Modeling or rock slope stability and rockburst by the rock failure process analysis (RFPA) method

Chun'an Tang*, Shibin Tang

Institute for Rock Instability and Seismicity Research, Dalian University of Technology, Dalian, 116024, China

Abstract: Brittle failure of rock is a classical rock mechanics problem. Rock failure not only involves initiation and propagation of single crack, but also is a complex problem associated with initiation, propagation and coalescence of many cracks. As the most important feature of rock material properties is the heterogeneity, the Weibull statistical distribution is employed in the rock failure process analysis (RFPA) method to describe the heterogeneity in rock properties. In this paper, the applications of the RFPA method in geotechnical engineering and rockburst modeling are introduced with emphasis, which can provide some references for relevant researches.

Key words: rock slopes and foundations; stability analysis; rock burst; numerical modeling

1 Introduction

Rock failure is induced by damage evolution of initial defects. The mechanical behaviors of rocks are determined by the internal mesoscopic structures, the mesoscopic damage and its evolution, and development of cracks. Krajcinovic [1] indicated that the phenomenological model could not effectively deal with the mesoscopic damage process, which can only be overcome by using the mesoscopic mechanical models. Dougill et al. [2–7] and other researchers used the damage mechanics to study the failure of rock from different points of view. Furthermore, many research established the damage mechanics based method to study the behaviors of rocks at mesoscopic level. The relevant results are further extended to general brittle damage problems, and the researches on mesoscopic damage mechanics are continuously enriched.

Since 1995, the authors and their research group have been committed to study the rock failure process analysis (RFPA). Based on the statistical distribution method in rock material properties, mesoscopic damage theory and numerical computation method, a RFPA system was developed and applied to a great number of fundamental researches. Currently, the RFPA system has been widely used in geotechnical engineering. In this paper, engineering applications of the RFPA system in geotechnical engineering and rockburst research are introduced.

2 Stability analysis of the left slope of Dagangshan hydropower station

2.1 Introduction to geology and calculation model

The case study herein is the right bank slope of Dagangshan hydropower station, which is located at the Dadu River in Sichuan Province, southwest China. The project is one of the large-scale hydroelectric constructions that are currently exploited along the mainstream of the Dadu River. The right bank slope of the project has complicated geological conditions, such as faults, dykes, unloading cracks zones and joints with cracks development. In particular, faults X316-1 and f231 are the most significant factors that influence the stability of the rock slope. Weathering and unloading of rock mass inside the slope are also very serious. Natural slope surface orients N25°–35°E and

Corresponding author. Tel: +86-13840899558; E-mail: catang@mechsoft.cn

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there are a variety of dikes with bank slopes oblique at a small angle. Therefore, tensile fractures will easily occur at such a shallow slope. Plenty of investigations and excavations reveal that deformation failures have superficially occurred on the slope due to stress redistribution as a result of incision of the Yalong River. The layout of key water control project and the right bank slope after excavation are presented in Fig.1.

This study attempts to show that the seismicity induced by continuous excavation of the rock slope is related to the behavior of the faults in deep rock mass. The influential region of the working areas governs this behavior. Two important tools have been applied: microseismic monitoring and large-scale numerical modeling. By analyzing the excavation-induced seismicity, it is possible to identify and delineate the particular failure regions that underlie the seismic activity. The objective herein is to correlate the recorded micro-seismic activity during continuous excavation with numerical modeling, as well as conventional measurements and in-situ observations, for early-warning and prevention of associated risks in the right bank slope of Dagangshan hydropower station.

Fig.1 The right bank slope after excavation of the dam

2.2 Numerical geometry

A typical transverse section VI-VI of the right bank slope is obtained. The specific geometry and constraint condition for this model are shown in Fig.2.

