A NUMERICAL STUDY OF ROCKBURST DAMAGE AROUND EXCAVATIONS
INDUCED BY FAULT-SLIP

by

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A thesis submitted to the Faculty and the Board of Trustee of the Colorado School of Mines in partial fulfillment of the requirements for the degree of Doctor of Philosophy (Mining and Earth Systems Engineering).

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ABSTRACT

The increasing depths of mining and associated in-situ stresses have made rockburst a serious risk for underground mining and tunneling. Therefore, it is important to understand the energy mechanisms of unstable rock failure to reduce rockburst hazards. In this dissertation, an energy approach is developed by integrating energy equations into UDEC software where energy components including elastic strain energy, plastic strain work, and joint friction work can be tracked in each individual zone or contact of the numerical model at every time step. Rapid and large changes in energy components were used to identify unstable rock failure modes, and the magnitudes of these changes were used to quantify the unstable rock failure intensity.

Simulations of compression testing confirmed that unstable rock failure tends to occur in stiff and brittle rock loaded with a soft loading system, resulting from the fact that a small loading system stiffness (LSS) and rock stiffness will increase the amount of stored elastic strain energy in the model, while a brittle rock will require less amount of elastic strain energy for plastic strain work during the rock damage process. Direct shear test modeling results show that unstable slip failure at the same peak shear stress is less likely to occur in discontinuities with smaller shear stiffness, larger roughness, and/or that are embedded in a rock matrix with smaller stiffness. A stiff rock matrix surrounding the discontinuity can store less elastic strain energy to be transferred to the joint during failure, while a smaller shear stiffness represents a larger slip-weakening distance and requires more elastic strain energy for the slip failure.

Rockburst damage induced by fault slip at the excavation scale was investigated with a circular excavation model with a nearby discontinuity, and a critical discontinuity distance parameter was proposed to quantify the rockburst potentials for this type of rockburst. The critical discontinuity distance refers to the minimum normal distance between the excavation and discontinuity plane for the excavation to stay stable; therefore, a larger critical discontinuity distance represents a higher rockburst potential. The influence of the in-situ stress, discontinuity dip angle, excavation radius, and fault length, are analyzed with the hypothetical base model. Finally, a rockburst event from the drainage tunnel at Jinping II hydropower station is simulated to validate the numerical results with the measured released energy and the estimated normal fault distance.
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CHAPTER 1

INTRODUCTION

1.1 Overview

With the expanding demand for mineral resources due to rapid economic growth in many countries, the increasing depths of hard rock mines have increased safety risks in the mining industry (Ortlepp 2005). As mining depth increases, the magnitude of stress in both the vertical and horizontal directions around mining excavations becomes unfavorable to the safety of underground workers and equipment. The impacts of rockbursts on mine production can range from small mine production disruptions to workforce injuries and fatalities.

In high-stress conditions, rock failure may occur suddenly and violently around underground openings, when the rock loses its strength spontaneously and rapidly after the attainment of peak strength. Therefore, the occurrence of a rockburst is always accompanied by a large amount of released kinetic energy or seismic energy, while stable rock failure occurs gradually and releases minimal kinetic energy during its strength weakening process. Mine Safety and Health Administration (1984) defines rockburst as “a sudden and violent failure of overstressed rock resulting in the instantaneous release of large amounts of accumulated energy.” A rockburst also can be defined as a large volume of ejected rock fragments caused by the instantaneous release of energy from highly stressed surrounding rock in deep underground openings (Cook 1965).

Fault-slip is related to the unstable slip failure along pre-existing discontinuities or weak planes in a rockmass, whose failure also can result in a large amount of seismic energy, and can trigger rockburst damage around excavations (White and Whyatt 1999; Sainoki and Mitri 2014). Damage inflicted by fault-slip can be caused by either slip along pre-existing discontinuities near excavations, or by seismic waves associated with slip movement deeper into the rockmass (White and Whyatt 1999; Ortlepp 2002). Most slip-induced seismicity in underground mines is not hazardous, and only few seismic events will cause rockburst damage. However, fault-slip close to excavations can change the stress to a hazardous stress level around tunnels or mine stopes and directly result in severe rock damage around openings. This type of rockburst damage usually involves rock failure in both intact rock material and along discontinuities in the rockmass (White
Understanding the underlying mechanism of rockburst damage induced by fault-slip is critical to the reduction of rockburst hazards in underground mines and tunnels. The mechanical causes of unstable rock failure in intact rock, along discontinuities, and their combination are studied from the perspective of energy balance in this dissertation.

1.2 Mechanical causes of rockbursts

Rockburst has become a serious mining hazard with the increase of mining depth and in-situ field stress. Numerous rockburst cases have been documented with extensive details in the past decades (Whyatt, J, W. Blake 2002; Zhang et al. 2012a; Sainoki and Mitri 2016; Leveille et al. 2017; Keneti and Sainsbury 2018). The primary contributing factors leading to unstable rock failure can be summarized from the rockburst cases, which can greatly benefit engineering judgment on the potential for rockburst under given stress or geological conditions. Based on the primary locations of rockburst events and their associated inducing factors, rockburst events can be classified into strain bursts, pillar bursts, and fault-slip, although a complex rockburst occurrence could also be a combination of any two or more of these categories. The primary contributing factors for each type of rockburst will be discussed with documented cases in underground excavations in the following sections.

1.2.1 Strain burst

Strain bursting, a sudden and intense fracturing of rock with associated ejection of material into an excavation, is very hazardous to the safety of mine workers in deep hard-rock mines, though it usually does not induce large seismic magnitudes. During the last decades of 20th century, there were 22 fatalities at Coeur d’Alene district mines resulting from the occurrence of rockbursts, while nine and eleven of the fatalities were caused by strain bursts and pillar bursts respectively (Whyatt, J, W. Blake 2002). Strain bursts typically occur in deep excavations under high stress conditions, where damage can result from the violent fracturing of intact rock around the perimeter of deep openings. Another example is a Tunnel using Boring Machine (TBM) in the Jinping hydropower station in Sichuan, China, where the overburden exceeds 2,500 meters, leading to high principal stresses such that the shattering of the brittle rockmass around the circular excavation is possible, as illustrated in Figure 1.1 (Zhang et al. 2012b). The depth of the failure zone was as much as 3 m after the first rockburst on October 9, 2009, and a second rockburst followed on the same day.
Figure 1.1 Failure zone morphology in the drainage tunnel (R=7.2 m) after two rockbursts (Zhang et al. 2012b).

Figure 1.2 Schematic diagram of strain burst resulting from localized superficial stress concentration (Ortlepp and Stacey 1994)

Strain bursts also can be induced by local stress concentrations resulting from the different stiffness of the rock blocks bounded by rock structures around excavations (White et al. 2002). The different stiffness magnitudes of the rock blocks could result in local stress concentration around the surface of excavations and violent rock failure after the peak strength is reached (Ortlepp and Stacey 1994). An example diagram is shown in Figure 1.2. The redistributed stress in stiff rock blocks will be higher than that in soft rock blocks, but the stored elastic strain energy
in the soft rock blocks can be transferred to the failed rock blocks, resulting in the ejection of rock fragments (Kaiser and Cai 2012).

