spatial variability of rock shear strength parameters on the stability of shield tunnel face should be studied. In this study, a random field method was used to establish the numerical model of heterogeneous rock mass, and FEM was used to analyze the failure mechanism and critical face pressure of the shield tunnel face. The influence of autocorrelation length of internal friction angle was analyzed. Failure mechanism and critical face pressure with different variability levels were recorded. Moreover, the reliability method was used to analyze the failure probability of shield tunnel face under different shield pressures, and the suggested face pressure under each variability level was studied. Finally, the failure analysis model was used to analyze the shield tunnel face collapse risk of section B3 of the Pearl River Delta Water Allocation Project. The results showed that the variability level of φ has a significant influence on the failure mechanism of the tunnel face. The stratum of high autocorrelation length refers to high suggested face pressure.

Keywords: Tunnel face failure; Shield tunnel; Random field; Autocorrelation Length.

1 Introduction

Face stability analysis is of great interest for shield tunnel excavations. The face pressure provided by the shield machine should give a good balance between preventing tunnel collapse and reducing economic costs. Besides, unsuitable face pressure could lead to soil disturbances impacting the safety of surface buildings and pipelines. Therefore, face pressure selection has important practical significance for maintaining the stability of the tunnel face and ensuring normal construction of the shield tunnel.

Broms et al. (1967) proposed an analysis method for describing the stability of excavation face in clayey soil under undrained conditions with stability coefficient. Based on the limit analysis method, Davis et al. (1980) deduced the range of upper and lower limit solutions of stability coefficient under a plane strain condition. Horn et al. (1961) proposed a wedge model based on the limit equilibrium method to describe the instability of the excavation face. The model assumes that the instability area is composed of the wedge in front of the excavation and the prism above. The wedge model is suitable for sandy soil strata and has a large calculation error for clay strata. Therefore, Jancsecz (1994), Broere (1998), Hu (2012), and Zixin Zhang (2014) have made appropriate improvements on the basis of the wedge model.

With the development of computer technology, numerical analysis has gradually become an important research method. Many scholars have carried out relevant research on the stability analysis of shield excavation face. In recent years, the effect of soil heterogeneity on the stability of tunnel surfaces has been studied. Mollon et al. (2010) studied the influence of the uncertainty of soil parameter cohesion and internal friction angle on the ultimate support stress of shield excavation surface based on the random response surface method. The analysis results show that the uncertainty of cohesion and internal friction angle has a certain influence on the ultimate support stress of the shield excavation face, and the influence of internal friction angle is more obvious than that of cohesion. Eshraghi et al. (2014) studied the limit support stress of excavation face at nine sections of Tehran Metro Line 3 constructed by earth pressure balance shield using probability analysis method and compared with numerical analysis results. The above conclusions reveal the influence of the uncertainty of soil parameters on the stability of shield excavation face to some extent. However, the method of considering soil parameters as random variables cannot fully reflect the spatial variability of parameters. Besides, the influence of spatial variability of soil parameters on tunnel excavation face is mostly analyzed under two-dimensional conditions, and the corresponding calculation results cannot truly and comprehensively reflect the stability degree and instability mode of the excavation face.

In this research, random field and 3D numerical analysis methods were used to analyze the influence of spatial variability of internal friction angle in argillaceous siltstone on the stability of shield tunnel face, including autocorrelation distance. The failure mechanism and critical face pressure of each variability level were analyzed, and the suggested face pressure was discussed.

2 Methodology

2.1 Random Field

Due to the deposition and tectonic movement, the spatial variant of rock parameters is not completely random but random and correlated. Vanmarcke (1977) used the first random field theory and established a random field model to describe the spatial variability of soil parameters. Based on the research results, this paper adopts the method of combining random field and numerical analysis with studying the stability of shield excavation face, mainly including the following five steps:

(1) build the 3D numerical model of shield tunnel and stratum by ABAQUS, which is a FEM numerical analysis software, as shown in Fig. 1. Output center coordinates of all elements in the model for generating random field models.

