Calculation Method for Unstable Rock Stability based on Dominant Fissure Model

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Abstract. In accordance with in-situ exploration and years of experience in unstable rock-related design, this paper states that the Mohr–Coulomb model is insufficient for the selection of the shear strength parameters of the dominant fissure of unstable rocks. Therefore, a JRC-JCS model of dominant fissure for three types of unstable rocks was established, which is more applicable to construction practice. Based on this model, calculation methods were obtained for the three types of unstable rocks, and the stability coefficient expressions of the unstable rocks were determined. The stability of five typical unstable rocks in Wanzhou District, Chongqing Municipality, China, namely, W12, W22, W59, W4, and W16, was calculated by these methods. As indicated by the calculation results, the unstable sliding rock W12, unstable toppling rock W22, and unstable falling rock W4 are stable, while the unstable toppling rock W59 and unstable falling rock W16 remain unstable and thus need reinforcing. These calculation results are relatively in line with the actual situation, according to a verification by in-situ monitoring from 2003 to 2013. This further verifies the practicality and applicability of the above calculation methods, which are based on the dominant fissure model. These methods also offer a theoretical basis for the amendment of the local standard, namely, Chongqing’s design specification for prevention and control projects of geological hazards (DB50/5029-2004), in terms of calculation methods for unstable rock stability.

List of Symbols

\( \tau \) shear strength [kPa]
\( \sigma \) normal stress [kPa]
\( \varphi \) angle of internal friction [°]
1. Introduction

Unstable rocks refer to rock blocks and their combinations that are cut from the structure of multiple sets of rock masses; unstable rocks are located on cliffs and steep slopes and have poor stability. They can be classified into three types (Figure 1) according to their instability modes, namely, unstable sliding rocks, unstable toppling rocks, and unstable falling rocks (Chen et al. 2006; 2007)[1,2]. China is a mountainous country; mountainous and hilly areas account for more than two-thirds of the total area of its territory. The collapse of unstable rocks, which is a main geological hazard in mountainous areas, is an evolutionary process of geomorphology induced by geological disasters. According to the National Geological Disaster Bulletin, issued by the Ministry of Land and Resources of the People’s Republic of China, an average of 2000 disasters caused by the collapse of unstable rock have been occurring every year in China over the past decade. For example, 5575 disasters occurred in 2011, and 2319 disasters occurred in 2010. Disasters caused by the collapse of unstable rocks account for 30%–40% of geological disasters, leading to direct economic losses of more than RMB 1.3 billion per year.
Furthermore, about 70% of these disasters occur in the western regions. For instance, a mountain collapse happened near the Chediguan Bridge, located in Duwen Road (K44+200) of No. 213 National Road in Wenchuan County, Sichuan Province, on July 25, 2009. The bridge pier was damaged by the collapsed rock mass, the 60 m long bridge deck collapsed, and two trucks fell into Min Jiang River, resulting in the death of six persons and direct economic losses of about RMB 100 million. Collapse disasters have been increasingly occurring with the rapid development of western China, especially the execution of large-scale projects, such as the construction of reservoir regions and tunnel excavation. Therefore, disasters caused by unstable rocks should be urgently reduced or prevented by calculating the stability of unstable rocks and analyzing its mechanism.

![Figure 1. Examples of three types of unstable rocks: a) unstable sliding rock, b) unstable toppling rock, c) unstable falling rock.](image)

Presently, the calculation methods for the stability of unstable rocks are still in the initial research stage and focus on the leading role played by the dominant fissure model in the stability of unstable rocks. For example, Chen and Tang[1,3,4,5,6,9,10,15] proposed the rigid limit equilibrium method and a calculation method for fracture mechanics based on the Mohr–Coulomb model of dominant fissure. In the design specification for prevention and control projects of geological hazards (DB50/5029-2004; hereinafter referred to as the Specification), a local standard of Chongqing issued in 2004, an equivalent calculation method that integrates the intensity parameters of the dominant fissure was established based on the intensity parameters of the connecting and disconnecting portions of the dominant fissure and in accordance with the length weighting. According to the penetration level of the dominant fissure and safety grades for the prevention and control of unstable rocks, Tang and Chen[1,2,3,5,6,9,10,15] developed the penetration rate method through model tests; they also built a damage model for the dominant fissure and a life calculation method for the fatigue fracture of the dominant fissure of unstable rocks. The above academic achievements concerning the analysis and calculation of unstable rocks’ stability based on the dominant fissure provide many valuable theories for subsequent research. On this basis, the authors of the current paper explore and discuss dominant fissure models for unstable rocks that are highly applicable to engineering practices; furthermore, calculation methods are proposed for the stability of three types of unstable rocks based on a dominant fissure model. The research achievements obtained in this study can be treated as a theoretical basis for the evaluation and design of the stability of unstable rocks.
2. Dominant fissure models for unstable rocks