Buckling of rock slabs and their ejection can be classified as the type of strain burst, which usually has a large intensity and represents a serious risk to both workers and equipment. Discontinuities such as joints, fractures and bedding planes parallel to the excavations can cause the buckling of rock slabs. A strain burst at the Lucky Friday mine caused 320 tons of rockmass from both ribs to be ejected into the mine drift, as illustrated by the schematic of the cross section in Figure 1.3 (White and Whyatt 1999; White et al. 2002). The width of the drift was nearly doubled after the occurrence of the rockburst, and this particular case was recognized as buckling failure due to slip failure along the bedding planes.

![Diagram of rockburst site in Lucky Friday Mine](image)

Figure 1.3 Schematic diagram of the cross section of a rockburst site in Lucky Friday Mine (White et al. 2002).

### 1.2.2 Pillar burst

In underground mines, pillars are designed to provide support to the overburden with a safety factor utilized in design, which implies that the estimated strength of the pillars should be larger than the expected in-situ stress magnitude. Since rockmass tend to be anisotropic, discontinuous and heterogeneous, however, there is still a slight chance of unexpected pillar failure even when standard pillar design procedures are followed. In some cases, the pillars can fail suddenly and violently before any preventive measures (such as additional support) can be applied. In certain cases, it is even possible for one pillar failure incident to trigger a chain of pillar failures in a large area, resulting in injuries, fatalities, and equipment damage (Gale 2018a; Mark 2018).
The Crandall Canyon Mine had a large scale of pillar collapse in 2007, whose failure was attributed to the inappropriate dimensions of the pillars and barriers (Gates et al. 2007). This underground coal mine was located in Utah with overburden depths ranging from 120 to 480 meters. Six mine workers were trapped in the pillar collapse area when the entries in the mines were completely blocked by the pulverized rockmass, and the violence of the pillar burst can be inferred from the resulting rock debris as shown in Figure 1.4. In the later rescue, the excavation towards the position of trapped mine workers induced secondary collapse, which took the lives of two rescue workers and one mine safety inspector. Crandall Canyon was permanently sealed after weeks of attempted rescue, leading to the death of nine people. This large-scale pillar burst resulted in ground surface subsidence as much as 0.27 m, and an average roof-floor closure between 0.3 m and 0.5 m.

![Figure 1.4 The rock debris after the pillar burst in Crandall Canyon Mine barriers (Gates et al. 2007).](image)

The Solvay Trona Mine, a room-and-pillar trona mine in Wyoming, had a sudden and large-scale collapse without any apparent warning on February 3, 1995 (Ferriter, R. L. 1996). The mining depth was approximately 480 m, and the Trona seam thickness was about 3.5 m, with an
excavated thickness of 2.5 m. Almost two square kilometers of mine workings were damaged by the large-scale pillar burst, resulting in a local seismic magnitude of 5.3. At the moment of collapse, 55 miners were working underground and ten of them were injured, resulting in one fatality. Because this was a large scale collapse, the maximum measured subsidence was as much as 1 m, and the average subsidence was estimated to be 0.6 m (Pechmann et al. 1995).

![Figure 1.5 The debris after the collapse from the entry floor with a relatively intact ceiling (Pechmann et al. 1995).](image)

1.2.3 Fault-slip

Pre-existing large discontinuities such as faults, bedding planes, and dyke contacts can have large shear stress long before the commencement of mining and excavation activities. A discontinuity close to failure can be activated by the decrease in normal stress or increase in shear stress along the discontinuity during excavation, and the failure can release a large amount of seismic energy. Consequentially, the rock damage induced by the fault-slip can result from the large seismic energy or redistributed stress concentrations (Rice 1983; White and Whyatt 1999).

Seismicity at the Lucky Friday was recognized to be dominated by the fault-slip in different rock structures, and the sources of seismic events were found to be coincident with major faults (White and Whyatt 1999). All the large seismic events with magnitudes ranging from 2.5 to 4.2 were identified as fault-slip, vein-slip or shear zone-slip at Lucky Friday, but only a few would be expected to cause severe rock damage, unless they intersected underground excavations where the rock was not strong or poorly supported. White and Whyatt (1999) summarized the commonly observed rockburst damage occurred caused by movement along the discontinuities (Figure 1.6). The slip movements would cause unfavorable stress concentrations and violent rock failures around excavations.
Mining-induced seismic energy can also trigger rockburst damage around excavations by destabilizing the rock equilibrium with seismic waves. This type of rockburst damage can be observed after a large seismic event in deep mines. Figure 1.7 shows severe rockmass damage associated with a large seismic event, and a large volume of rockmass where the support system was destroyed during the event (Hudyma and Potvin, 2004).
1.3 Methods for studying unstable rock failures

1.3.1 Analytical methods

Analytical methods play a significant role in developing key insights into the rockburst mechanisms in deep mines and tunnels, and can be used to identify the most important contributing factors for a given case. Because a rockburst represents a sudden rock failure resulting from the violent release of energy, studies on the energy balance within a rock have been performed to explain the energy mechanisms of rockbursts since the 1960s. Although analytical methods are usually based on the simplifying assumption that a rockmass is homogeneous, isotropic and linear elastic, the analytical solutions are valuable and can benefit geotechnical engineers in solving complex problems. Rock failure modes, including stable and unstable rock failure, are significantly influenced by the rock failure behavior after the peak strength. Cook (1965) and Salamon (1970) put forward the concept that the rock failure modes highly depended on the stiffness of the loading system and post-peak stiffness of the rock sample during a compression test. The post-peak behavior of rocks could not be obtained in the laboratory until a rigid 100-ton compression test machine was developed by Wawersik and Fairhurst (1970).

In the laboratory, the stress at the contact between a rock specimen and the loading system has the same magnitude; however, after the peak stress, the resistance from the rock specimen transitions to the strain weakening stage and the loading system transitions to unloading stage (Figure 1.8). The stress in the stiff loading system is smaller than that in the rock specimen at the strain of $\varepsilon_x$, which implies that the strain in the rock cannot increase to $\varepsilon_x$ and the equilibrium point is reached before the strain magnitude of $\varepsilon_x$. However, the stress in the soft loading system is larger stress than that in the rock specimen at a strain magnitude of $\varepsilon_x$. Therefore, the unbalanced condition resulting from this stress difference will accelerate the rock failure spontaneously and the equilibrium can only be reached after that strain magnitude. Theoretically, the failure of rock will occur in a violent and unstable mode if the stiffness of loading system is smaller than the post-peak stiffness of the rock (Cook 1965; Salamon 1970, 1984; Wawersik and Fairhurst 1970), and the enclosed area by the stress-strain curves of rock and loading system represents the released energy during unstable rock failure.
From an energy perspective, a relatively softer loading system can provide more elastic strain energy towards the rock during its failure, while a brittle rock, also presented by the slope of the post-peak stress-strain behavior, determines the amount of dissipated elastic strain energy during the rock failure process (Cook 1965; Gu and Ozbay 2014; Xu and Cai 2017a). Rice (1983) defined the stable and unstable response of discontinuity-slip based on a direct shear test in the laboratory, where the stability of the discontinuity-slip depends on the shear stiffness of the
discontinuity and surrounding rock material. The curves of shear force-displacement in the discontinuity and the surrounding rock material are shown in Figure 1.9, where the surrounding rock material serves as the loading system of the discontinuity. Moreover, the enclosed area between the curves of shear force-displacement and soft loading system was the amount of released energy during the unstable slip failure.

