Composite collapse mechanism of an anti-dip rock slope

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Abstract. Collapse is a common instability mode of anti-dip rock slopes. This type of failure involves a complex geo-mechanical process. This work investigated the collapse mechanism of an anti-dip rock slope in a granite mine in Shanxi Province, China. A detailed field survey was carried out to analyze the inducing factors and failure process of collapse. Then, discrete element numerical simulations were conducted by the Universal Distinct Element Code (UDEC) to determine the microscopic damage and macroscopic failure of collapse. Through these works, the composite collapse failure mechanism of the anti-dip rock slope was investigated comprehensively. Results show that the structural planes perpendicular to the slope surface and the steep anti-dip structural planes are the prerequisites of collapse. Strength softening of the structural planes caused by rainfall is the triggering factor of collapse. The collapse of such a rock slope can be considered as a composite failure composed of shear sliding, block toppling, and flexural toppling. In addition, the numerical results indicate that the composite failure of shear sliding and block toppling occurs at the toe of and the middle of the slope, followed by flexural toppling failure at the rear edge of the slope. Therefore, the monitoring and treatment of such type of collapses should be concentrated on the composite failure zone where shear sliding and block toppling occur. The high consistency between the numerical results and field observations shows the feasibility and rationality of using UDEC to study such collapse from the microscopic damage to macroscopic failure. The results enrich the type of collapse of anti-dip rock slopes and provide a scientific reference to the design treatment for such slopes.

Keywords: anti-dip rock slope, slope stability, collapse mechanism, microscopic damage, numerical simulation

1. Introduction

On August 23, 2017, the anti-dip rock slope on the south side of the fractured zone of the mining slope in Hunyuan County, Shanxi Province, suffered collapse failure. The main body of the collapse was located in the lower area of 1950 m elevation. The collapsed material accumulated on the construction platform of 1900 m elevation. Fortunately, construction personnel evacuated without causing casualties, but some equipment and materials were smashed.

Collapse is a typical failure mode of anti-dip rock slope, and it exists in a large number of engineering slopes, such as in the construction of water conservancy and hydropower, highways, and open-pit mines [1-7]. For instance, the four controlling factors of the Honglianchi Iron Mine collapse in Hefeng,
Hubei are layered block-fractured rock mass structure, open-pit mining, underground mining, and rainfall [8]. The Jiweishan collapse in Chongqing was also taken as an example to carry out the cause and mechanism of mountain collapse under action of the gravity, karst, and mining activities [9]. Using various collapse examples, some scholars studied the impact of internal and external factors, such as rock mass strength, rock layer dip angle, weathering, and rainfall, on collapse through field investigations and numerical simulations. The critical dip angles of the rock layer for anacinal and cataclinal slopes were obtained, and the collapse mechanisms, such as sliding, toppling, cantilever cracking, and dislocation, were investigated comprehensively [10-11]. Moreover, the stability of individual sliding, toppling, and falling collapses was studied [12-16]. The above studies considered factors such as lithological conditions, stratum structure, weathering, rainfall, and excavation. The collapse process and deformation mechanism were investigated through geological analysis and simple numerical evolution. However, most of the studies focused on collapse with a single failure mode.

Given the brittleness of rocks and the irregularity of joints, anti-dip rock slopes may also be subjected to the sliding–toppling composite collapse failure. Alejano et al. [4] described a complex failure occurring in a slope comprised of limestone rock mass in the upper part and a claystone-sandstone sedimentary formation in the lower part. This failure mechanism was interpreted as complex toppling combining toppling and circular failure. Mohtarami et al. and Amini et al. [5-6] introduced a composite failure composed of the upper circular–lower toppling failure through model test and numerical simulation. Transverse joints or cracks in an anti-dip slope do not penetrate from the toe to the crest of the slope, and special composite slope failure may occur. The rock layer at the slope toe may have shear sliding failure along the transverse joints because of the relatively small depth-to-width ratio of the rock block. In addition, block toppling failure may occur in the rock layer cut by the lateral joints and located in the middle of the slope. Meanwhile, flexural toppling failure may occur in the upper rock layer because it is not cut by lateral joints. Such slope failure may be referred to as a composite failure composed of shear sliding, block toppling, and flexural toppling.