(2) Generate the random field model based on covariance matrix decomposition. The output center coordinates of the unit are substituted into the autocorrelation function and form the covariance matrix $\boldsymbol{C}_{n\times n}$, which is a positive definite symmetric matrix, where n represents the number of elements in the numerical model. Cholesky matrix decomposition method was used to get upper triangular matrix $\boldsymbol{L}$ and lower triangular matrix $\boldsymbol{U}$ in Mathematica, as shown in formula (1):

$$\boldsymbol{C} = \boldsymbol{L}\boldsymbol{U} = \boldsymbol{L}\boldsymbol{L}^T \tag{1}$$

Suppose $\boldsymbol{Y}$ is a vector of independent N(0,1) distributed random numbers. The standard normal random field $\boldsymbol{Z}$ could be expressed as formula (2):

$$\boldsymbol{Z} = \boldsymbol{L}\boldsymbol{Y} \tag{2}$$

Multiple generations of random fields could be realized by random generation of vector $\boldsymbol{Y}$.

(3) The random field vector $\boldsymbol{Z}$ is mathematically transformed to obtain random field models with different distribution characteristics. For example, the normal random field $\boldsymbol{Z}$ with mean value μ and variance σ^2 can be expressed as formula (3):

$$\boldsymbol{Z} = \sigma \boldsymbol{Z} + \mu \tag{3}$$

(4) Using Python programming language embedded in ABAQUS, the parameters in the random field model are mapped to the finite difference grid to realize the transformation from the random field model to the numerical analysis model, and then the numerical calculation is carried out for the stability of the shield tunnel face.

(5) Using Monte-Carlo Method, the random analysis of the stability of the shield tunnel face could be realized by repeated steps above. The data interaction process is shown in Figure 2.

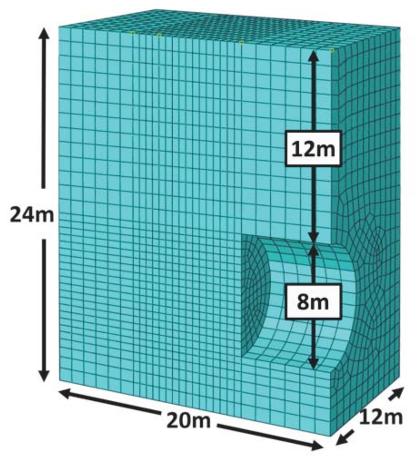

Figure 1. Numerical model of the tunnel.

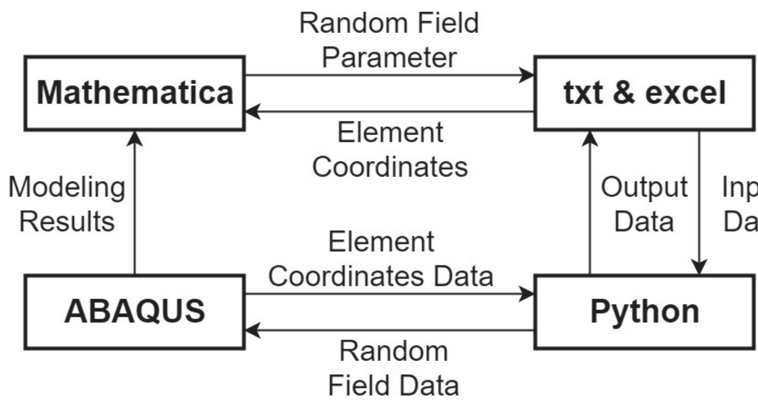

Figure 2. Data interaction process of random field generation.

2.2 Numerical Model

The geometry size for the geological model and the tunnel of interest is 20m × 12m × 24m. The diameter of the tunnel is 8m, and the depth is 12m. The top of the model is a free boundary, the surrounding is a normal displacement constraint boundary, and the bottom is a fixed boundary. Shell element was used to simulate tunnel lining. The physical and mechanical parameters of surrounding rock and lining structure materials are shown in Table 1. The influence of spatial variability of internal friction angle of surrounding rock on the stability of shield excavation face is considered in the model, and other parameters are constant. Since the parameters are positive, let the internal friction angle obey lognormal distribution with mean μ'_φ and standard deviation σ'_φ, as shown in Equations (4) and (5). Using the random field generation method introduced in Section 2.1 to generate a three-dimensional random field model with lognormal distribution. Firstly, a normal distribution random field with mean μ_φ and standard deviation σ_φ was established.