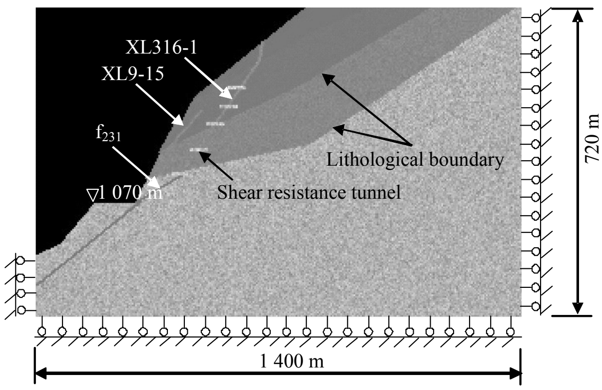
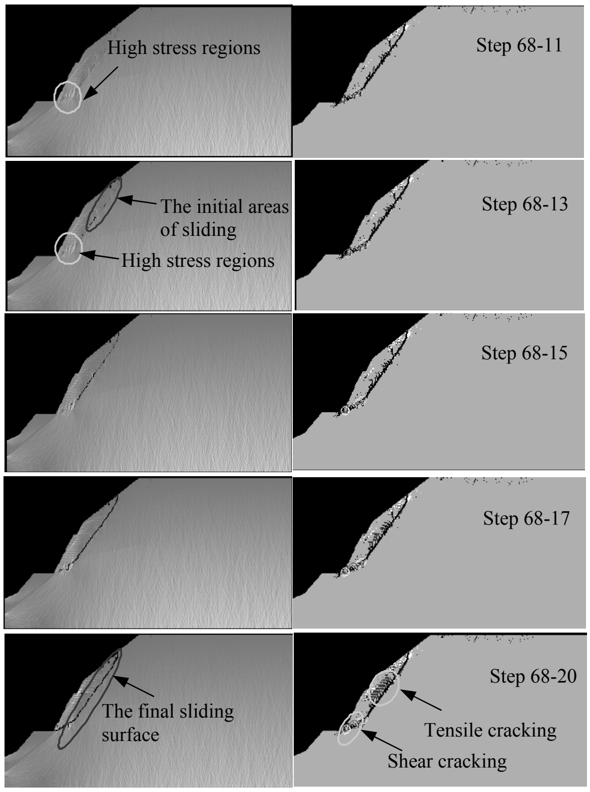


Fig.2 Numerical model of transverse section VI-VI of the right bank slope in Dagagnshan hydropower station, Dadu River, Sichuan Province (the brightness of rock mass in the figure denotes the mechanical parameters such as elastic modulus and the magnitude of strength of elements: the brighter the color is, the larger the values of the parameters are).

The numerical domain of the rock slope, modeled by the RFPA-SRM, has a dimension of 1 400 m × 720 m, and is composed of 375×180 (67 500) quadrilateral iso-parametric finite elements. For the sake of simplicity, the analysis presented here is performed under plane strain condition. The rock is assumed to be heterogeneous with its mechanical properties defined according to the Weibull distribution, whose parameters are listed in related report, and no heterogeneity is introduced in Poisson's ratio and internal frictional angle. The coefficient of strength reduction is 0.05 per step.

2.3 Modeling results

With the RFPA-SRM, both the factor of safety (F_s=1.515) and the corresponding shape and position of the potential slip surface of the rock slope have been obtained. Furthermore, slope instability phenomenon was thus displayed through the progressive failure processes.



(a) Maximum shear stress. (b) AE distribution.

Fig.3 Progressive failure processes of the right bank slope based on the SRM. The brighter the color is, the higher the shear stress in (a); the red circles in (b) present tensile failure, while the white ones express the compressive failure (using RFPA code).