The present study investigated the special composite collapse mechanism of an anti-dip rock slope in a granite mine located in Hunyuan County. First, a detailed field survey and geological analysis were conducted in the collapsed area. Then, the microscopic damage and macroscopic failure were simulated by the Universal Distinct Element Code (UDEC). Through these investigations, the inducing factors and failure mechanism of collapse were obtained. Finally, monitoring and treatment approaches for such collapses were proposed.

2. Geological setting

2.1. Geology of the study area
The granite mine in Hunyuan County is situated in a moderately mountainous area. Thus, they encounter steep terrain and cut deep into the valley. The mountain is mostly composed of ancient gneiss that has suffered strong weathering and denudation. After different stages of long-term tectonic movement, geologic bodies that have formed show signs of migmatization. They are mixed with a large number of ancient intrusive rock masses, forming a large-scale granite deposit [17].

The main lithology of the slope in the study area is biotite gneiss. The thin overlying soil layer is composed of residual deposits and diluvium.

2.2. Field survey
The slope in the study area is low in the south and high in the north. A fractured zone running from top to bottom in the middle of the slope generates a large free surface. It subdivides the south and north sections of the whole slope into anti-inclined and cataclinal slopes, as shown in Figure 1. The depth of the fractured zone is about 5 m, and the rock masses on both sides are well developed and highly weathered. The structure of these rock masses shows the characteristics of slab structure.
Figure 1. Overview of slope geological structure (1 month before collapse): (a) slope overview, and (b) slope rock mass structure.

The orientation of the anti-dip slope is $338^\circ \angle 65^\circ$ (i.e., the dip angle and dip direction of the slope are $65^\circ$ and $338^\circ$, respectively). Three joint sets (J1–J3) were found in the slope region from the site investigation (Figure 1b). The first joint set, J1, has an orientation of $67^\circ \angle 73^\circ$. It has high persistence with spacing of about 30 cm. The second joint set, J2, has an orientation of $136^\circ \angle 81^\circ$. It shows medium persistence with spacing of about 50 cm. Finally, the third joint set, J3, has an orientation of $172^\circ \angle 25^\circ-45^\circ$ and spacing of about 65 cm. The joint is generally partly open and has low-to-very low persistence. A pole plot of the discontinuous structural planes and slope surface is shown in Figure 2. In addition, the toe of the anti-dip slope has a bottom fissure zone with a depth of 0.2 m and a length of 13 m, dipping at an angle of $30^\circ$.

To simplify the analysis, the slope geological profile only considers the steeply inclined structural plane J2 and the bottom fissure zone because of the poor persistence of the gently inclined structural plane J3. Thus, the geological profile of the anti-dip slope is obtained, as shown in Figure 3, based on the slope geological structure and pole plot of discontinuous structural planes.

Figure 2 Pole plot of discontinuous structural planes.

Figure 3 Geological profile of the anti-dip slope.

2.3. Hydrogeology

The hydrogeological condition of the slope is relatively simple. The groundwater present is classed as phreatic bedrock fissure water because it is deeply buried, and surface water is not well developed. The fissure water is mainly supplied by meteoric water penetrating along the interbedded fissures, such as joint sets, faults, or weak zones. In addition, a snow cover period of nearly 5 months occurs every year. During this period, the freeze–thaw action of snow accelerates the weathering of the
surface rocks and causes the weathering and rupturing of rock masses near the surface, thus adversely affecting the stability of the slope.

3. Analysis of collapse mechanism
During collapse failure, slope anchor reinforcement and anchor cable treatment were carried out. The construction platform has 1900 m elevation. Scaffolding from 1900 m to 1950 m was planned for overall slope treatment and local dangerous rock mass reinforcement, as shown in Figure 4.

3.1. Characteristics of collapse
The main collapse is located in the range of 5–15 m below the 1950 m elevation platform. The front and middle areas of the collapse suffer the composite failure of sliding–block toppling along the fissure zone, while flexural toppling deformation occurs in the rear rock layer. Furthermore, the depth of the surface collapse body is about 0.5 m, and the deepest collapse body is about 4 m, forming an L-shaped collapse cavity, as shown in Figure 5.

