Finite element analysis of steep excavation slope failure by CFS theory*

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Abstract The distribution of Coulomb failure stress (CFS) change in the steep excavation slope is calculated by finite element method in this paper, and the failure mechanics under different conditions have been investigated. Comparing the CFSs before and after the slope excavation (stress loading and unloading processes), the dangerous internal zone and the most likely failure external area are attained. Given the shear cracks on the top surface while tensile stress or cracks along the toe of the slope, we analyze the high cutting-angle steep slope in Kaixian county of the Three Gorges Reservoir region. We bring forward that the peak value of CFS after excavation can reach to the order of 0.1 MPa, which is greatly higher than that of before. Our preliminary results are useful for optimizing the reinforcement structure during the steep slope stabilization engineering.

Key words: Coulomb failure stress; stabilization and failure evaluation; steep slope excavation
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1 Introduction

With rapid development of economy, more and more engineering activities are being carried out especially in the hilly regions. Slope excavation is one of these behaviors, stability of which is one of the major concerns of design and construction. Especially in Three Gorges region, there are thousands of steep excavation slopes (Luo and Wu, 2004). The stabilization evaluation of steep excavation slope is immensely complex in geotechnical engineering. After excavation, the mechanic balance and the stress state of the slope will self-adjust in a certain range, which may produce grave-ness influence on engineering safety.

In recent years, both qualitative and quantitative methods have been put forward to assess the stability of the excavation slope. For example, limit equilibrium method (LEM), which includes Bishop's method, Janbu's simplified method, and improved sophisticated method, bases on the shear strength along the failure

surface to obtain the minimum safety factor (Griffiths and Lane, 1999; Singh et al., 2008; Pastor and Loue, 2010; Zhang and Zheng, 2004; Zheng et al., 2004; Li et al., 2009; Li and Zhang, 2009). Fuzzy analytical method, neural network method and hierarchical cluster analysis method are widely adopted to estimate the slope stability (Zhang, 1997; Luo and Wu, 2004; Su et al., 2005). Finite difference method (FDM), boundary element method (BEM) and strength reduction method (SRM) are developed to calculate the realistic stress distribution during slope excavation. Recently, the finite element method (FEM) becomes one of the most popular simulation tools to solve this kind of problem (Xu et al., 2003; Song et al., 2005; Zhu et al., 2009b; Zheng et al., 2009; Liang and Ma, 2010).

Once the initial stress, the stress change and the rock stress strength of slope are known, we can evaluate when and where the slope will failure during the excavation process. It is very difficult to obtain the initial stress distribution of the slope due to the lack of high resolution observation stress data in respective slope. However, the relative stress changes before and after the slope excavation is easier to be obtained by the numerical calculation. Based on the previous work (Zhu et al., 2009a, 2011), we take the Coulomb failure stress (CFS)

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theory to evaluate the steep slope excavation (Zhang et al., 2009). The most dangerous internal area, the most possible failure external surface area and the slip angle can be attained through this method. We apply this theory to the high cutting-angle steep slope in Kaixian county of the Three Gorges Reservoir region so as to explore the failure mechanism of the steep excavation slope under a variety of initial stress conditions using finite element method. The CFS change pattern is helpful to evaluate the stability of steep excavation.

2 Coulomb failure stress (CFS) theory

2.1 Introduction

Recently, the Coulomb failure stress (CFS) theory is used frequently to the study on earthquake triggering. Many researchers have studied the relationship between aftershocks and major shock using CFS theory so as to discuss the earthquake triggering (Stein and Lisowski, 1983; Stein et al., 1992; King et al., 1994; Freed and Lin, 2001; Shi, 2001). Shi and Cao (2010) improved the dynamic CFS formula and calculated the CFS increment of Wenchuan earthquake using three different models. Wan et al. (2007) proposed that the vast majority of aftershocks occurred in the region where the $\Delta\text{CFS} > 0$ during their study on Tangshan earthquake. Lei et al. (2008), Lei (2010) and Ge et al. (2009) discussed the impoundment and drainage processes of Zipingpu reservoir and their impact on the Coulomb stress migration in Longmenshan fault zone to investigate the possibilities of triggering the Wenchuan earthquake.

The focal mechanism of an earthquake describes the coseismic deformation in the source region. It is derived from the moment tensor solution of the earthquake, which itself is estimated by analyzing the observed seismic waveforms right after the earthquake. By adopting the constrains from the field observation and aftershock distribution, the rupture direction along the fault plane, the specific fault dip angle, strike angle and the slip angle are achieved, respectively. According to the theory of Mohr-Coulomb and Hook-Brown strength criteria, the shear stress can be expressed as

$$\tau = c + \mu(\sigma_n - p), \quad (1)$$

where τ is the shear stress of the fault plane, c is the cohesion, μ is the friction coefficient and σ_n is the normal stress of the fault plane (in this paper, the tensile stress is defined as positive), p is the pore pressure, respectively.