The theories of unstable failure described above for both intact rock material and discontinuities are widely recognized, and many research efforts have been undertaken based on those analytical solutions of released energy, to study unstable rock failure mechanisms (Gu and Ozbay 2014; Manouchehrian and Cai 2015; Leveille et al. 2017; Xu and Cai 2017a; Khademian and Ozbay 2018). However, analytical solutions of geotechnical problems have to be based on many assumptions relative to the complexity of rockbursts in underground mines and tunnels.

1.3.2 Experimental study of unstable rock failure

Laboratory testing of rock failure behavior is helpful to study the factors influencing rockbursts, and some rock properties such as the post-peak behaviors of rock are essential for studying rockburst. As previously discussed, a stiff loading machine can be employed to obtain the post-peak behavior of the rock, while a soft loading machine helps to investigate the violent rock fragmentation process by reproducing unstable rock failures in the laboratory.

He et al. (2010, 2012, 2018) modified a true-triaxial rockburst testing platform to simulate the rock fragmentation process during a strain burst in a tunnel wall or face. The schematic of the test machine is presented in Figure 1.10a. The rock specimen was loaded on its six faces from three mutually perpendicular directions to reach the in-situ field stress before unloading one face. With increasing loading stress from the other five faces, the sound, velocity, and volume of ejected rock fragments from the unloading surface were recorded to study the strain burst behaviors and intensity in the laboratory (Figure 1.10b). The acoustic emission signals tend to shift towards a signature of higher amplitude and lower frequency with the increase in loading stress before rockburst.
Li et al. (2011) carried out a large-scale physical simulation test for rockbursts in tunnels by opening a hole in a constructed rock model after loading it from four sides (Figure 1.11). The rock model was designed to simulate the physical and mechanical properties of the rockmass in the Jinping Hydropower Station in China. After excavation, the boundary normal stress was increased to 1.85 times of the previous stress to induce fractures around the opening, and spalling, cracking, buckling and breaking in the model tunnel was observed during the loading process. The induced fractures were identified as tensile or tensile-shearing mode fractures, and they concluded that the damage mechanism associated with strainburst was mainly tensile in nature.

Figure 1.10 Illustrations of the (a) experimental system and (b) ejected rock fragments during rockburst (He et al. 2012, 2018).

Figure 1.11 Schematic diagram of the rock model geometry and loading condition (Li et al. 2011).
Fakhimi et al. (2016) designed a soft steel loading frame and placed it between the top of the rock specimen and loading machine in compression tests so that a large amount of stored elastic strain energy in the soft steel frame became an energy source for unstable rock failure. The steel frame structure and the rock specimen position are shown in Figure 1.12, where the top of the frame is meant to represent the roof structure of a pillar. The uniaxial compression testing of sandstone specimens showed that the particle velocities of failed specimens caught by a high-speed camera can be as much as 4 m/s, which was close to the ejected pulverized rock in an actual field rockburst.

Figure 1.12 The frame structure of the steel beam and position of rock specimen (Fakhimi et al. 2016).

Zhou et al. (2015) conducted direct shear tests using brittle, cement mortar to investigate fault-slip mechanisms. The experimental models with different asperity heights within the discontinuity are shown in Figure 1.13. The slip failure intensity and scale was found to approximately increase with the asperity heights and normal stress. The development process of the fault-slip and shear rupture was also investigated to gain insight into the mechanisms of the structure-controlled rockbursts in deep hard-rock excavations. The shear failure of the asperities and tensile failure from the root of asperities was observed during the slip failure or shear failure along the discontinuity.
Experimental methods for studying unstable rock failure are usually limited by the measurements of stress or strain changes within a rock, although some violent rock failure phenomenon attributes such as the ejected particle velocity or acoustic emission counts can be measured. Generally, the contributing factors of rockburst can include geotechnical conditions, mining activities, excavation volume, unfavorable stress conditions, and other factors, making it difficult to mimic actual underground conditions in the laboratory.

### 1.3.3 Numerical simulation of unstable rock failure

Numerical methods for analyzing different aspects of unstable rock failure have been widely applied in recent years. The numerical simulation of unstable rock failure ranges from laboratory strain bursts to large-scale fault-slip cases in underground mines. In addition, many other factors such as geotechnical conditions, excavation sequence and mining geometry can be considered.

Zubelewicz and Mroz (1983) used the finite element method to simulate the dynamic failure process of rockburst, and concluded that the sudden increase in kinetic energy can be used as an indicator of unstable rock failure. This indicator was also observed during the dynamic rock failure process by Müller (1991). The change in kinetic energy as a function of model time step was plotted as shown in Figure 1.14, where the sudden increase in kinetic energy from time step of 600 could be observed during an unstable pillar failure process.
Mitri (1999) proposed the burst potential index (BPI) with the parameters of the estimated seismic energy release rate and strain energy storage rate in a simple FEM model. The BPI considered both the rock strength and rock stiffness characteristics in evaluating the rockburst potential. The model results showed that smaller mining steps can greatly reduce the amount of released kinetic energy, resulting in a lower rockburst potential.

Jiang (2010) utilized the local energy release rate (LERR) as an index of rockburst potential to simulate the rockburst events at Jinping hydropower station in China. They estimated the decrease in elastic strain energy density before and after the tunnel failure with stress results from FLAC3D. Their numerical analysis was compared with known rockburst events, showing that the LERR could be used to predict the rockburst intensity.

Kias (2013) studied the indicators of unstable compressive rock failure using displacement-softening continuum and discontinuum bonded particle models in the discrete element program Particle Flow Code in Two Dimensions (PFC2D). The results showed that the strain-softening model had better performance in simulating unstable rock failure, based on the post-peak softening
characteristic of the rockmass. Rapid increases in damped kinetic energy, instantaneous kinetic energy, and unbalanced force were identified to represent the unstable rock failure.

Garvey (2013) investigated compression-induced brittle rock failure with the strain-softening model in FLAC, where the maximum unbalanced force, maximum velocity, and maximum shear strain rate were identified as indicators of unstable rock failure by comparing their corresponding magnitudes between known stable and potentially unstable failure cases. These three indicators were also used to identify pillar failure modes given various width-to-height ratios under different loading system stiffness (LSS).

Levkovitch and Beck (2002, 2014) proposed to use the energy release rate to evaluate the likelihood of seismic events and hazards in mines. The released energy can be estimated with the monitored seismic data during a period of time in a mine, and a higher energy release rate in a specific area of a mine indicates a larger rockburst potential. Moreover, distributions of modeled energy release rate and measured seismicity were compared for the Nickel Rim South Mine in Canada.