The collapse materials accumulated on the construction platform at 1900 m elevation. The accumulation volume was nearly 120 m³, mainly composed of block and crushed stones. The volume of individual block stones was about 6 m³ (2 m×2 m×1.5 m). Gneissic schistosity can be observed in the collapse source area of the anti-dip slope, and a few residual dangerous rock masses and more soil appeared in the collapse cavity.

3.2. Terrain conditions of collapse
The open-pit mining method was adopted to mine granite rocks, thus generating relatively high and steep slopes. In addition, excavation along the structural plane J1 formed a lateral free face, and this structural plane J1 relieved the lateral constraint of the collapse. At the same time, the fracture zone in the middle of the slope was frequently subjected to washing by rainwater and freezing–thawing by snow, forming a front free face (i.e., slope surface of the anti-dip slope). These two free faces significantly influenced the slope deformation and stress adjustment, and they provided favorable terrain conditions for collapse, as shown in Figure 6.

Figure 4 Overview of collapse area (2 days before collapse).

Figure 5 Failure of anti-dip slope collapse.
3.3. Geological conditions of collapse

As mentioned above, three sets of structural planes were developed in the collapsed area, and a fissure zone was developed at the bottom. The bottom fissure zone penetrated the steeply inclined structural plane J2 to produce the potentially unstable rock blocks. In the collapse source area of the anti-dip slope, gneissic schistosity developed, which is in an anti-inclined form and belongs to the typical easily collapsed structure. Moreover, soil adherence onto the surface of the structural plane in the collapse source area weakened the mechanical strength of the structural plane. The block-layered rock mass structure and the low-strength anti-inclined structural plane provided favorable geological conditions for the collapse.

3.4. Rainfall

The triggering factor of collapse was rainfall. Before the collapse, the study area experienced two long periods of continuous rainfall. The first one was on August 20. The rainfall lasted for nearly 15 h with a rainfall of about 15 mm. The next day, the fracture zone and surrounding soil were wet with a relatively large water content. The bottom fissure zone extended upward and nearly penetrated. The rock mass at the front area of the collapse was stable, and the opening of the structure plane at the rear edge (J2) increased, as shown in Figure 4.

The second rainfall was about 12 mm from 17:00 on August 22 to 4:00 in the morning on August 23, with no obvious water flow. Rainwater infiltrated through the cracks and fissures in the rock mass to erode the internal rock masses and structural planes and reduce their strength parameters. It intensified the toppling deformation of the rock mass and expanded continuously the cracks and fissures. Subsequently, collapse occurred facing the front free face, as shown in Figure 5.

3.5. Collapse process

On the basis of the above analysis, the internal controlling factors and external triggering factors of the composite collapse were obtained. The terrain and geology of the anti-dip slope were the controlling factors of collapse, and the rainfall was the triggering factor. Moreover, the front free face formed by the fracture zone provided the space for the rock mass to slide or topple. In consideration of the internal and external factors of actual collapse, further detailed analysis was carried out on the site collapse of the anti-dip slope, as shown in Figure 7.

Figure 7 shows the formation mechanism of the collapse. First, the slope rock mass deformed under the action of the cyclic blasting and excavation disturbance. It generated and expanded the cracks and fissures in the rock mass, providing channels for rainfall infiltration (Figure 7a). Two long-term continuous rainfalls gradually infiltrated along these fissures to the fracture zone, softening the rock masses and structural planes.

Due to the high depth-to-width ratio of the rock mass on the middle position of slope, block toppling deformation occurred under the action of weight and interlayer thrust in this area, pushing the rock mass to deform at the slope toe. Subsequently, the sliding–block toppling composite failure occurred
(Figure 7b). After losing the supporting resistance of the front rock mass, the rock layers at the rear area of slope formed cantilever beams. Then, the bend deformation of cantilever beams occurred, producing the flexural toppling failure (Figure 7b). In the end, an L-shaped failure formed after the slope completely collapsed (Figure 7c).

4. Evolution of collapse mechanism
In the previous section, the mechanism of collapse was analyzed through the geological conditions of the slope. In this section, UDEC numerical simulation and evolution were carried out to investigate the collapse process. The dynamic and static water pressures in the rock mass fissures were ignored and only the weakening effect of rainfall on the strength parameters of the rock masses and structural planes was considered because of the minimal accumulation of continuous rainfall during the collapse and light water flow in the slope body.