Table 1. Parameters of material.

Material	Unit Weight ***γ*** (kN/m³)	Internal Friction Angle ***φ*** (°)		Cohesion ***c*** (kPa)	Poisson Ratio ***v***	Elastic Modulus ***E*** (GPa)
		Mean	Standard Deviation			
Rock	18	22	8	50	0.4	45
Lining	24	/	/	/	0.2	30

$$\sigma'_\phi = \sqrt{\ln(1+\frac{\sigma_\phi^2}{\mu_\phi^2})} \tag{4}$$

$$\mu'_\phi = \ln(\mu_\phi) - \frac{1}{2}\sigma'^2_\phi \tag{5}$$

3D-Gaussian autocorrelation function was chosen to describe the variability of internal friction angle, which could be expressed as formula (6):

$$\rho(\tau_1,\tau_2,\tau_3) = \exp[-\pi(\frac{\tau_x^2}{\theta_x^2}+\frac{\tau_y^2}{\theta_y^2}+\frac{\tau_z^2}{\theta_z^2})] \tag{6}$$

where ρ is the autocorrelation coefficient, which is determined by the distance (τ_x, τ_y, τ_z) between two points in the random field. $\theta_x, \theta_y, \theta_z$ refer to autocorrelation lengths along x-, y-, and z-axis. Assuming that the random field of internal friction angle is isotropic distribution. Let $\theta_x = \theta_y = \theta_z = 1$m. One of the distributions of φ is shown in Figure 3, where deep color refers to a higher internal friction angle, and light color refers to a lower internal friction angle.

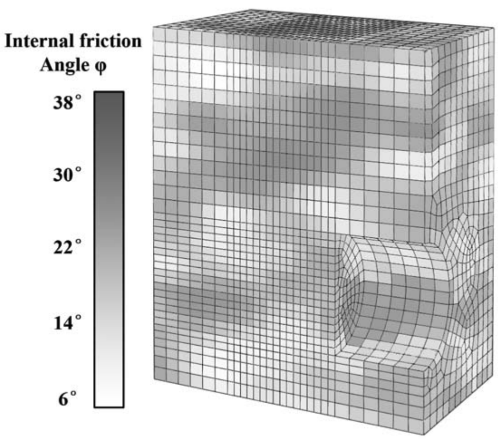

Figure 3. Distribution of φ with the random field.

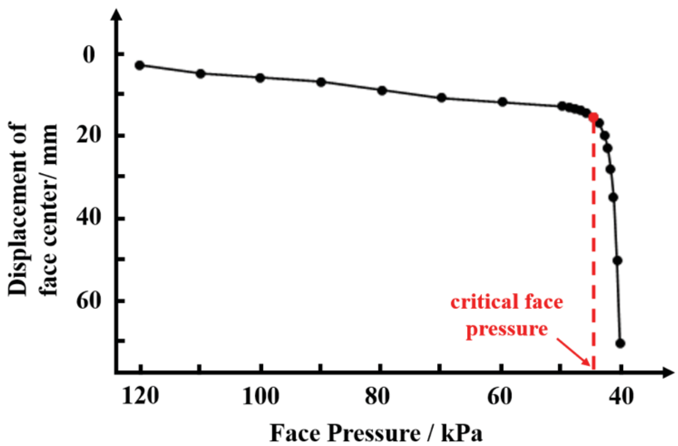

Figure 4. Definition of critical face pressure.

2.3 Limit State

Limit state of tunnel face stability could be obtained by decreasing face pressure gradually in numerical simulation. As shown in Figure 4, there is an inflection point in the face pressure and horizontal displacement diagram. The horizontal displacement is approximately linear with the face pressure and descent slowly before the inflection point. While after the inflection point, a steep drop occurs in the diagram, which could lead to tunnel collapse. Therefore, the face pressure at the inflection point could be chosen as critical face pressure.