Because of the visco-elastic properties of Earth's crust, the stress migration will happen after an earthquake. The local stress state varies dramatically due to the highly focused deformation and strain, especially in the regions close to the epicenter. Ideally, the overall stress state is available through either kinematical or dynamical models. However, if the initial stress is unknown, only the stress change may be calculated according to the Coulomb failure criterion, in which ΔCFS is formulated as

$$\Delta\text{CFS} = \Delta\tau + \mu(\Delta\sigma_n + \Delta p), \quad (2)$$

where $\Delta\tau$ and $\Delta\sigma_n$ are the shear stress and normal stress variations along the fault plane, and Δp denotes the change in pore pressure, respectively. Coulomb's friction hypothesis is used to determine the combination of shear and normal stresses that will introduce a fracture of the material when ΔCFS is positive (Hariri and Bilek, 2011; Katsumata, 2011; He et al., 2011; Ogata, 2010; Santoyo et al., 2010; Soltanzadeh and Hawkes, 2008; Heidbach and Ben-Avraham, 2007; Robinson, 2003).

In this paper, we put forward the ΔCFS criterion to assess the stability of the excavation slope without considering the pore pressure. Given that every particle of slope is surrounded by pseudo fault planes on all sides, we can attain an optimally oriented plane that is most likely instable by rotating the strike angle and slip angle from 0° to 360° , and the dip angle from 0° to 90° , as shown in Figure 1. To 2-D problem, we just rotate the strike and dip angles.

Figure 1 Sketch of attaining the optimal direction of CFS.

Assumed that the initial stress is σ_{ij}^0 and the three characteristic angles of the pseudo fault plane are strike angle φ , dip angle δ and slip angle λ , respectively, the normal vector of the pseudo plane can be expressed as

$$\mathbf{n} = (n_1, n_2, n_3) = \begin{Bmatrix} \sin \delta \cdot \cos \varphi \\ -\sin \delta \cdot \sin \varphi \\ \cos \delta \end{Bmatrix}. \quad (3)$$

The normal stress σ_n^0 and shear stress τ_0 of the pseudo fault plane are obtained by

$$\sigma_n^0 = \sigma_{ij}^0 n_j n_i \quad (4)$$

and

$$\tau_0^0 = \sigma_{ij}^0 n_j s_i, \quad (5)$$

where s_i is the slip direction which can be expressed as

$$\mathbf{s} = \begin{Bmatrix} \sin \varphi \cos \lambda - \cos \varphi \cos \delta \sin \lambda \\ \cos \varphi \cos \lambda + \sin \varphi \cos \delta \sin \lambda \\ \sin \delta \sin \lambda \end{Bmatrix}. \quad (6)$$

After the excavation, the stress field of the slope is rebuilt. If the stress tensor change is $\Delta\sigma_{ij}$, the new stress field can be written as

$$\sigma'_{ij} = \sigma_{ij}^0 + \Delta\sigma_{ij}. \quad (7)$$

The new normal stress and shear stress are denoted by σ'_n and τ' , respectively. According to equation (2), the CFS and the orientation of the maximum Δ CFS can be calculated on each pseudo fault plane, which are prone to approximate the direction of the rupture. The envelope of these maximum Δ CFS directions on each pseudo planes forms the potential failure surface.

2.2 Model validation

In order to verify the reliability of this method, we use a simple example to compare it with the result from traditional Morgenstern-Price method, which is a general method of slices developed on the basis of limit equilibrium and requires satisfying equilibrium of forces and moments acting on individual blocks, based on Geo-SLOPE/W accepted as an integrated tool about geotechnical modeling. A homogeneous rock slope with a height of 10 m and length of 20 m is considered, as shown in Figure 2. The cohesive strength of the rock is 5 kPa and the friction angle is 20° . The unit weight, elastic modulus and Poisson ratio of the rock slope are 15 kN/m^3 , 1.0 GPa and 0.3, respectively.

We calculated the minimum safety factor and the most likely sliding surface after excavation using Morgenstern-Price method (Figure 3a). In general, the minimum safety factor is an indicator of estimating the stability of slope based on the Coulomb failure criterion. Its value is equal to the ratio of sliding force to skid resistance. Figure 3b shows the result from our finite

element method based on the linear elastic theory and Δ CFS calculation. By introducing the FEM method, we can detect not only the most dangerous part but also the possible crack direction and the crack properties on each point of the slope, whether they are tensile crack or shear crack according to the normal stress and shear stress changes. If the normal stress variation is positive and larger than shear stress change, the crack is defined as dilatation crack. Otherwise, the crack is shear crack. At the same time, combining the Δ CFS contours, we joint the optimally oriented plane of every point to attain the slip plane. As shown in Figures 3a and 3b, those results from different methods agree with each other very well.