4.1. Numerical model and calculation parameters
In this study, a Mohr–Coulomb model was used to describe the mechanical behavior of the rock mass, and a Coulomb slip model was used to describe the structural plane between them. The numerical simulation of the collapse was conducted under natural and rainfall. Furthermore, the calculation parameters of these two conditions were obtained by combining the results of the laboratory tests, engineering analogy, and parameter back analysis (Table 1) [7, 18]. Some experimental samples of the rock masses and structure planes are shown in Figure 8.

Figure 8 Experimental samples of (a) unconfined and confined compression test, (b) Brazilian disk splitting test, and (c) direct shear test of the structural plane.

A numerical model for the anti-dip slope was established based on its geological profile (Figure 3), as shown in Figure 9. The model has a width of 35 m, a height of 25 m, and a slope angle of 65°. The
steeply inclined structural surface dips in the model at an angle of 81° and spacing of 1 m. In addition, the bottom fissure zone has a depth of 0.2 m and a length of 13 m, dipping at an angle of 30°.

Table 1. Material parameters using in numerical simulation.

| Type            | \( \rho / \text{kg} \cdot \text{m}^{-3} \) | \( E / \text{GPa} \) | \( \mu \) | \( \sigma_y / \text{MPa} \) | \( c / \text{MPa} \) | \( \varphi / (^\circ) \) |
|-----------------|-------------------------------------------|----------------------|---------|-----------------------------|-----------------|---------|
| Rock mass       | 2700                                      | 45\(^a\) (42\(^b\)) | 0.21\(^a\) (0.23\(^b\)) | 7.5\(^a\) (6.6\(^b\)) | 1.0\(^a\) (0.3\(^b\)) | 45\(^a\) (40\(^b\)) |
| Structural plane| /                                         | /                    | /       | /                           | /               | /       |
| Fissure zone    | /                                         | /                    | /       | /                           | /               | /       |

Note: \(^a\) Applicable to natural conditions, \(^b\) Applicable to rainfall conditions.

4.2. Numerical simulation of natural conditions

The numerical calculation converges under natural conditions, with a total of 57,486 steps. The final slope calculation results are shown in Figure 10. Figure 10a shows that the displacement and deformation of the slope are relatively small, indicating that the slope is stable in its natural state. Figure 10 also shows that the displacement of the rock mass at the slope toe is minimal under this natural condition, and the fissure zone at the bottom is closed without sliding. Under the action of its own weight, the horizontal displacement of the slope crest is larger than that of the slope toe, which causes a tendency to topple outward, but the amount of toppling displacement is small. The slope displacement increases linearly with the increase in slope height, with the maximum displacement of 1.0\( \times 10^{-3} \) m near the slope shoulder, and the slope is stable.

![Figure 9 UDEC numerical model of the anti-dip slope.](image)

Figure 9 UDEC numerical model of the anti-dip slope.

![Figure 10 Numerical simulation results of the slope under natural conditions: (a) horizontal displacement contour map and (b) displacement vector diagram.](image)

Figure 10 Numerical simulation results of the slope under natural conditions: (a) horizontal displacement contour map and (b) displacement vector diagram.

4.3. Numerical simulation of rainfall conditions

The weakening effect of rainfall reduces the strength parameters of slope rock masses and structural planes. Under the condition of saturated strength parameters, the numerical calculation does not
converge, indicating that the slope is unstable under rainfall conditions. Figure 11 displays the numerical simulation results of the slope calculation at 500,000 steps under rainfall conditions.

**Figure 11** Numerical simulation results of slope under rainfall conditions: (a) displacement contour map and (b) displacement vector diagram.

Figure 11a shows that the displacement deformation of the slope is relatively large under rainfall conditions, and the slope displacement no longer linearly increases with slope height but presents obvious zoning characteristics that can be subdivided into sliding zone, block toppling zone, and flexural toppling zone.