A dominant fissure refers to a surface or belt with penetration or intermittent penetration and poor mechanical strength [1,2,6]. The formation of a dominant fissure is a key factor affecting the stability of unstable rocks and a necessary stage for the development of unstable rocks. The stability of an unstable rock mass is determined by dominant fissure models for unstable rocks and shear strength parameters. The Mohr–Coulomb criterion is the currently used dominant fissure model for unstable rock masses[4,5,10].

\[ \tau = \sigma \tan \phi + c \]  

(1)

where \( \tau \) means the shear strength of the dominant fissure (kPa), \( \sigma \) refers to the normal stress of the dominant fissure (kPa), \( \phi \) means the (equivalent) angle of internal friction of the dominant fissure (°), and \( c \) refers to the (equivalent) cohesion of the dominant fissure (kPa).

Through an in-situ exploration, the authors found that the dominant fissures of unstable rocks are mostly the weak interlayers or joint planes with penetration or intermittent penetration. These lead to considerable difficulties in the calculation and selection of the shear strength parameters \( \phi \) and \( c \) of the dominant fissures. In the Specification, an equivalent calculation method (short for the Specification method) that integrates the intensity parameters of the dominant fissure was established based on the intensity parameters of the connecting and disconnecting portions of the dominant fissure and in accordance with the length weighting.

\[ c = \frac{(H_0 - e_0)c_1 + e_0c_0}{H_0} \]  

(2)

\[ \phi = \frac{(H_0 - e_0)\phi_1 + e_0\phi_0}{H_0} \]  

(3)

where \( c_0 \) and \( \phi_0 \) refer to the average cohesion (kPa) and average angle of internal friction (°) of the connecting portion of the dominant fissure, respectively; \( c_1 \) and \( \phi_1 \) refer to the cohesion (kPa) and angle of internal friction (°) of the disconnecting portion of the dominant fissure, respectively; and \( H_0 \) (m) and \( e_0 \) (m) refer to the vertical height of the unstable rock mass and the vertical height of the disconnecting portion of the dominant fissure, respectively. Tang and Chen [9,15] proposed the “penetration rate method” through model tests based on the penetration level of the dominant fissure of unstable rocks and safety grades for the prevention and control of unstable rocks. Their method is as follows:

\[ c = k_c \widetilde{c} [R_c] \]  

(4)

\[ \phi = k_\phi \widetilde{\phi} [R_c] \]  

(5)

where \([R_c]\) refers to the standard value of the uniaxial compressive strength for the complete rock (MPa); \([R_0]\) refers to the standard value of the uniaxial compressive strength for the rock mass (MPa); \( \widetilde{\phi} \) means the angle of internal friction for the complete rock of the unstable rock mass (°); \( \widetilde{c} \) means the average cohesion for the complete rock of the unstable rock mass (°); and \( k_c \) and \( k_\phi \) refer to the corrected coefficients of the strength parameters, whose values can be selected according to the following principles based on the safety grades of prevention projects:
Safety grade 1: \( k_c = 0.80, k_{\varphi} = 0.75 \);
Safety grade 2: \( k_c = 0.85, k_{\varphi} = 0.80 \);
Safety grade 3: \( k_c = 0.90, k_{\varphi} = 0.85 \).

Figure 2. Dominant fissure models for three types of unstable rocks: a) unstable sliding rock, b) unstable toppling rock, c) unstable falling rock.

The Specification’s method, which is used to determine shear strength parameters, lacks a theoretical basis, which leads to errors up to 40% when the method is applied to design engineering related to unstable rocks. Meanwhile, the penetration rate method remains in the theoretical research stage and has not been promoted in the field of geotechnical engineering. Therefore, the authors adopted the JRC-JCS model of dominant fissure, which is an empirical estimation method that is only applicable to the determination of the shear strength parameters of fissures in engineering practice [11]. Figure 2 shows schematic diagrams of dominant fissure models for unstable sliding rocks, unstable toppling rocks, and unstable falling rocks, and the dominant fissures conform to the JRC-JCS model [12,13].

\[
\tau = \sigma \tan \left( \varphi + JRC \cdot \lg \left( \frac{JCS}{\sigma} \right) \right) \tag{6}
\]

where \( JRC \) is the joint roughness coefficient and has a value of 0–20; \( JCS \) is the joint compressive strength of the dominant fissure (kPa); \( \varphi \) is the basic internal friction angle (°).