3 Failure Analysis

Failure analysis was carried out based on the random field method. Four different uncertainty levels of θ (1m, 4m, 8m, and 16m) are considered in this study, and two-hundred times numerical simulations were conducted under each condition. Failure mechanism and critical face pressure were discussed in this part.

3.1 Failure Mechanism

In general, the collapse region of the tunnel face mainly includes two parts: a wedge in front of the face and a prism above it. The slip line will firstly exist at the rock mass with the lowest strength and highest stress ahead of the tunnel face, which is likely to appear at the lower and upper edge of the tunnel face. The slip line would continue to extend along the weak region and finally lead to collapse. Besides, since the distribution of rock shear strength is random, the results of each numerical simulation show different slip lines. Figure 5 and Figure 6 shows two typical failure mechanism: overall collapse and local collapse.

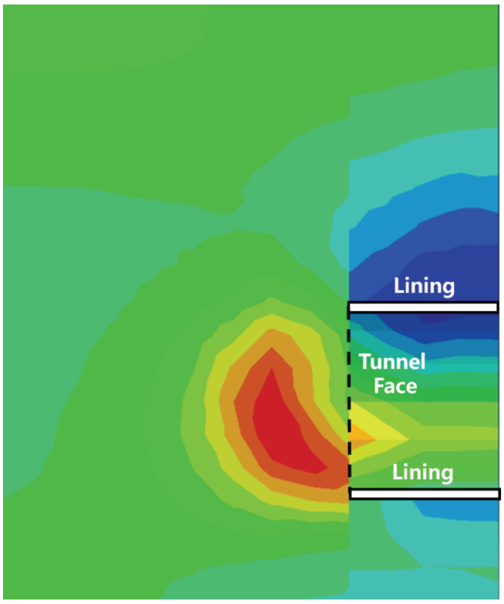

Figure 5. Overall collapse.

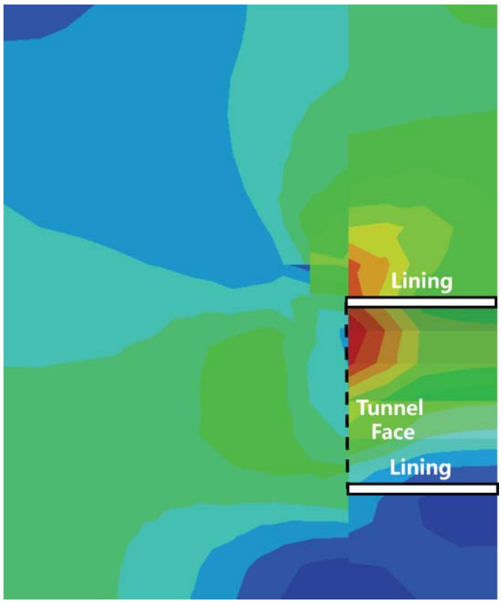

Figure 6. Local collapse.

It is found that the failure mechanism is closely related to the rock shear strength (φ) distribution within 6m ahead of the face. If the rock shear strength distribution within the range is uniform, the failure mechanism would likely be overall collapse. On the other hand, if the rock shear strength changes greatly in the region, especially if obvious stratification exists, a local collapse will be the main failure mechanism. In summary, when θ is close to D (tunnel diameter), the failure mechanism is multiple. Large high shear strength of rock elements group is difficult to form in the random field of low autocorrelation length, which could not affect the propagation of slip line and failure mechanism greatly. In limited random field implementation, the difference in shear strength between elements is not obvious. So the failure mechanisms of the cases with too large or too small autocorrelation length are single, which mainly are overall collapse.

3.2 Critical Face Pressure

Based on a large number of numerical simulations, the critical face pressure of each case was recorded and shown in Figure 7. It is found that critical face pressure is likely to obey normal distribution and be concentrated near 40kPa. Besides, the cases of high autocorrelation length exhibit more obvious discreteness than that of low autocorrelation length.