Manouchehrian (2016) studied unstable rock failure using the finite element software Abaqus-Explicit. Indicators of Loading System Reaction Intensity (LSRI) and the Maximum Unit Kinetic Energy (KEmax) were proposed to distinguish between stable failure and unstable failure (Figure 1.15). When a rock failed in an unstable mode, its stress-strain curve was observed to have some deviations from the rock failure in a stable mode. The LSRI indicator was utilized to study a fault-slip event at the Jinping II hydropower station in China.

![Figure 1.15 A schematic diagram showing the indicator of LSRI in a compressive test.](Manouchehrian 2016)
Poeck (2017) simulated two in-situ pillar burst cases with field-scale models in UDEC. With the analytical method, the released energy density during a pillar burst was obtained by calculating the intersected area between the stress-strain curves of the pillar and overburden load (Figure 1.16). Only a 1.3% difference was observed between the analytical released energy results and the numerical results from UDEC, indicating that the calculation of damped kinetic energy in UDEC was relatively accurate. The numerical results of damped kinetic energy and ground surface subsidence for a mine-scale model were calibrated and compared with the field measured data, indicating that the UDEC software could simulate damped kinetic energy reasonably well in large scale models.

Gu and Ozbay (2014, 2015) studied fault-slip failure using both the laboratory direct shear test and a large-scale, fault-slip model in UDEC. The numerical results confirmed previous analytical work by Rice (1983) showing that the stiffness of the surrounding rock material had to be smaller than the discontinuity’s post-peak stiffness for the occurrence of unstable slip failure. The indicators of fault-slip were the sudden decrease in shear stress and increase in shear displacement along the discontinuity (Figure 1.17). The potential for fault-slip would be higher if the rockmass stiffness were smaller.

![Pillar Stress and Overburden Load vs Strain](image)

Figure 1.16 Pillar stress-strain data with the ground reaction curve of the overburden (Poeck 2017).
Sainoki and Mitri (2014) analyzed the factors which affected fault-slip at the Garson Mine in Canada using FLAC3D. Their results showed that mining depth, friction angle, and location of the fault influenced the maximum slip displacement and rate during fault-slip. Moreover, fault-slip taking place in a fault with a rough surface tended to induce high peak particle velocity. A back-analysis of fault-slip for the Garson Mine in Canada was carried out by calibrating the fault surface roughness and friction angle. The model results were validated with measured seismicity and peak ground acceleration by the monitoring system.

Kim and Larson (2017) simulated a coal pillar and concluded that more retained elastic strain energy in rock may indicate a higher rockburst potential, resulting in more plastic strain work during rock failure. The cleating orientation would greatly influence the ratio of retained elastic strain energy to plastic strain work within zones of a pillar, and a larger ratio could probably represent a higher rockburst potential.

Vazaios et al. (2019) investigated the rock fracturing process and energy mechanisms of rockburst using the hybrid finite-discrete element method (FDEM). The impact of rock structure on rockbursting within a deep hard-rock tunnel under different stress conditions was analyzed after the calibration of the rockmass properties and integration of discrete fracture network. The results indicated that the rock structure could affect strain burst intensity around the excavation; where smaller blocks closer to the tunnel boundary tended to have larger ejected velocity, while larger blocks had a larger amount of kinetic energy but smaller velocity (Figure 1.18). The mass and
velocity of the ejected rock blocks in numerical models were compared with empirical solutions, demonstrating the potential of the FDEM method to realistically estimate released kinetic energy.

Figure 1.18 Contour of ejected rock size and speed around the tunnel with radius of 1.75 m (Vazaios et al. 2019).

1.3.4 Summary

Several previous studies have recognized that the presence of discontinuities such as bedding planes and faults near an excavation can not only reduce the rockmass strength, but increase the potential of unstable rock failure in high stress conditions (White and Whyatt 1999; Whyatt, J. W. Blake 2002). Unfavorable stress conditions resulting from geological structures are the major factors causing the occurrence of rockbursts, and this effect is more obvious when the discontinuity plane is parallel to the excavations (Cai 2008; He et al. 2012). As previously discussed, discontinuities such as bedding planes can cause the buckling failure of rock slab (Figure 1.3). Mining activities near major faults or large bedding planes might result in rockburst damage around excavations and within pillars, whose failure can be any combination of strain burst, pillar burst and fault-slip (Keneti and Sainsbury 2018). Therefore, it is critical to consider the influence of structural weakness planes on the occurrence of rockburst.

Analytical solutions are helpful for simplified geotechnical problems and laboratory tests can help enhance fundamental knowledge on the occurrence of unstable rock failures. However, analytical solutions and laboratory methods have great limitations in explaining the complex
phenomenon of rockburst around deep excavations, given that they are based on many simplifications and assumptions. Numerical approaches represent useful tools to track the detailed changes of stress and strain tensors in rock material and discontinuities during simulated excavation activities. Additionally, unstable rock failure is a dynamic process with rapid and large changes in different mechanical energy component; explicit numerical methods for simulating rockbursts can capture the sudden rock failure and rock ejection process, which can help gain insights into why rockbursts occur and where their energy comes from.

Previous research work has been conducted to simulate the unstable rock failure in continuum and discontinuum models from laboratory tests to large scale rockbursts leading to helpful insights. Discontinuum models have an advantage over continuum models in simulating the influence of discontinuities on rockburst occurrence and damage around deep excavations. UDEC is a two-dimensional numerical software that has been used by many researchers to study unstable rock failure in intact rock material and along pre-existing discontinuities, and has shown great potential in assisting in the analysis of rockbursts influenced by geological planes (Garvey 2013; Kias and Ozbay 2013, 2014; Gu and Ozbay 2014; Poeck 2017; Khademian et al. 2018). The previous modeling results with UDEC are also very helpful for understanding of the interactions between rock material and discontinuities during unstable rock failure. For example, the sudden increases in damped kinetic energy and unbalanced force from UDEC models have been successfully utilized as reliable indicators of unstable rock failure in numerical models (Garvey 2013; Poeck 2017).

In this dissertation, energy equations of different energy components within both intact rock and discontinuities are integrated into the numerical software UDEC. Therefore, the energy transformations during compression failure and shear failure can be tracked to study the energy mechanisms of unstable rock failure in numerical models and to gain fundamental knowledge about the factors that influence the energetic aspects of rockburst occurrence. In addition, fault-slip induced rockburst damage around a large-scale circular excavation is also simulated to investigate the energy transformations behind this type of rockburst event. Finally, a large-scale fault-slip event from Jinping II hydropower station is analyzed using the energy approach developed in this dissertation.
1.4 Thesis outline

This thesis consists of six chapters. A brief summary of the chapters is described below.

- The introductory chapter (Chapter 1) presents a general overview of the research topic, mechanical causes of rockbursts, common methods for studying rockbursts, and the research scope of this study.