To make in-situ data accord with laboratory test results, Barton and Bandis (1980) proposed the following formulas of JRC and JCS for the size effect:

\[
JRC_n \approx JRC_0 \left( \frac{L_n}{L_0} \right)^{-0.02JRC_0} \tag{7}
\]

\[
JCS_n \approx JCS_0 \left( \frac{L_n}{L_0} \right)^{-0.03JRC_0} \tag{8}
\]

where \( L_0 \) is the length of the fissure for the laboratory sample, i.e., 100 mm; \( L_n \) refers to the length of the fissure at the site; \( JRC_0 \) and \( JCS_0 \) are the joint roughness coefficient and the surface strength, respectively, of the fissure under the fissure length of 100 mm of the laboratory sample; \( JRC_n \) and \( JCS_n \) are the joint roughness coefficient and the surface strength, respectively, of the fissure at the site.
3. Calculation methods for stability of unstable rocks based on JRC-JCS model

3.1 Load calculation of unstable rocks

The dead load of an unstable rock ($W$) is the product of the volume of the unstable rock and its unit weight.

$$W = V \gamma$$  \hspace{1cm} (9) 

where $W$ is the weight of the unstable rock (kN), $V$ is the volume of the unstable rock ($m^3$), and $\gamma$ is the unit weight of the unstable rock (kN/m$^3$).

The pore water pressure ($Q$) of the unstable rock is calculated as follows. With hydrostatic pressure as the main consideration, the column of water is one-third the vertical height of the dominant fissure under natural conditions, while it is two-thirds the vertical height under rainstorm conditions (Tang and Chen 2008); i.e.,

Under natural condition:

$$Q = \frac{1}{2} \xi \gamma_w \left( \frac{1}{3} e \right)^2 = \frac{1}{18} \xi \gamma_w e^2$$  \hspace{1cm} (10) 

Under rainstorm conditions:

$$Q = \frac{1}{2} \xi \gamma_w \left( \frac{2}{3} e \right)^2 = \frac{2}{5} \xi \gamma_w e^2$$  \hspace{1cm} (11) 

where $Q$ is the fissure hydrostatic pressure of the unstable rock (kN/m); $e$ is the vertical height of the cut-through fissure of the unstable rock (m); $\gamma_w$ is the unit weight of water (kN/m$^3$). As for an unstable rock with an opening of 0.2–5.0 cm, $\xi = k_1 a_0^2 + k_2 a_0 + k_3$, where $a_0$ means the connecting length of the dominant fissure; $\xi$ is the reduction coefficient of the fissure water pressure; and $k_1$, $k_2$, and $k_3$ are 0.57, −0.36, and 0.45, respectively [9,15].

The seismic load ($P$) of an unstable rock is calculated as follows. With horizontal seismic load as the main consideration, $P$ is the product between the dead weight of the unstable rock and the horizontal seismic coefficient, in which the point of load action is the center of gravity of the unstable rock; i.e.,

$$P = \mu W$$  \hspace{1cm} (12) 

where $P$ is the horizontal seismic load (kN) and $\mu$ is the coefficient of the horizontal seismic load. The coefficient of the vertical seismic load should also be considered for unstable falling rocks.

3.2 Load groups of unstable rocks

Group 1: dead weight + pore water pressure (under natural conditions);

Group 2: dead weight + pore water pressure (under rainstorm conditions);

Group 3: dead weight + pore water pressure (under natural conditions) + seismic load.

Groups 1 and 3 (seismic load = horizontal seismic load and vertical seismic load) should be considered for unstable falling rocks. Groups 2 and 3 (seismic load = horizontal seismic load) are required only for unstable toppling rocks. All three groups (seismic load = horizontal seismic load) should be applied to unstable sliding rocks.