Figure 3 demonstrates the progressive failure processes and associated AE distribution of the right bank slope in Dagangshan hydropower station. It can be seen that the maximum shear stress firstly occurs at the toe of the slope (step 68-11 in Fig.3(a)) with rock masses strength reduction. In the wake of continuous excavation at the bottom of the rock slope, sliding deformation at first happens at the top of the slope along the hanging wall of the fault XL316-1(step 68-13 in Fig.3(a)), then gradually extends and runs through the bottom from the top of the slope, finally leads to whole instability failure. As shown at step 68-20 in Fig.3(b), tensile failure (denote with circles) focuses on the top of the slope, while shear failure (white circles) concentrates at or near the toe of the slope, accompanied with high energy release. Both the shear and tensile failures are triggered at the weakest elements, because the strength profile of the rock material is randomly distributed with specified mean and variance. Although under certain stress conditions, fractures can occur in the rock mass, preexisting fractures in the high stress zones in a slope will tend to open up. As the preexisting joints and fractures in the high stress or stress difference zones open up due to the extension process, fractures would tend to grow into rock bridges, causing micro-seismicity in such zones even if the magnitude of extension is not large enough to cause fresh fractures in the rock masses. These can also interpret why majority of seismic events recorded during the selected period occur around the elevation 1 150 m and higher positions along the main faults XL316-1 and f231, but not nearby the excavation faces at the bottom of the right bank slope. If the rock masses strength at the bottom of the slope can not withstand the high stress induced by production excavation, slippage deformation of fractures will occur along the deep fissure zones from the top of the rock slope. Plenty of micro-seismic events can be thus induced.

As we know, the excavation of rock slope inevitably leads to stress transfer in the surrounding rocks, in the form of either stress release or stress accumulation. Microcracking (i.e. AE herein) may take place in the regions of stress accumulation.

The microcracking phenomenon exactly reflects the response of rock slope to stresses, or so-called "outcome" of stress field. The numerical modeling using by the RPFA-SRM displayed the progressive failure processes of the right bank slope successfully. Furthermore, the high stress field and high stress difference zones and related AE distribution of the rock slope were thus obtained. Accordance between modeled and recorded spatial zones of high stress is important for a more complete understanding of potential instability evolution of the right bank slope. We can conclude that the zones identified by the location of the predominant microseismic activity are well correlated with the zones of high stress identified by the numerical modeling. The demonstration through the location of microseismic activity and the results of numerical modeling of an extension of the stress pattern developed in the working areas are the most significant.

3 Numerical study of unloading-induced strainburst

3.1 Introduction

The world's first recorded rockburst in underground excavation occurred in the South Stanford Coalfield in UK in 1738, and since then rockbursts have been reported in more than 20 countries or regions, including South Africa, Canada, Australia, Germany, USA and other countries. South Africa, particularly in its gold mines, has many rockbursts in record. In China, the first rockburst was recorded in 1933 at Shengli Coalmine in Fushun [8, 9]. In recent years, rockburst hazards occurred frequently in China, not only in mining but also in civil tunneling [10].

Rockburst is defined as damage to an excavation that occurs in a sudden or violent manner and is associated with a seismic event [11]. It is observed that a rockburst often occurs in brittle hard rock subjected to high in-situ and/or mining-induced stress, and is associated with a sudden release of strain energy accumulated in the rock as a result of rock failure. The occurrence of rockburst depends on many factors, such as geology, mining conditions (including geometry of underground openings, excavation method and sequence etc.), and stress condition. It is generally recognized that geology and stress are the two major factors that dictate the occurrence of a rockburst.

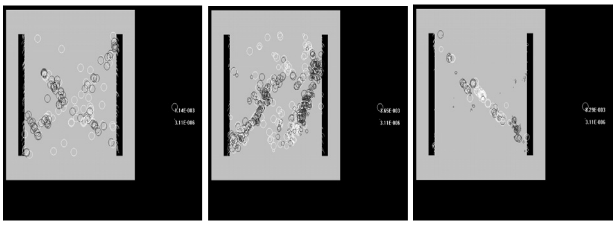
Rock is essentially a heterogeneous material, and the effect of rock heterogeneity cannot be ignored in studying the rock failure process. In RFPA, we introduced an approach of using meso-element to represent rock heterogeneity at the mesoscopic level, and studied the influence of material heterogeneity on rock failure. A constitutive model based on damage mechanics and statistics theory was employed. It was found that rock heterogeneity was the root cause of rockburst precursors.