- In Chapter 2, numerical simulations of unstable rock failure mechanisms through analysis of energy transformations are conducted through uniaxial compression tests simulated using UDEC software. The influences of the LSS, the rock stiffness and the rock brittleness on rock failure modes are interpreted from an energy perspective. In addition, the released energy from numerical results is compared with analytical solutions.

- In Chapter 3, numerical simulations of unstable fault-slip failure along pre-existing discontinuities in rock are carried out by tracking the changes in different energy components to investigate the energy mechanisms of unstable slip failure. A model with a discontinuity is used in the context of a direct shear test model with multiple different values of Young’s modulus of the intact rock material, joint shear stiffness of the discontinuity and normal stress acting on the discontinuity.

- In Chapter 4, a circular excavation near a discontinuity is simulated to study the rockburst damage induced by nearby unstable slip failure. Rockburst potential as a function of the critical discontinuity distance near underground excavations is utilized to investigate the influence of horizontal-to-vertical in-situ stress ratios, angles between the discontinuity plane and the major principal stress, tunnel radii (or excavation volume) and discontinuity lengths.

- In Chapter 5, the application of the developed energy approach for simulating rockburst cases is presented. A fault-slip failure which occurred in the Jinping II drainage tunnel is investigated with UDEC, where the fault distance and the amount of released energy from numerical results are calibrated and compared with the measured data in the field.

- Chapter 6 summarizes the main conclusions from this study, limitations and suggestions for future work.
2.1 Abstract

For rock specimen in uniaxial compression, the energy transformations from elastic strain energy in both the rock and the loading system to plastic strain work in the rock can be identified with the changes in these energy components, whose rates are also useful indicators for distinguishing stable and unstable rock failure. In this study, the influences of the loading system stiffness (LSS), the rock stiffness and the rock brittleness on rock failure modes are examined. The observed energy transformations during rock failure in numerical models are interpreted from an energy perspective. The results show that unstable rock failure tends to occur in rock with large brittleness and small stiffness under a soft loading system. A low LSS and rock stiffness will increase the magnitude of stored elastic strain energy before rock failure, while a brittle rock requires less elastic strain energy to be converted plastic strain work than a ductile rock during its failure. This energy-based approach is useful for investigating potential unstable rock failures that could ultimately be applied to analyze complex mine-scale rockburst cases.

2.2 Introduction

Understanding the mechanisms of unstable failure is of critical importance to reduce unstable rock failure hazards in deep underground mines and excavations. During a rockburst event, a large amount of pulverized rock can be violently expelled into underground openings, causing injuries or fatalities and disrupting mining activities (Whyatt, J. W. Blake 2002). Many previous experimental studies and numerical simulations have been conducted that focus on the underlying mechanisms of unstable rock failure behaviors (He et al. 2012; Levkovitch et al. 2013; Gu and Ozbay 2015).

Rock failure modes can be classified into stable failure and unstable failure, depending on whether or not a volume of rockmass can consume the energy from its surrounding rockmass during rock failure (Cook 1965; Salamon 1970, 1984; Rice 1983; Kias and Ozbay 2013; Zhao and
Cai 2014). Although many types of unstable rock failure exist in different geological and physical conditions, the general requirements for unstable rock failures are brittle material behavior, and a relatively soft loading system (Kaiser and Cai 2012; Walton and Diederichs 2015). Cook (1965) and Salamon (1970, 1984) proved that the rock failure modes were highly dependent on the LSS and the post-peak stiffness of the failed rock. The loading system can vary from a loading machine in the laboratory to the rockmass surrounding an excavation. Generally, a steep slope of the post-peak stress-strain curve of a rock specimen can be interpreted to indicate brittle rock behavior, while a gentle slope represents ductile rock behavior (Hajiabdolmajid and Kaiser 2002; Zhang et al. 2016). The LSS affects the amount of elastic strain energy surrounding the target rock, and a relatively soft loading system can therefore store more elastic strain energy and tends to cause unstable rock failure (Beck and Brady 2002). In theory, if the LSS is larger than the post-peak stiffness of the rock material, the rock will fail gradually in a steady and stable mode; however, rock failure will occur in a violent and unstable mode if the LSS is smaller than the post-peak stiffness of the rock (Board et al. 2006; Xu and Cai 2017a). From an energy perspective, a high rock material brittleness represents a low capability of dissipating energy during failure, and a soft loading system provides more elastic strain energy to the rock during its failure.

Various laboratory-based experiments have been conducted to analyze the underlying mechanisms of unstable rock failure (Wawersik and Fairhurst 1970; Singh 1987, 1988; Zhao and Cai 2014; Heinze et al. 2015; He et al. 2016a; Kim and Larson 2017; Leveille et al. 2017). To study pillar bursting in the laboratory, Fakhimi et al. (2016) designed a steel beam to act as an energy absorber, which was placed between the loading platen and rock specimen. The stored elastic strain energy in the steel beam could fail the sandstone samples violently, and a maximum rock fragment velocity of over 4 m/s was identified using a high-speed camera. A true tri-axial rock test system was designed by He et al. (2010) to simulate strain bursting at a tunnel face. The rock was loaded in three mutually perpendicular directions to a pre-defined stress condition, and abrupt unloading in one horizontal direction was used to cause a strain burst.

Numerical simulation of laboratory rock failures and mine-scale rockbursts have been commonly conducted to analyze mechanisms of unstable rock failure (Zubelewicz and Mróz 1983; He et al. 2010; Leveille et al. 2017; Su et al. 2017). Manouchehrian and Cai (2015) proposed to use the transferred energy ratio and loading system reaction intensity as indicators to distinguish between stable and unstable failures in a UCS test. Fakhimi et al. (2016) found that the rock
specimen’s diameter and strength had large impact on the ejected velocity of rock fragments, and weakening of the rock by drilling holes in it could reduce the violence of rock failure in numerical models. Poeck (2017) simulated two pillar burst events with mine-scale numerical models based on actual mine cases, and compared the released energy and ground subsidence from numerical models with field monitored data to evaluate the reliability of the numerical results. Sainoki and Mitri (Sainoki and Mitri 2016) simulated a fault-slip rockbursting mine in Canada, and they concluded that mining depth, friction angle and fault location had significant impacts on the maximum magnitude and rate of slip displacement along the fault. Vazaios et al. (Vazaios et al. 2019) simulated the failure and fracturing process of strain bursting in a tunnel with the finite-discrete element method, and ejected rock fragment velocity was utilized to estimate the released energy during bursting.

Previous rockbursting studies conducted from an energy perspective have mainly focused on estimating the amount of elastic strain energy prior to rock failure and kinetic energy during failure using analytical method from the stress and strain tensors in numerical models (Gu and Ozbay 2014; He et al. 2016b; Vazaios et al. 2019). Although great progress has been made in studying the energy mechanisms of unstable rock failure, the details of energy transformations between different energy components during the rock failure process have still has not been fully explained. The goal of this study is to investigate the energy transformation and indicators of the unstable rock failure in numerical models by integrating energy expressions into the numerical modeling software UDEC. The advantage of this approach is that the magnitudes of different energy components in each zone at every time step can be obtained based on different energy densities and the volume of each zone.