3.3 Calculation methods

3.3.1 Unstable sliding rocks
The physical and mechanical models for unstable sliding rocks are shown in Figure 3. $e_i$ is the column height of full water of the unstable rock (m). Loads acting on the unstable rock are factorized into the normal component $N$ and tangential component $T$ along the orientation of the dominant fissure. The normal component $N$ is as follows:

$$N = W \cos \beta - P \sin \beta \quad (13)$$

The tangential component $T$ is as follows:

$$T = W \sin \beta + P \cos \beta \quad (14)$$

Supposing that $N$ and $T$ are distributed uniformly along the dominant fissure, the average normal stress $\sigma$ and average shear stress $\tau$ are as follows:

$$\sigma = \frac{N \sin \beta}{H} \quad (15)$$

$$\tau = \frac{T \sin \beta}{H} \quad (16)$$

The shear strength of the dominant fissure $\tau_f$ can be determined by substituting Eq. (15) into Eq. (6).

$$\tau_f = N \sin \beta \tan(\varphi + JRC \frac{JCS \cdot H}{N \sin \beta}) / H \quad (17)$$

The stability coefficient $F_i$ of the unstable sliding rock is as follows:

$$F_i = \frac{\tau_f}{\tau} = \frac{(W \cos \beta - P \sin \beta) \tan(\varphi + JRC \frac{JCS \cdot H}{(W \cos \beta - P \sin \beta) \sin \beta})}{W \sin \beta + P \cos \beta} \quad (18)$$

where $H$ is the height of the unstable rock (m) and $\beta$ is the angle of dominant fissure of the unstable rock ($^\circ$).

### 3.3.2 Unstable toppling rocks

The physical and mechanical models for unstable toppling rocks are shown in Figure 4. $e_i$ is the column height of full water of the unstable rock (m). Loads acting on the unstable rock are factorized into the normal component $N$ and tangential component $T$ along the orientation of the dominant fissure. The normal component $N$ is as follows:

$$N = W \cos \beta - P \sin \beta \quad (13)$$

The tangential component $T$ is as follows:

$$T = W \sin \beta + P \cos \beta \quad (14)$$

Supposing that $N$ and $T$ are distributed uniformly along the dominant fissure, the average normal stress $\sigma$ and average shear stress $\tau$ are as follows:

$$\sigma = \frac{N \sin \beta}{H} \quad (15)$$

$$\tau = \frac{T \sin \beta}{H} \quad (16)$$

The shear strength of the dominant fissure $\tau_f$ can be determined by substituting Eq. (15) into Eq. (6).

$$\tau_f = N \sin \beta \tan(\varphi + JRC \frac{JCS \cdot H}{N \sin \beta}) / H \quad (17)$$

The stability coefficient $F_i$ of the unstable sliding rock is as follows:

$$F_i = \frac{\tau_f}{\tau} = \frac{(W \cos \beta - P \sin \beta) \tan(\varphi + JRC \frac{JCS \cdot H}{(W \cos \beta - P \sin \beta) \sin \beta})}{W \sin \beta + P \cos \beta} \quad (18)$$

where $H$ is the height of the unstable rock (m) and $\beta$ is the angle of dominant fissure of the unstable rock ($^\circ$).
Figure 5. Mechanical models for two types of unstable toppling rocks.

The physical and mechanical models for unstable toppling rocks are shown in Figures 3 and 4. Unstable toppling rocks can be classified into two types according to whether or not their centers of gravity are closer to their dominant fissure or to their toppling points in the horizontal direction.

Type I: The center of gravity of the unstable rock is closer to the dominant fissure than to the toppling point in the horizontal direction. The upsetting moment $M_{\text{toppling}}$ of the toppling point is as follows:

$$M_{\text{toppling}} = Ph + Q\left(\frac{e_1}{3 \sin \beta} + \frac{H - e}{\sin \beta}\right)$$  \hspace{1cm} (19)

The resistance moment $M_{\text{resistance}}$ is as follows:

$$M_{\text{resistance}} = Wa + [\sigma_t]H - e + l\sigma_t$$  \hspace{1cm} (20)

The stability coefficient $F_i$ is as follows:

$$F_i = \frac{M_{\text{toppling}}}{M_{\text{resistance}}}$$  \hspace{1cm} (21)

With the substitution of Eq. (19) and Eq. (20) into Eq. (21),

$$F_i = \frac{Wa + [\sigma_t]H - e + l\sigma_t}{Ph + Q\left(\frac{e_1}{3 \sin \beta} + \frac{H - e}{\sin \beta}\right)}$$  \hspace{1cm} (22)

Type II: The center of gravity of the unstable rock is farther from the dominant fissure than to the toppling point in the horizontal direction. The upsetting moment $M_{\text{toppling}}$ of the toppling point is as follows:

$$M_{\text{toppling}} = Wa + Ph + Q\left(\frac{e_1}{3 \sin \beta} + \frac{H - e}{\sin \beta}\right)$$  \hspace{1cm} (23)