Figure 2 The sketch of the benchmark model.

Figure 3a shows the safe factor and failure surface using Morgenstern-Price method. The red quiver is the feature of the GEO/SLOPE soft to show the entry and exit location of the slip surface, which means that the most possible start parts of the slope crest and the end parts of the slope foot. Figure 3b shows the envelope of the CFS changes and the crack type (every 100 points). The black pseudo fault plane shows the contour of the Δ CFS and the most possible slip surface.

3 Application of Δ CFS method to the high cutting slope stability analysis

In this paper, we analyze the high cutting rock slope located in the north of Kaixian in Three Gorges region, as shown in Figure 4. After excavation, the slope deformation and rock collapse appeared, which are the serious threat to lives and property of the local people. A reasonable stress analyses are needed to be proposed on the possible engineering solutions before the appropriate measures of reinforcement are taken.

Figure 3 Comparison between Morgenstern-Price method (a) and ΔCFS method in this study (b).

Figure 4 Geological survey of the excavated slope. The slope is composed of consequent part (section I) and tangential part (section II). It is one of the forty nine protective slopes in the city, and it is only less than 4 m away from the civilian buildings. The maximum height of the slope is 37.1 m and the length is 80 m.

The slope inclination varies from 57° to 333° and its dip angle varies from 53° to 57° . The total area is $3\,520\text{ m}^2$. Based on the geological field observation data, we constructed the numerical simulation model according to the section profile 1-1, as shown in Figure 5. We found that the rock slope is composed of the moderately weathered mudstone (J_2s). The slope summit is

composed of Quaternary sediment (Q_4^{el+dl}), gravel stone and intense weathering mudstone (Q^s).

The geological profile 1-1 shows clearly that the Quaternary sediment (Q_4^{el+dl}) and intense weathering mudstone (Q^s) are comparatively thin. Their thicknesses is only 0.2–3.0 m, while the thickness of moderately weathered mudstone (J_2s) is around 1 288 m.

Figure 5 Geological profile 1-1 (upper-left) and the 2D physical model of the investigated slope.

3.1 Numerical model

The rock mass in the Kaixian region is regarded as elastical media. Based on linear elastical theory, the continuity equations for model can be expressed by the following:

$$\nabla \cdot \mathbf{u} = 0, \quad (8)$$

the geometric equation is

$$\dot{\varepsilon}_{ij} = \frac{1}{2}(u_{i,j} + u_{j,i}), \quad (9)$$

the equilibrium equation (where the body forces are neglected) is

$$\frac{\partial \sigma_{ij}}{\partial x_j} + f_i = 0, \quad (10)$$

and the constitutive equation is

$$\begin{pmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz} \\ \sigma_{yz} \\ \sigma_{xz} \\ \sigma_{xy} \end{pmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{pmatrix} 1-\nu & \nu & \nu & & & \\ \nu & 1-\nu & \nu & & & \\ \nu & \nu & 1-\nu & & & \\ & & & 0.5-\nu & & \\ & & & & 0.5-\nu & \\ & & & & & 0.5-\nu \end{pmatrix} \begin{pmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz} \\ \varepsilon_{yz} \\ \varepsilon_{xz} \\ \varepsilon_{xy} \end{pmatrix}, \quad (11)$$

where E is the Young's modulus and ν is Poisson's ratio. In order to improve the accuracy of our numerical model, the whole computing domain is enlarged to a great extent to reduce the numerical errors from boundary conditions. In our numerical model, the horizontal size dimension is around 2.5 times as large as that of the slope height. Vertically, the computing domain's highness is twice as high as that of the actual slope height. In total, there are 14 600 triangle elements consisting of 7 530 nodes. Because the initial stresses are unknown, we compared the results from four different kinds of models with different kinds of boundary con-

ditions. Those physical models and their correspondent parameters are shown in Tables 1 and 2. About the value of the friction coefficient μ , we define $\mu = \tan\phi$, where ϕ is the friction angle.