2.3 Energy analysis of rock failure

2.3.1 Energy balance in rock

The energy components considered in this study include elastic strain energy, plastic strain work, total strain energy, and damped kinetic energy; gravitational energy is not considered in this study. Elastic strain energy is one form of stored energy associated with elastic deformation, while plastic strain work and damped kinetic energy are the energy dissipated during rock failure. External work for a volume of rock can result from external forces such as excavation or mining activities, but such work must take the form of elastic strain energy within an intact rock volume
before it can be dissipated (Andrianopoulos and Manolopoulos 2014). Based on the energy methodology proposed by Salamon (1984), the energy balance in Eq. (2.1) summarizes the conversion between external work ($\Delta W$), elastic strain energy ($W_e$), plastic strain work ($W_p$), and damped kinetic energy ($W_d$) when rock failure develops and progresses from one state to another. The left- and right-hand sides of Eq. (2.1) correspond to the states of energy components within a volume of rock before and after rock failure, respectively.

$$\Delta W + W_e + W_p + W_r = W_e' + W_p' + W_d'$$

(2.1)

After rearranging Eq. (2.1), the energy balance during rock failure can be represented as follows:

$$\Delta W + \Delta W_e = \Delta W_p + \Delta W_d$$

(2.2)

In Eq. (2.2), the changes in external work and elastic strain energy in a volume of intact rock is equal to the changes in plastic strain energy and released kinetic energy.

For rock failure, the available energy resource is elastic strain energy through elastic deformation, which also could explain why failure also can be defined as the loss of the ability of the material to store elastic strain energy (Andrianopoulos and Manolopoulos 2014; Chen et al. 2017). Rock is anisotropic and behaves both elastically and inelastically during its failure process. Total strain energy is defined as the sum of elastic strain energy and plastic strain work in a volume of rock, and changes in this total quantity indicate the amount of kinetic energy released during unstable rock failure in this study.

### 2.3.2 Energy densities in intact rock

According to Jaeger et al. (2007), the elastic strain energy density in a volume of rock can be determined by assuming that the rock has an isotropic elastic behavior. The elastic strain energy density of linear isotropic materials undergoing small strains can be written as:

$$U_e = \frac{1}{2E} \left[ \sigma_x^2 + \sigma_y^2 + \sigma_z^2 - 2\nu (\sigma_x \sigma_y + \sigma_x \sigma_z + \sigma_z \sigma_y) 
+ 2(1 + \nu) (\tau_{xy}^2 + \tau_{xz}^2 + \tau_{zy}^2) \right]$$

(2.3)

Under plane strain conditions, the strain in the third direction ($z$ direction) is zero, so the shear stress components $\tau_{xz}$ and $\tau_{zx}$ become zero. Eq. (2.3) can then be rewritten as
When the stress and strain condition of a unit volume of rock changes from one state to another state, then as the change is very small, the increment in total strain energy density \( \Delta U_t \) can be obtained with the average stress tensors and the change in strain tensors.

\[
\Delta U_t = \frac{1}{2} \left[ \left( \sigma_x + \sigma_x' \right) \Delta e_x + \left( \sigma_y + \sigma_y' \right) \Delta e_y + \left( \sigma_z + \sigma_z' \right) \Delta e_z + \left( \tau_{xy} + \tau_{xy}' \right) \Delta \gamma_{xy} + \left( \tau_{xz} + \tau_{xz}' \right) \Delta \gamma_{xz} + \left( \tau_{yz} + \tau_{yz}' \right) \Delta \gamma_{yz} \right]
\]

(2.5)

Under plane strain condition, the strain components \( e_z \), \( \gamma_{xz} \) and \( \gamma_{yz} \) are zero, so Eq. (2.5) can then be rewritten as

\[
\Delta U_t = \frac{1}{2} \left[ \left( \sigma_x + \sigma_x' \right) \Delta e_{xx} + \left( \sigma_y + \sigma_y' \right) \Delta e_{yy} + \left( \tau_{xy} + \tau_{xy}' \right) \Delta \gamma_{xy} \right]
\]

(2.6)

The total strain energy density in a volume of rock can be obtained by adding the increase in total strain energy density \( \Delta U_t \) to its previous total strain energy density \( U_t' \). Before any stress disturbance, total strain energy density is equal to elastic strain energy density at in-situ stress field conditions, so the total strain energy density can be represented as

\[
U_t = U_t' + \Delta U_t
\]

(2.7)

Given the total strain energy density \( U_t \) and elastic strain energy density \( U_e \) in a unit volume of rock, the plastic strain work density \( U_p \) is equal to their difference

\[
U_p = U_t - U_e
\]

(2.8)

Damped kinetic energy is the accumulated amount of instantaneous kinetic energy removed from the system in numerical models, which can be considered as a form of cumulative kinetic energy, and it can be several orders of magnitude larger than the current kinetic energy within a model at any given point of time. It should be noted that the instantaneous kinetic energy will be dissipated by the damping coefficient with solution time in numerical models, which is equivalent to the radiated seismicity in the field (Salamon et al. 2003; Sainoki and Mitri 2014; Gu and Ozbay 2015). The released kinetic energy in numerical models accounts for the monitored energy or seismic energy in underground mines (Poeck 2017; Khademian and Ozbay 2018). In this study, the default local damping coefficient of 0.8 in UDEC was used.
2.4 Model Setup

To study unstable rock failure, the rock properties used during the simulations of UCS tests were obtained from laboratory-based experimental results presented by Zhao and Cai (2015). The granite specimens studied were collected from the Tianhu area in China and were described as Tianhu granite by Zhao and Cai (2015). The geometry, mesh and boundary conditions of the simulated UCS platform are represented in Figure 2.1, where the sizes of the rock specimen, steel platen, and loading beam are 50 mm (width) × 100 mm (height), 54 mm (width) × 4 mm (height) and 54 mm (width) × 100 mm (height), respectively. This simplified platform is utilized to study the contributing factors of unstable rock failure in a UCS test, but both its geometry and parameters cannot be directly compared with the real loading machines in the laboratory. The square zone edge length throughout the entire model is 2 mm, and each square zone has four internal triangular zones.

![Figure 2.1. Geometry, mesh and boundary conditions of the simplified UCS test platform in UDEC.](image)

The bottom of the model was fixed in both the horizontal and vertical directions, but the left and right boundaries of the model were only fixed in the horizontal direction, as shown in Figure 2.1. A constant velocity of 0.002 m/s was applied to the top boundary of the model, and a time step of $2.0 \times 10^{-8}$ seconds was used for all models in this study. UDEC uses the explicit finite-
difference method for numerical analysis, so its solution time cannot be compared with actual physical time. In this case, one second represents fifty million steps due to the time step of $2.0 \times 10^{-8}$ second. Therefore, the numerical loading rate also can be expressed as $4.0 \times 10^{-11}$ m/step, which is sufficiently small to fail the rock specimen in a quasi-static manner. Under the quasi-statically loading condition, the external work resulting from the top boundary can be assumed to be zero during a short period of solution time, such as during the rapid rock failure process. The friction angle of the contact between the rock specimen and the loading platens is assumed to be zero.