In specific, the horizontal displacement of the rock masses in the sliding zone is the same from top to bottom, indicating that the sliding failure mainly occurs along the fracture zone. However, the horizontal displacement at the top of the block masses in the block toppling zone is greater than that at the bottom, indicating that block toppling failure mainly occurs along the lower end of the rock mass bottom. Meanwhile, the rock layers in the flexural toppling zone are in a state of bending deformation because of the absence of a penetrating fissure zone at their bottom. The maximum displacement of the rock mass is 2.4×10^{-2} m, which is located at the top of the rear edge of the block toppling zone. Therefore, the collapse is mainly caused by toppling failure of the rock mass in the block toppling zone, and then the upper transfer force pushes the front rock mass and triggers the sliding failure of the front rock mass.

Furthermore, the failure characteristics of these three failure zones can also be analyzed basing from the characteristics of the slope displacement vector (Figure 11b). The sliding zone is concentrated at the foot of the slope. The displacement directions of these rock blocks are parallel to the bottom fissure surface downward, and the displacement of each height is approximately equal. It shows the failure characteristic of the overall downward shear sliding. The block toppling zone is located above the sliding zone, the displacement directions of these rock blocks are approximately horizontal outward, the top displacement of the rock block is large, and the bottom displacement is small, and the displacement vector is approximately perpendicular to the anti-dip structure plane. It shows a failure characteristic of outward rotation and toppling along the lower end of the bottom fissure surface. Moreover, the flexural toppling zone is located above the block toppling zone. The local tensile yield failures of these rock layers occur at the back of the bottom of the rock block (the small purple circle indicates the tensile failure plastic zone), showing the characteristic of bending and toppling outward. Given that the flexural tensile stresses of these rock blocks have not reached their tensile strengths, the rock blocks in flexural toppling zone are in a state of toppling deformation and do not exhibit toppling failure. Therefore, under rainfall conditions, the slope mainly suffers from the sliding–block toppling failure of the front and middle positions of the rock mass, accompanied by the flexural toppling deformation of the rear edge rock mass.
In addition, the instability mechanism of collapse can be further verified from the enlarged view of tension crack in Figure 11b. In specific, the failure characteristics of the rock block in the sliding zone can be observed from the enlarged view (A1) of the crack of the last rock block in that zone: the bottom of the rock block is in pressure-tight contact with the fracture zone (failure surface), and shear dislocation occurs, resulting in compression shear failure. The obvious tension crack shows the shape of narrow top and wide bottom. This phenomenon is mainly due to the rotation and toppling of the back rock block along the lower end of the bottom. The deformation of the upper part is larger than that of the lower part, and the upper part pushes the front rock block, resulting in the formation of “tensile crack” at the lower part of the rock block at the rear edge of the sliding zone, which can be used as the interface between the sliding zone and the block toppling zone.

From the enlarged view (A2) of the tension crack of the last rock block in the block toppling zone, the rock blocks in this zone produce two tension cracks at the bottom and at the back. The tension crack at the bottom presents a shape of front narrow and back wide along the failure surface, and another tension crack at the back presents a shape of top wide and bottom narrow along the height of the rock block. This phenomenon shows obvious block toppling characteristics. Therefore, the sliding–block toppling failure of the rock blocks in the front and middle positions of the slope is further confirmed from the local enlarged view of rock failure.

Figure 11 shows a composite collapse failure mechanism of sliding, block toppling, and flexural toppling in UDEC simulation. These failure characteristics of the rock blocks in three zones simulated by the numerical method are consistent with the site collapse characteristics of the anti-dip slope described in Section 3, both of which are the sliding–block toppling failure in the front and middle positions of the slope, followed by the flexural toppling deformation of the rear edge rock masses. It indicates that the numerical simulation results are consistent with the actual field survey.

4.4. Numerical simulation of multi-scale collapse failure

In the preceding section, a detailed analysis of the displacement, plastic zone, and local cracks of the slope at 500,000 steps under rainfall conditions was carried out. This section further analyzes the microscopic damage simulation before 500,000 steps and the macroscopic failure simulation after 500,000 steps.

A Coulomb slip model is used for block contact in UDEC. When the stress on the contact surface exceeds its tensile strength or shear strength, the cracks are gradually generated and expanded, resulting in the tensile or shear failure. Therefore, the cumulative contact length of the tension and shear failure and the total contact length in the collapse area are recorded by the FISH function, and the tension and shear damage degrees are defined as the ratio of the cumulative contact length of tension and shear failure to the total contact length [19-20].