Table 1 Model boundary conditions

Boundary	Model I	Model II	Model III
Surface boundary	Freedom	Freedom	Freedom
Left boundary	Fixed	$0.5\rho gh$ (uniform load)	ρgh (linear load)
Lateral and bottom boundary	Fixed	Fixed	Fixed

Table 2 Physical parameters of the slope

Rock type	Weight/(kN·m ³)	Young's modulus/MPa	Poisson's ratio	Cohesion/MPa	Friction angle/°
Quaternary sediment	19.5	18.5	0.25	0.015	18
Intensely weathered mudstone	24.80	6 000	0.22	0.20	28
Moderately weathered mudstone	25.38	7 000	0.20	0.64	32

3.2 Simulation and discussion

3.2.1 Stress increments of each model

Figure 6 shows the variation of stress (σ_{xx} , σ_{xy} and σ_{yy}) for each model. After excavation, the stresses are adjusted, especially the one at the foot of the slope. Comparing three models with different boundary conditions, on the whole, the distributions of σ_{xx} , σ_{xy} and σ_{yy} are nearly the same, but we can see that there are still many differences in the distributions of σ_{xx} and σ_{xy} . Because of the existing crust tectonics in the left boundary of model II, there emerge σ_{xx} centered largely in the left part of the slope and more obvious σ_{xy} concentration in the right toe of the slope, which indicates that the crust tectonic stress field should not be neglected, especially in dealing with the tension crack near the toe of the slope.

3.2.2 Displacement increments of model I

Figure 7 shows each component of the displacement increment vector in model I. Here, we take model I for an example to analyze the characteristics of the excavated slope. Compared with the displacement increments in x and y directions, the vertical displacement increments is one order of magnitude larger than horizontal ones. And the largest normal stress increments are nearly twice as many as the horizontal normal stress at the foot of the excavated slope, where appears the stress concentration. There are positive shear stress increments near the slope surface while negative shear stress increments are at the foot of slope.

3.2.3 Δ CFS of model I

Substituting the variable stress parameters into

equations (3) and (4), we attain the distribution of the Δ CFS in each model with the maximum value of each model being 1.95×10^5 Pa, 2.95×10^5 Pa and 2.2×10^5 Pa respectively. Taking the model I for an example, the optimal fracture direction is shown in Figure 8. We know that the largest CFS increments appear at the foot of the slope which are the most dangerous parts. For the more upper load reduction, the more vertical normal stress reduced. From equation (2), the CFS of the excavated slope is larger than that before excavation. There are shear cracks on the top excavation surface while there are tensile cracks at the toe of the slope. This can interpret the phenomenon why there are a lot of rubble and broken stones in the field. So we should adopt the reinforcement measures at the foot of the slope when we excavate slope and reinforce the excavated face. Comparison of the four models shows that the maximum Δ CFS appears in model II, which demonstrates the bigger the tectonic stress, the more unstable the excavation slope.

4 Conclusions

Now, we can not attain exactly the unstable parts of the excavation slope when we do not know the tectonic stress. According the CFS theory, we can know the relatively more dangerous parts where the Δ CFS is larger than zero. Based on CFS theory, the steep excavation slope is analyzed. From the variable simulation of CFS field before and after the excavation, the most dangerous internal area and the most likely failure

Figure 6 The results of stress changes of each model. (a) Stress σ_{xx} ; (b) Stress σ_{xy} ; (c) Stress σ_{yy} .

external surface area are attained. Through analyzing the high cutting-slope at the Kaixian county in the Three Gorges Reservoir region, we obtain that the CFS after excavation is bigger than that before excavation; there are shear cracks on the top excavation surface while the tensile cracks along the toe of the slope and the largest CFS increment reaches to the order of 0.1 MPa. King et al. (1994) indicated that obvious seismic activity could be observed if the stress change reaches 0.01 MPa. So, even the ΔCFS is only several kPa, the rock may be destroyed and the slope may collapse. At the same time, the results show that the tectonic stress boundary conditions have a great influence on the val-

ue of ΔCFS . The difference of the maximum ΔCFS can reach to 30% in the three models. The CFS distribution as position function can be helpful to study the failure mechanism of steep excavation slope, which is favorable for designing and taking reinforcement measurement in engineering application.

In this paper, we just put forward the effects of the stress boundary. According to mechanical mechanism, we will make further study on the differences between Dirichlet displacement boundary and Newman stress boundary in the problems about excavation slope. And we also will consider the factor of rainfall and the softening effect of the rock or fault into our models.

Figure 7 Increments of several vectors in model I. (a) Horizontal displacement increment Δu ; (b) Vertical displacement increment Δv ; (c) Stress increment $\Delta\sigma_{xx}$ in x direction; (d) Stress increment $\Delta\sigma_{yy}$ in y direction; (e) Shear stress increment $\Delta\sigma_{xy}$.

Figure 8 Distribution of the ΔCFS of model I.

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