In recent years, He et al. [12] used a self-developed rockburst testing system simulate strainburst caused by sudden unloading in laboratory. In the experiment, a cuboid sample was loaded triaxially to a pre-defined stress state. The minor principal stress on one side of the specimen was unloaded suddenly and a free surface was formed. This free surface resembles the tunnel wall in an underground opening. The horizontal stress (intermediate principal stress) perpendicular to the unloading direction was remained unchanged. Strainburst may occur upon the unloading of the minor principal stress. If there is no rock failure upon the abrupt unloading, the maximum principal stress (in the vertical direction) was gradually increased until rock failure occurred. In such a fashion, strainburst can be successfully reproduced in laboratory.

Due to the complexity of the rock itself and the unloading-induced rock failure process, very little is known about the strainburst process, from micro-cracking to impending unstable rock failure and associated energy release. With the development of computer technology, numerical simulation has become a powerful tool in studying rock mechanical problems, particularly for the study of rockburst mechanism. Unfortunately, few numerical studies on unloading-induced strainburst have been reported. Hence, in this paper, the RFPA is firstly adopted to conduct numerical tests on square granite samples under uniaxial compressive loading, followed by unloading tests under biaxial loading. In addition, the mechanism of strainburst induced by unloading during tunnel excavation in layered rock masses is investigated. A better understanding of the rock failure process under unloading is important for rockburst prediction and prevention in underground construction.

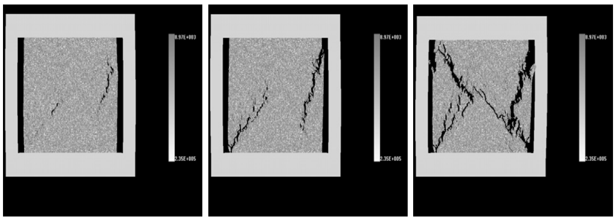
3.2 Results of uniaxial compression test

The RFPA assumes the correspondence between rock damage and AE. Figure 4 presents the failure process of specimen under uniaxial compression. In the AE plot, white and red circles stand for compressive and tensile failure, respectively. Brightness in the stress plot represents the stress magnitude. Every circle in the AE plot represents an AE event and the corresponding event location. Each plot shows the total AE count at the current loading step. The specimen fractures after the peak stress is reached. The greatest count of AE events occurs at step 69. Most AEs originate from tensile cracking and a few from compressive failure. In the elastic modulus plot, the locations of microcracking and the crack propagation process are clearly shown. As more cracks initiate and propagate, the cracks tend to coalesce. As a result, a large number of AE events occur mainly due to the damage of meso-elements at various locations and subsequently a few macro-cracks are observed. The stress variation after loading is shown in the stress plot. During the failure process, stress concentration occurs mainly near the macro- cracks. As the cracks further propagate, stresses are re-adjusted until a new balance state is reached.



Step 63 Step 69 Step 99

(a) AE location evolution during failure process.



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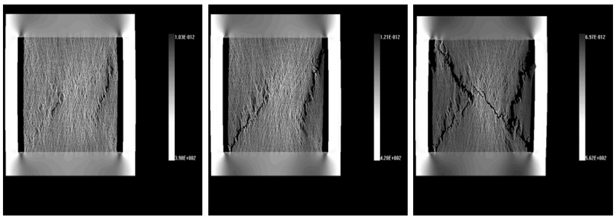
(b) Elastic modulus evolution during failure process.



Step 63 Step 69 Step 99

(c) Stress evolution during failure process.

Fig.4 Failure processes of sample under uniaxial compression.