The rock properties presented by Manouchehrian and Cai (2015) were used for parameters of the rock model, and the Mohr-Coulomb model with strain-softening behavior was used as the constitutive law for the Tianhu granite specimen (Table 2.1). The stiffness of the steel platens was increased by using a small height and an extremely large elastic modulus of 500 GPa, and the maximum stored elastic strain energy in these two steel platens was just 4 J across all the models in this study. This geometry and elastic modulus of the steel platens are designed to minimize the influence of stored elastic strain energy in the steel platens on rock failure modes.

Table 2.1 Physical and mechanical properties of the Tianhu granite and steel platens [granite data from Zhao and Cai (2015)]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Granite</th>
<th>Platens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2650</td>
<td>7600</td>
</tr>
<tr>
<td>Young’s modulus (GPa)</td>
<td>51</td>
<td>500</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.27</td>
<td>0.30</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>10.2</td>
<td></td>
</tr>
</tbody>
</table>

The post-peak stress-strain behavior of the Tianhu granite was not obtained by Zhao and Cai (Zhao and Cai 2014) in the laboratory. A convex post-peak stress-strain behavior is observed in the curves of the Tianhu Granite, which may not reflect the post-peak stress strain curves of a typical hard rock. Because no data exist for the post-peak behaviors of the Tianhu Granite, the cohesion-weakening parameters of the granite is adjusted to achieve an unstable rock failure and close-to-zero damped kinetic energy during rock failure under LSS of infinity in Table 2.2. The cohesion-weakening parameters make the granite specimen behave more ductile than it should be,
so that rock failure from stable to unstable can be obtained in the rock specimen under different LSS.

Table 2.2 Cohesion-weakening parameters of the strain-softening model for the Tianhu granite specimen.

<table>
<thead>
<tr>
<th>Plastic shear strain</th>
<th>Cohesion yield stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>37.5</td>
</tr>
<tr>
<td>0.001</td>
<td>37</td>
</tr>
<tr>
<td>0.02</td>
<td>32.5</td>
</tr>
<tr>
<td>0.08</td>
<td>25</td>
</tr>
<tr>
<td>0.15</td>
<td>15</td>
</tr>
<tr>
<td>0.25</td>
<td>0.1</td>
</tr>
</tbody>
</table>

2.5 Influence of LSS on rock failure modes

The LSS, determined by elastic modulus and geometry of the loading system, can influence the amount of elastic strain energy stored in the loading system. Since the loading system is simplified into a loading beam in this study, the geometry of the loading system is kept constant. Elastic modulus values of 20 GPa, 30 GPa, 40 GPa and 80 GPa are used for the loading beam to study the influence of LSS on rock failure modes, which can presents the general change from unstable rock failure. The LSS value increases with the elastic modulus of the loading system at the same geometry, so a higher elastic modulus of the loading system represents a larger LSS value in this study.

2.5.1 Physical behavior of the rock with different LSS

The stress-strain curves of the Tianhu granite specimen under a loading system with different elastic modulus values are shown in Figure 2.2, where the axial strain equals the displacement closure of the two ends of the rock divided by the rock specimen length. A convex and ductile post-peak stress-strain curve for the Tianhu granite is obtained with the parameters in Table 2.2. It is expected that the post-peak rock behavior will facilitate the study of LSS on rock failure modes, but not influence the numerical results of indicators and contributing factors of unstable rock failure in this study.
The same stress-strain behavior can be observed before the rock peak strength, but large differences can be seen in the post-peak stress-strain curves. The rock specimen under a stiffer loading system tends to display a steeper post-peak slope, which also has been interpreted as a sign of unstable rock failure in numerical models by some other researchers (Gu and Ozbay 2014, 2015; Manouchehrian and Cai 2015).

![Stress-strain curves of the Tianhu granite specimen under the loading system with different elastic modulus values.](image)

Figure 2.2. Stress-strain curves of the Tianhu granite specimen under the loading system with different elastic modulus values.

In Figure 2.3, the curves of axial stress with solution time illustrate that rapid stress decrease and strain increase can be seen in the rock specimen under the loading systems with elastic modulus values of 20 GPa, 30 GPa and 40 GPa. When the elastic modulus values of the loading system is 80 GPa, no large and vertical axial stress decrease and strain increase from the initiation of rock failure to the total loss of rock strength can be observed, suggesting that the rock failure will not develop and progress without the constant external work from the velocity boundary. A linear increase in axial strain with solution time can be observed in all curves after the loss of rock strength, as the displacement closure between two ends of a rock specimen is equivalent to the product of the constant boundary velocity and the solution time.
2.5.2 Energy transformations

The total strain energy, elastic strain energy, and plastic strain work in different sections of the model can be obtained using energy values calculated in each individual zone. The summation of the energy values in the model enables the energy transformations between the rock specimen and its loading system to be analyzed. In Figure 2.4, the total strain energy represents the total strain energy in the entire model, and the elastic strain energy in LS (loading system) represents the stored elastic strain energy in the loading system. Conversely, the elastic strain energy in rock and the plastic strain work in rock only cover the corresponding energy components in the rock specimen. Since the loading beam is perfectly elastic, the plastic strain work only exists in the rock due to its strain-softening behavior. The total strain energy equals the sum of elastic strain energy in the LS, elastic strain energy in the rock and plastic strain work in the rock.

The elastic strain energy and plastic strain work increase continually prior to peak stress. Large and rapid decreases within elastic strain energy for both rock and loading system can be observed for the elastic modulus values of 20 GPa, 30 GPa and 40 GPa during the rock failure process, while plastic strain work increases simultaneously and rapidly. During this rapid process, part of the stored elastic strain energy is dissipated into kinetic energy in the entire model, which can be seen from the rapid decrease in total strain energy.

Figure 2.3. Stress-strain and stress-time curves of the Tianhu granite specimen under the loading system with different elastic modulus values.
To make a distinction between stable and unstable rock failures in UDEC, a rock failure can be identified as unstable failure when over 50% of elastic strain energy from its peak to residual value within a target rock is released instantaneously during the rock failure process. It should be noted that this criterion is aimed at providing a consistent objective basis to identify the rock failure modes in numerical models. However, the exact numerical threshold adopted was based solely on the author’s judgement. This criterion of distinguishing stable and unstable rock failure with the change of elastic strain energy indicates that the rock failures under the loading system with elastic modulus values of 20 GPa, 30 GPa and 40 GPa are unstable rock failures. For the cases of 80 GPa, the rapid and large change in elastic strain energy in the rock after peak value is less than 50%, indicating this rock failure is stable rock failure. For all cases, the curves of total strain energy will
always merge with curves of the plastic strain work when the rock loses all its strength, while elastic strain energy decreases to nearly zero once the rock specimen has lost its strength. It should be noted that the curve of “elastic strain energy in LS” is identical to that of “elastic strain energy in rock” for the case of 40 GPa.