The resistance moment $M_{\text{resistance}}$ is as follows:

$$M_{\text{resistance}} = [\sigma_t]H - e + l\sigma_t$$  \hspace{1cm} (24)

The stability coefficient $F_i$ is as follows:

$$F_i = \frac{M_{\text{resistance}}}{M_{\text{toppling}}}$$  \hspace{1cm} (25)
With the substitution of Eq. (23) and Eq. (24) into Eq. (25),

\[
F = \frac{[\sigma_t]H - e + l\sigma_t}{Wa + Ph + Q\left(\frac{1}{3}\sin \beta + \frac{H - e}{\sin \beta}\right)}
\]

where \([\sigma_t]\) is the standard value of the tensile strength of the unstable rock (Pa), \(\sigma_t\) is the standard value of the tensile strength between the unstable rock and the foundation bed (kPa), \(a\) is the horizontal distance between the center of gravity of the unstable rock and the toppling point (m), and \(l\) is the distance between the tip of the dominant fissure at the bottom of the unstable rock and the toppling point (m).

### 3.3.3 Unstable falling rocks

![Physical and mechanical models for unstable falling rocks.](image)

The physical and mechanical models for unstable falling rocks are indicated in Figure 5. The methods used to determine the stability of such rocks are the same as those for unstable sliding rocks, and thus, the stability of unstable falling rocks can be calculated by referring to Section 3.3.1.

### 4. Stability calculations of unstable rocks in Taibai, Wanzhou District, Chongqing Municipality, China

Taibai, with two-stage cliffs developing, is in Wanzhou District, Three Gorges Reservoir. The unstable rock masses are composed of sandstones, which are medium–fine-grained gray or off-white arkose and feldspathic-quartz sandstone and cemented by mud and calcite. Sixty-one unstable rocks with a total volume of 24,562 m\(^3\) have developed on the cliffs on the southern slope of Taibai. They can be classified according to their possible collapse modes into 18 unstable sliding rocks, 11 unstable toppling rocks, and 26 unstable falling rocks. Geological survey results show that the tensile strength and unit weight of the rocks are 516 kPa and 25.6 kN/m\(^3\), respectively. The stability of five typical unstable rocks W12, W22, W59, W4, and W16 was calculated by the aforementioned methods in this paper. Meanwhile, their stability was assessed based on the Specification. The calculation results are shown in Table 1.

| No | Category | Geometrical characteristics (m) | Seismicic pressure (kPa) | Water pressure (MPa) | JRC | JCS (°) | Coefficient under state | Stability state |
|----|----------|-------------------------------|-------------------------|---------------------|-----|--------|------------------------|-----------------|

![Table 1. Stability calculation for unstable rocks in Taibai, Wanzhou District](image)
The calculation results in Table 1 indicate that the unstable sliding rock W12, unstable toppling rock W22, and unstable falling rock W4 are stable, while the unstable toppling rock W59 and unstable falling rock W16 are unstable. Therefore, the unstable rocks W59 and W16 need to be reinforced. Field monitoring from 2001 to 2013 indicates that the calculation results are relatively consistent with the actual situation, which further verifies the practicality and applicability of the above calculation methods for unstable rock stability based on the dominant fissure model.

5. Conclusions

A JRC-JCS model of dominant fissure for three types of unstable rocks, which is applicable to construction practice, was established in this study. Based on this model, calculation methods for three types of unstable rocks were obtained, and stability coefficient expressions of unstable rocks were determined. They can be used to evaluate the stability of unstable rocks in combination with the local standard, namely, Chongqing’s design specification for prevention and control projects of geological hazards (DB50/5029-2004). The stability of five typical unstable rocks in Wanzhou District, Chongqing Municipality, China, namely, W12, W22, W59, W4, and W16, was calculated by using the aforementioned methods. As indicated by the calculation results, the unstable sliding rock W12, unstable toppling rock W22, and unstable falling rock W4 are stable, while the unstable toppling rock W59 and unstable falling rock W16 are unstable and thus need reinforcing. As indicated by field monitoring from 2001 to 2013, the calculation results are relatively consistent with the actual situation, which further verifies the practicality and applicability of the above calculation methods, which are based on the dominant fissure model. These methods also offer a theoretical basis for the amendment of the aforementioned local standard in Chongqing in terms of calculation methods for unstable rock stability.
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