Figure 12 presents the simulation results of microscopic damage in the collapse area, in which magenta and light red respectively represent the shear and tensile cracks. Figure 12 shows that under rainfall conditions, the structural planes and bottom fissures in the collapse area become damaged through shearing and slipping (Figure 12a). Tension cracks gradually appear with prolonged calculation time (Figure 12b–12e). This result indicates that the steeply inclined structural plane and the bottom fissures undergo shear and slip first under rainfall conditions, and then the joints begin to appear tension failure with further deformation.

As shown in Figure 12e, almost all steeply inclined structural planes show tension cracks, but the top contact positions still maintain the state of slipping; the back of the bottom fissure zone shows tension cracks, but the front still maintains the state of slipping. This finding is consistent with the macro-fracture characteristics of the slope at 500,000 steps in Figure 11b, with the front and middle positions of the rock block sliding–block toppling failure, accompanied by the flexural toppling deformation characteristics of the rear edge rock block.

Figure 13 shows the cumulative damage degree of steeply inclined structural planes and bottom fissures in the collapse area for further quantitative analysis of the microscopic damage. Figure 13a shows that all steeply inclined structural planes in the collapse area are damaged, with a total damage
degree of 1.00. The initial damage stage of joints is mainly shearing and slipping. As the calculation time increases, the tensile damage increases gradually with a final tensile damage degree of 0.89. In addition, the final shear damage degree is 0.11, which mainly exists at the root of the rock mass in the flexural toppling zone. Figure 13b shows that the bottom fissures are extensively damaged, with a total damage degree of 0.94. The initial damage stage is also dominated by shearing and slipping, and then the tensile damage increases gradually with a final tensile damage degree of 0.55. The final shear damage degree is 0.39, which mainly exists in the bottom fissure position corresponding to the sliding zone.

Figure 12 Development of microscopic damage of collapse at (a) 100 steps, (b) 1000 steps, (c) 10,000 steps, (d) 100,000 steps, and (e) 500,000 steps.

Figure 13 Accumulation curve of damage degree in the collapse area: (a) steeply inclined structural planes, and (b) bottom fissures.

Figure 14 shows the collapse process of the anti-dip slope under rainfall conditions in the numerical simulation. At 500,000 steps, the slope is in a critical instability state, and as mentioned above, sliding–block toppling failure occurs (Figure 14a). With prolonged calculation time, the deformation of the rock blocks at the front and middle positions of the slope is further intensified. As a result, the rock blocks at the rear edge are in cantilever state, leading to flexural toppling deformation. However, the rock blocks in the flexural toppling zone are in the bending deformation state because their tensile stress is lower than the tensile strength, which is insufficient for flexural toppling failure (Figures 14b and 14c). At 5 million steps, the slope is completely collapsed, forming an L-shape of failure (Figure 14d).

The collapse failure shape shown in Figure 14 simulated by the UDEC is highly consistent with the shape of field survey shown in Figure 5, and both methods are mutually verified. In addition, Figures 14 and 7 reveal the composite failure mechanism of sliding, block toppling, and flexural toppling from the numerical simulation and site investigation.
Figure 14 Development of macroscopic failure of collapse at (a) 500,000 steps, (b) 1,000,000 steps, (c) 3,000,000 steps, and (d) 5,000,000 steps.

5. Conclusions
The inducing factors and collapse mechanism of an anti-dip rock slope in a granite mine in Shanxi Province are investigated through geological analysis and discrete element numerical simulations. The key results of the study are summarized as follows:
1. Structural planes perpendicular to the slope surface (J1) and steep anti-dip structural planes (J2) are the prerequisites of collapse, and stiffness softening of the structural planes caused by rainfall is the triggering factor of collapse.
2. Anti-dip rock slopes with nonpenetrating transverse joints or cracks at the slope toe are prone to suffer the composite collapse failure, which includes the three zoning characteristics of sliding, block toppling, and flexural toppling zones.
3. The process of composite collapse is that the failure of shear sliding and block toppling first occurs at the toe of and the middle of the slope, followed by flexural toppling failure at the rear edge of the slope.
4. Monitoring and treatment of such collapses should be concentrated on the failure zone where shear sliding and block toppling occur.
5. The high consistency of the numerical results and field observations shows the feasibility and rationality of using UDEC to study such collapse from the microscopic damage to macroscopic failure.

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