3.3 Numerical simulation of strainburst under unloading from high confining pressure

The mechanical behavior of rock mass under unloading condition is of great importance for underground construction at depth. It has been recognized by many researchers that a rockburst, in terms of its mechanism, results from catastrophic tensile or shear/tensile failure due to a sudden release of a large amount of elastic strain energy stored in the failed rock itself and in the surrounding rocks, during excavation and unloading, or stress change due to nearby mining. In this study, the mechanism due to sudden unloading is studied using the RFPA.

In the simulation model, an initial confining pressure is applied to the specimen by displacement-controlled loading through the steel frame surrounding the specimen. Then, the horizontal displacement is maintained and the vertical displacement is increased until a pre-defined stress state is reached, and then the vertical displacement is maintained. The steel plate between the specimen and the steel frame is removed suddenly and the right-hand side of the specimen becomes a free surface. In this way, the rock can fail in a brittle manner and strainburst due to sudden unloading is simulated.

The loading curves for the surface of the sample are presented in Fig.5. It can be seen that when the initial stresses are applied, the maximum and minor principal stresses are 83 and 73 MPa, respectively. When the maximum and minor principal stresses reach 996 and 479 MPa, respectively, unloading starts and the maximum and minor drop immediately. Compared with the results under uniaxial compression, it can be observed that there are no platform in the loading curves and no transition from brittle to ductile behaviors. The drop from peak to residual load is abrupt, showing a very brittle post-peak behavior. Thus, unloading is more likely to induce unstable rock failure and release a large amount of strain energy.

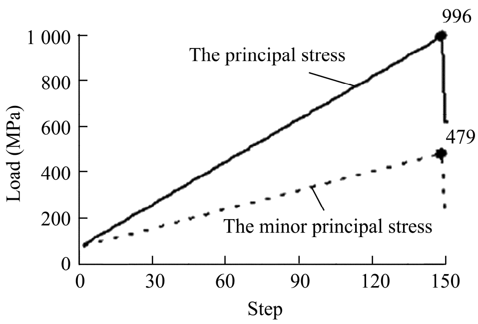


Fig.5 Loading curves at the lateral surface of the specimen under unloading test.

As seen from the failure plot presented in Fig.6, almost no failure occurs before unloading. When the confining pressure is removed suddenly, intensive cracking occurs on the free surface. Compressive failure dominates inside the specimen, accompanied by tensile failure in some areas. The figure clearly shows that there is a stress concentration near the free surface, indicating that irregular stress distribution is generated in the specimen due to unloading.

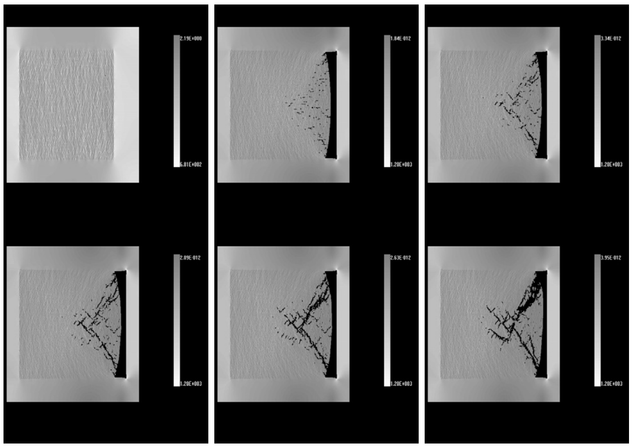
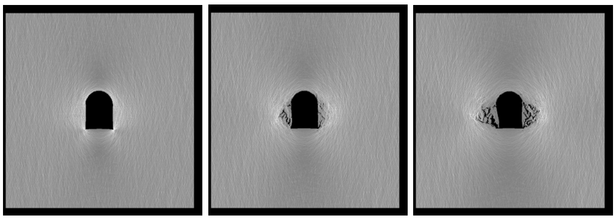


Fig.6 Failure process of Specimen SII due to unloading.