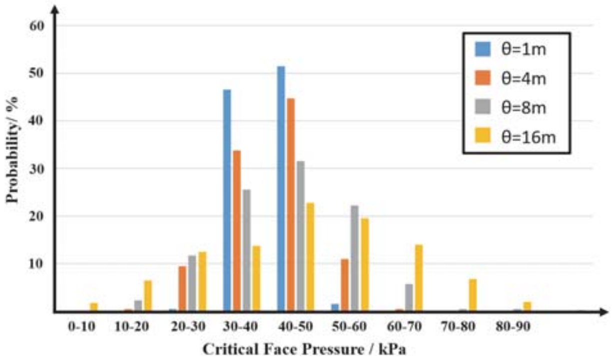

Figure 7. Distribution of critical face pressure.

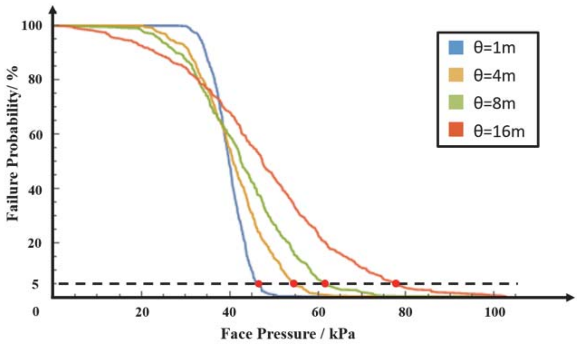

Figure 8. Relationship of failure probability and face pressure.

In order to analyze the appropriate face pressure for shield tunnel excavation of each case, the relation between failure probability and face pressure was shown in Figure 8, based on the distribution of critical face pressure in Figure 7. It is obvious that the failure probability decreases with the increase of face pressure, and each case shows a different trend. The case of high autocorrelation length shows a slow decline trend, while low autocorrelation length cases decline sharply in a small range.

According to the method of the eigenvalue of soil parameters selection in the European Geotechnical Design Code, the eigenvalue of face pressure (σ_k) should be combined with failure probability, and keep the failure probability less than 5%. The probability of critical face pressure larger than σ_k in multiple numerical simulations could be expressed as formula (7):

$$P_f = \frac{N_f}{N} \times 100\% \tag{7}$$

Where N is the times of numerical simulation of each case, N_f is the times of critical face pressure larger than σ_k in the simulation of each case. P_f refers to the failure probability with the face pressure of σ_k.

Let the failure probability equals 5%, the σ_k of each case can be found in Figure 8, and the relation between σ_k nd θ is shown in Table 2. It is found that high face pressure should be taken to reduce failure risk in high autocorrelation length cases. Besides, a reasonable selection of statistics to characterize the spatial variability of soil parameters has an important influence on the stability evaluation of tunnel faces.

Table 2. Eigenvalue of face pressure in each case

Autocorrelation Length (θ)	θ=1m	θ=4m	θ=8m	θ=12m
Eigenvalue of Face Pressure (σ_k)	47.23kPa	58.84kPa	62.16kPa	77.92kPa

4 Case Study

It is proposed to use the face pressure analysis model to evaluate face pressure and collapse risk based on section B3 of the Pearl River Delta Water Allocation Project. Section B3 is 11.36km long and has four shield sections. The underwater shield section is 4.2km long and crosses the Lianhua Mountain River and the Shiziyang River. The diameter of the tunnel is 8.65m, and the depth is about 46m. There are complex geological conditions in this project, including nine major faults, unidentified uneven weathering strata, and mud-bearing strata. The underwater section was excavated by mud balanced shield machine. The collapse of the tunnel face and ground settling are typical risks of this project. The roadmap of the B3 section and geological section map for shield tunnel section 2 is shown in Figure 9 and Figure 10.

Figure 9. Tunnel roadmap of B3 section.

