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New stability calculation method for rock slopes subject to flexural toppling failure

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ABSTRACT

Flexural toppling is one of the main failure modes of natural and manmade anti-dip layered rock slopes. Based on cantilever slab tensile theory, the failure mechanism of flexural toppling was analyzed. A toppling slope is divided into three parts: the stability, tensile and shear zones. By considering this failure mechanism, a new stability analysis method for slopes against flexural toppling failure is proposed using equilibrium theory. In addition, a sensitivity analysis is performed to investigate the locations of possible failure surfaces and zones as well as changes in the stability of anti-dip rock slopes under different conditions. The results show that the positions of the inter-column forces have almost no effect on the stability factor but affect the areas of the tensile zone and the shear zone. The angle of the most dangerous potential failure surface increases with increasing dip angle and slope height, whereas the stability factor is negatively correlated with the dip angle and slope height but is positively correlated with the layer thickness. The failure mode is essentially flexural toppling when the layer thickness is small, but the failure mode gradually transitions to shear-sliding with increasing layer thickness. Finally, a real case study is analyzed using this method, and the calculated results are consistent with the actual conditions.

1. Introduction

Toppling failures occur in rock masses containing a set of discontinuities that strike nearly parallel to the slope and dip into the slope, and these failures have been observed in both natural and manmade slopes. The term "toppling failure", as applied to rock slopes, was first suggested by Ashby.[1] Goodman and Bray[2] summarized toppling failures as having three basic modes: flexural, block and block-flexure toppling.

Based on the limit equilibrium method, a "step-by-step" approach was proposed by Goodman and Bray[2] for the analysis of block toppling. This approach was modified by Cruden[3] and later improved by Aydan et al.[4] and Kliche.[5] Zanbak[6] constructed a set of diagrams to calculate the required support forces. Following Goodman and Bray's solution, a general analytical solution that assumes that the blocks have an infinitesimal thickness was developed.[7–9] Aydan and Kawamoto[10] first presented a theoretical method based on the limit equilibrium method and applied the bending theory of cantilever beams to analyze flexural toppling failures. Based on the principle of compatibility equations, Amini et al.[11] presented a new method for analyzing and computing the safety factor for flexural toppling failure. Amini et al.[12] presented a new analytical approach for block-flexure toppling and developed a computer code for stability analysis and assessment. Tatone BSA and Grasselli[13] developed a Monte Carlo simulation procedure for the probabilistic analysis of block toppling and described its implementation in a spreadsheet-based program (ROCKTOPPLE).

Physical and numerical modeling is also used to understand the mechanisms underlying toppling failures as well as the potential for stabilizing toppling failures. Physical modeling methods involving base friction models and tilt tables were popular in the 1970s and early 1980s. Ashby[1] utilized base friction models and tilt tables to study the slipping and toppling mechanisms acting on jointed rock slopes. Bray and Goodman[14] carried out base friction tests and analyzed the corresponding theoretical and experimental results. Recent physical modeling of rock toppling has involved centrifuge modeling. Adhikary et al.[15,16] performed a series of centrifuge experiments to investigate the mechanism of flexural toppling failure and observed the following: (1) the basal failure plane extended from the toe of the slope and was oriented at an angle of 12–20 degrees upward from the normal to the layers; and (2) the two main failure mechanisms of flexural toppling, instantaneous and progressive failure, were controlled by the magnitude of the joint friction angle. Zhang et al.[17] observed in centrifuge tests that the failure mode did not follow a straight failure plane, as was proposed by Goodman and Bray.

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Because physical modeling requires considerable time and large monetary costs, numerical simulation, which is an effective method, is commonly used to investigate the toppling failure mechanisms of rock slopes. Since the 1970s, many numerical techniques have been developed and successfully applied to the modeling of toppling failures. These techniques include the distinct element method (DEM)[18–20] and the universal distinct element code (UDEC).[21–23] Adhikary et al.[24–26] developed a finite element model that was based on the Cosserat theory to investigate the mechanisms of flexural toppling failure. Alzo'ubi et al.[27] used the UDEC damage model (UDEC-DM), which is a numerical modeling methodology based on a discrete element framework, to investigate two centrifuge tests carried out by Adhikary et al.[25] and Zhang et al.[17] to examine the toppling process.

As described above, many studies have been performed on toppling failures and have resulted in significant achievements. Block toppling and flexural toppling are two distinct types of toppling failure mode, and the stability analysis method proposed by Goodman and Bray[2] on block toppling is not appropriate for flexural toppling. Reference[10–12,15,16] have developed a new technique, which consider the stratified rock slopes as a series of cantilever slabs, to calculate the stability of flexural toppling. However, the mechanism of flexural toppling has not been clarified, especially the location of the failure surface. Therefore, this paper investigates the mechanisms underlying flexural toppling failure based on cantilever slab theory and the limit equilibrium method, and presents a new stability calculation method for flexural toppling failure.