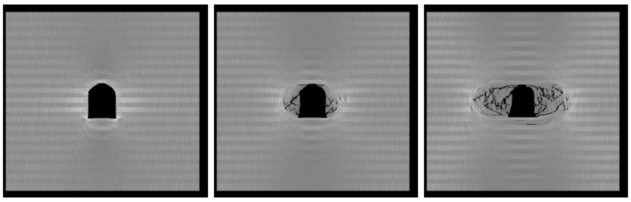
3.4 Numerical simulation of strainburst in tunnel excavation

In underground construction, rockburst, as an engineering hazard, is often encountered in tunnel excavation in brittle hard rock subjected to high in-situ stresses. As excavation leads to unloading and further stress redistribution in the surrounding rocks, the elastic strain energy stored in rocks can be suddenly released, leading to bursting and ejection of rock blocks.

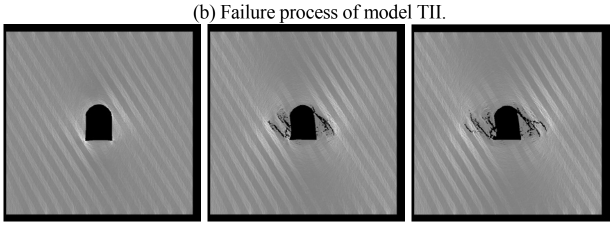
Figure 7 present the failure processes due to abrupt unloading for Models TI, TII and TIII. The crack evolution and stress distribution are simulated through rapid unloading during tunnel excavation in rock masses subjected to high in-situ stresses. Before tunnel excavation in the massive rock, the rock is stable. After tunnel excavation, stress redistribution takes place due to unloading. The radial stresses in the tunnel periphery are completely released, and a high tangential stress concentration appears on the sidewalls. The rock masses in the proximity of the sidewalls experience high compressive stresses. When the stress reaches the peak strength, damage occurs in the rock mass. Cracks first initiate from the tunnel boundary where compressive stress is the highest, and then propagate further away from the boundary as the stresses redistribute. In this case, the tensile failure with some shear failure is predominant.



(a) Failure process of model TI.



(b) Failure process of model TII.



(c) Failure process of model TIII.

Fig.7 Failure process of models TI, TII and TIII due to abrupt unloading.

In layered rock masses (models TII and TIII), stress distribution after tunnel excavation leads to a larger stress increment in the hard rock layers and a smaller increment in the soft rock layers. Hence, the maximum principal stress in the hard rock layers is much higher than that in the soft rock layers, making it burst-prone. Compared with the results of tunnel excavation in horizontally layered rock masses, rock deformation is asymmetrical for dipping rock layers. Rock layer movement along the bedding interfaces, especially along the interface with lower strength, was observed. In addition, the deformation mechanisms of the left- and right-hand sidewalls are different. Sliding along the interface between rock layers was observed on the left-hand sidewall, but toppling was observed on the right-hand sidewall. The deformation magnitude and the location of the maximum deformation are also different on both sides. The maximum depth of failure is observed in model TIII, 1.4 times of the tunnel width.

4 Conclusions

Rock brittle failure is a complex process, involving not only the initiation and propagation of single crack, but also the initiation, propagation and coalescence of many cracks.

In this paper, we discuss two basic engineering application of RFPA, i.e. the slop stability analysis and rockburst modeling. We can conclude that:

(1) The simulation results indicate that satisfactory results can be obtained by the RFPA in geotechnical engineering such as high slope stability

(2) Using the RFPA method, the mechanism of unloading-induced strainburst was investigated, which indicated that the stress path has a large impact on the strainburst mechanism. Under uniaxial compressive loading, rock failure is characterized by progressive tensile failure leading to instability. However, under biaxial loading and unloading, rock failure is characterized by a combined failure mode, showing shear failure in a large zone and local tensile failure near the free surface created by unloading.

(3) The modeling or rockburst suggest that rock failure due to abrupt unloading exhibits a distinct brittle failure behavior and is associated with large dilation in the unloading direction. There is no brittle to ductile transition during the failure process. Rock failure is sudden and the failed rock is highly fragmented.

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