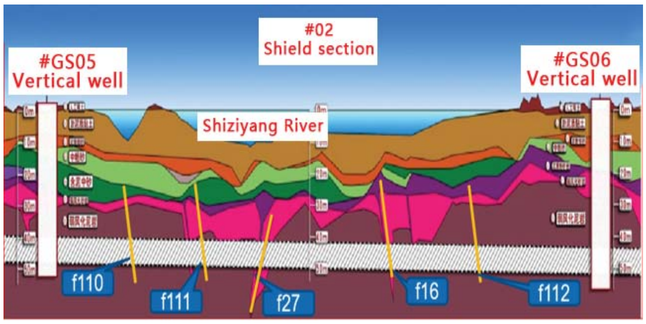

Figure 10. Geological section map of shield section 2.

According to the stratigraphic distribution map, various weathering levels of argillaceous siltstone were passed through by the tunnel. Based on the statistical data and the result of the geologic investigation, assume that the autocorrelation length of micro weathered, weak weathered, strong weathered, and fully weathered argillaceous siltstone are 16m, 8m, 4m, and 1m. To keep the failure probability less than 5%, according to the face pressure analysis in section 3, the suggested face pressure in this tunnel is shown in Figure 11. Compared with the actual face pressure (100kPa) used in the excavation, risk control of tunnel face collapse is eligible overall. Considering that the geologic investigation near the #06 vertical well was insufficient, soft rock mass may exist in this area, and face pressure control ought to be higher.

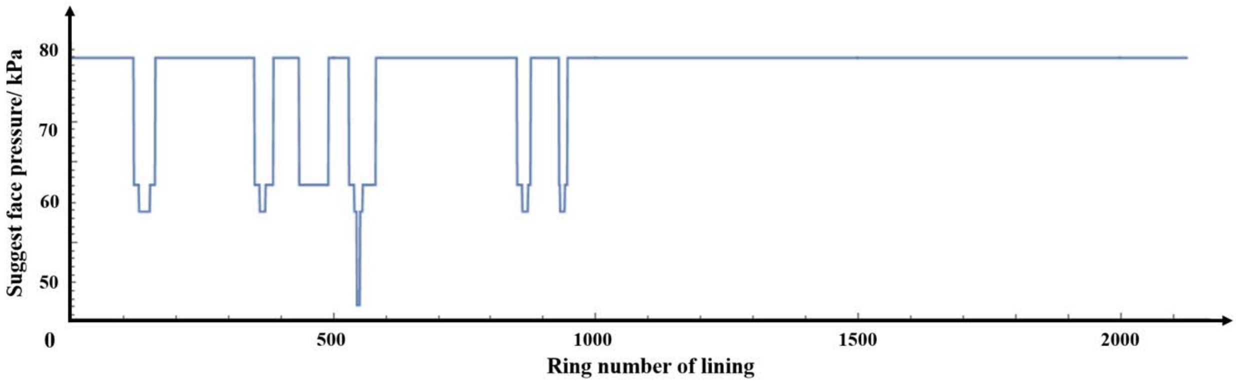

Figure 11. Suggest face pressure of B3 shield tunnel section.

5 Conclusion

A stability analysis method of tunnel face based on the random field method and numerical analysis is proposed. Spatial variability of internal friction angle of rock mass was studied, and the influence of variability on face collapse was analyzed. The conclusions are as follows:

- Failure mechanisms include local collapse and overall collapse, which are associated with autocorrelation length. Autocorrelation length near the tunnel diameter refers to multiple failure mechanisms, while too large or too small autocorrelation length refers to overall collapse more likely.
- The critical face pressure is likely to obey normal distribution. The critical face pressure distribution of low autocorrelation length has a smaller variance than that of high autocorrelation length.
- Based on the reliability method, the suggested face pressure of each autocorrelation length case was calculated. Without another factor, high autocorrelation length refers to high suggested face pressure, while low autocorrelation length refers to low face pressure.

Besides, the variation coefficient and other factors are not included in this study. Autocorrelation distance has advantages in describing the degree of parameter variation, which is suitable for the research of face pressure with different weathering degrees. While, the variation coefficient and the decline of the mean of shear strength needs to be considered for fault excavation.

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