2. The ultimate tensile length of a cantilever slab

Flexural toppling of stratified rock slopes can be considered to involve a series of cantilever slabs and the interacting forces between the adjacent slabs; a geomechanical model of this is shown in Fig. 1. Under the influence of external forces, these cantilever slabs deform and even fail via tensile or shear failure modes. According to the stability and failure mechanisms, a rock slope subject to flexural toppling is divided into three parts from the crest to the toe: the stability zone, the tensile zone and the shear zone. Furthermore, the tensile zone forms earlier than the shear zone. Employing the column theory from the theory of elasticity, the absolute value of the minimum normal stress σ_x at the base of a column with unit thickness is given as follows:

$$\sigma_x = \frac{M}{I}y - \frac{N}{A} \tag{1}$$

where y denotes the thickness of the column (m), N denotes the normal force (kN), M denotes the moment (kN m), I denotes the inertia modulus (m^4), and A denotes the cross-sectional area of the column (m^2).

For a cantilever slab with a base inclination of β that is acted on by gravity, Eq. (1) takes the following explicit form:

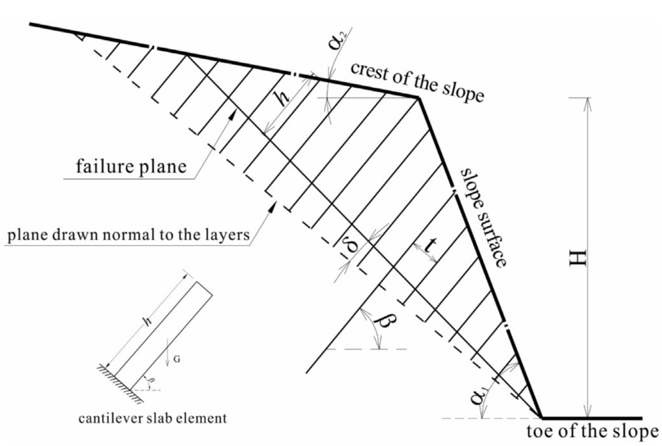

Fig. 1. Geomechanical model for flexural toppling failure of rock slopes.

$$\sigma_x = \frac{3\gamma h^2 \cos\beta}{t} - \gamma h \sin\beta \tag{2}$$

where γ denotes the unit weight of the column (kN), h denotes the column height (m), and t denotes the column thickness. In the tensile zone, every cantilever experiences tensile damage at the base of the column. Considering the stability factor k, the ultimate condition is expressed as follows:

$$\sigma_x = \frac{\sigma_t}{k} \tag{3}$$

where σ_t denotes the tensile strength of the column. By combining Eqs. (2) and (3), we obtain the limiting tensile length of the column $h_{\lim}$ under gravity:

$$h_{\lim} = \frac{t}{6}\tan\beta + \frac{1}{6}\sqrt{(t\tan\beta)^2 + 12\frac{[\sigma_t]t}{\gamma k\cos\beta}} \tag{4}$$

3. Failure mechanism of flexural toppling

Assuming that any one column of the toppling rock slope is first damaged by tension, its length should be longer than the limiting tensile length calculated using Eq. (4). This column then produces a flexural deformation larger than that of the upper column, which fails further due to tension if this column is also longer than the limiting tensile length. In this way, all columns with lengths that are not less than the limiting tensile length above the first column that was damaged by tension also fail due to tension. Then, the column above the top column that is damaged by tension deforms and separates from the overlying and underlying columns. There are no interactive forces between this column and the two adjacent columns, which is also true for all columns above the column with the limiting tensile length. Therefore, this column does not fail due to gravity only. In conclusion, the column with the limiting tensile length above the tensile zone is the lower boundary of the stability zone.

As shown above, all the columns experiencing tensile failure above the first column that failed in tension make up a retrogressive failure. The column immediately below the first column that experienced tensile failure due to gravity and the driving force is very likely to be damaged by tension; that is, the size of the tensile zone gradually increases until the normal stress is less than the tensile strength of the base of the column. This tensile region below the first column that experienced tensile failure is a push failure. Therefore, the tensile zone is divided into two subzones. The column length at the lower boundary of the tensile zone is denoted by h_x. The region between the lower boundary of the tensile zone and the toe of the slope is the shear zone. Based on this discussion, the failure mode of rock slopes that experience flexural toppling is described in Fig. 2. Furthermore, the size of each zone along the failure surface is expressed as shown in Fig. 2 and calculated using the following equations.

First, the length of the failure surface behind the base of the longest column, l_{down}, and the length of the failure surface ahead of the base of the longest column, l_{up}, are calculated as follows:

$$\begin{aligned} l_{down} &= -y_{\max}\tan(\alpha_1 + \beta) \\ l_{up} &= y_{\max}\tan(\alpha_2 + \beta) \end{aligned} \tag{5}$$

where y_{max} indicates the length of the column at the top of the rock slope, which can be obtained as follows:

$$y_{\max} = -\frac{\cos(\alpha_1 + \beta)}{\sin\alpha_1}H \tag{6}$$

Based on the geometric relationships, we obtain the following equations:

$$h_{\lim} = y_{\lim} - \Delta y_{\lim} = \frac{l_{up} + x_{\lim}}{l_{up}}y_{\max} - (l_{down} - x_{\lim})\tan\delta \tag{7}$$