The maximum elastic strain energy in the rock is the same for all the cases, but the maximum elastic strain energy in the loading system increases with the decreasing elastic modulus values of the loading system. The elastic strain energy in the rock specimen is also an energy source for rock failure, and it can be observed that the plastic strain work increase is larger than the elastic strain energy decrease in the rock for the cases of 20 GPa, 30 GPa and 40 GPa during rock failure. Further, the rapid decrease in total strain energy indicates that part of the elastic strain energy is dissipated into kinetic energy during the unstable rock failure process. Based the magnitude of plastic strain work increase and total strain energy decrease, we can conclude that the rock failure under the loading system with an elastic modulus of 20 GPa has the largest intensity. These four compression tests indicate that the elastic strain energy in the loading system is a significant energy source for plastic strain work and kinetic energy during the rock failure process.

2.6 Influence of rock stiffness on rock failure modes

When a constant velocity is applied directly on top of the steel platen (Figure 2.1), it can be assumed that the UCS platform without the loading beam has an LSS of infinity, to exclude the influence of elastic strain energy within the loading system on rock failure modes. The effective rock stiffness is a function of Young’s modulus and geometry of the rock, and the magnitude of rock stiffness can influence the amount of elastic strain energy stored in the rock specimen. While keeping the rock geometry constant, different Young’s modulus values (20 GPa, 25 GPa, 30 GPa, and 40 GPa) for the Tianhu granite specimen under a LSS of infinity were selected to present the different rock failure modes and evaluate the influence of rock stiffness on the rock failure modes.

2.6.1 Stress-strain behavior of rock with different stiffness

The axial stress-strain curves of the Tianhu granite specimen with different Young’s modulus values are shown in Figure 2.5a, where the LSS is infinity such that the elastic strain energy is only stored in the rock specimen itself. In this study, the axial strain represents the displacement closure between the two ends of the rock specimen divided by its length, which is a
common method to obtain the axial strain in the laboratory. Although rock strength weakening without axial strain increase can be observed for the case of 30 GPa and 40 GPa in Figure 2.5a, changes in strain tensors within the rock just cannot reflect the change of different strain tensors locally within part of the rock specimen. After the peak strength, the rock specimen having a smaller Young’s modulus value tends to have a steeper slope of the stress-strain curve during its strength weakening process.

![Stress-strain curves of the Tianhu granite specimen with different Young’s modulus values under LSS of infinity.](image)

Figure 2.5. Stress-strain curves of the Tianhu granite specimen with different Young’s modulus values under LSS of infinity.

In Figure 2.5b, the rapid decrease in axial stress from its maximum value to residual stress (near zero) can be clearly observed in the curves for modulus values of 20 GPa, 25 GPa and 30 GPa, suggesting these three failures are potential unstable rock failures. For the cases of 40 GPa, the gradual stress decrease from peak stress to the residual stress indicates stable rock failure, which could not have continued without the constant loading velocity on the upper boundary. Different amount of solution time for the rock specimen having different modulus values to reach the same peak strength results from the same constant boundary velocity but different rock stiffness.

### 2.6.2 Energy transformations

The rock stiffness, a function of rock geometry and Young’s modulus, can affect the amount of stored elastic strain energy at rock peak strength. As shown in Figure 2.6, the rock specimen with the modulus value of 20 GPa has the largest maximum elastic strain energy, while
the maximum elastic strain energy is lowest for the case of 40 GPa. A large and rapid decrease in elastic strain energy can be observed for the rock with modulus values of 20 GPa, 25 GPa and 30 GPa, where the plastic strain work increases simultaneously during the same period. The rapid elastic strain energy decrease is about 0.5 kJ, which is less than 50% of the elastic strain energy decrease from peak to residual values, indicating a stable rock failure for the case of 30 GPa. The rapid total strain energy decrease during these two failures indicates that part of the elastic strain energy within the rock is dissipated into kinetic energy. Given that no large and rapid change of elastic strain energy and plastic strain work can be observed for the rock specimen with the modulus value of 40 GPa, this rock failure can be classified as stable rock failure.

Figure 2.6 Curves of energy components with solution time in the Tianhu granite specimen for different values of Young’s modulus with an LSS of infinity.
As Young’s modulus increases from 20 GPa to 40 GPa, the maximum elastic strain energy decreases from 3.0 kJ to 1.5 kJ, while unstable rock failures are only observed in rock with Young’s modulus of 20 GPa and 25 GPa. Larger changes of elastic strain energy, plastic strain work and total strain energy (or damped kinetic energy) can be observed during the rapid rock failure process in the 20 GPa case than in the 25 GPa case, which implies that the rapid changes in these energy components during unstable rock failure also can be used as indicators of rock failure intensity. A rock having a smaller stiffness tends to store a larger amount of elastic strain energy at the same rock peak strength, and the stored elastic strain energy in the rock also can greatly influence the rock failure modes.

2.7 Influence of rock brittleness on rock failure modes

Rock brittleness can be understood as a rock’s ability to resist loading, with increasing deformation during its failure process, which is not an inherent material property. In geotechnical engineering, the slope of the post-peak stress-strain curve can be used as a brittleness index, and this can be controlled by the strain-softening parameters used in numerical models. Taking the Tianhu granite as a baseline, another hypothetical set of strain-softening parameters is tested to promote a larger rock brittleness or steeper slope of the post-peak stress-strain curve in the Tianhu granite, and this case is referred to as Brittle Tianhu granite in this study.

2.7.1 Physical behaviors rock with different brittleness

Cohesion-weakening parameters of the strain-softening model for the Brittle Tianhu granite specimen are listed in Table 2.3, while the rest rock parameters are the same as those of the Tianhu granite specimen as shown in Table 2.1. The only difference between these two rock specimens is the strain-softening behavior of the rock, as represented by the parameters in Table 2.2 and Table 2.3, respectively. By comparing the rock failure behaviors of the Tianhu granite and Brittle Tianhu granite, the influence of rock brittleness on rock failure modes can be analyzed. To exclude the influence of the elastic strain energy within the loading system, the Tianhu granite specimen and Brittle Tianhu granite specimen are loaded to failure under a LSS of infinity.
Table 2.3 Cohesion-weakening parameters of the strain-softening model for the Brittle Tianhu granite specimen.

<table>
<thead>
<tr>
<th>Plastic shear strain</th>
<th>Cohesion yield stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>37.5</td>
</tr>
<tr>
<td>0.0005</td>
<td>37</td>
</tr>
<tr>
<td>0.01</td>
<td>32.5</td>
</tr>
<tr>
<td>0.03</td>
<td>25</td>
</tr>
<tr>
<td>0.05</td>
<td>15</td>
</tr>
<tr>
<td>0.08</td>
<td>0.1</td>
</tr>
</tbody>
</table>

Under LSS of infinity, the axial stress-strain curves of the Tianhu granite and Brittle Tianhu granite are shown in Figure 2.7. The axial stress follows the same path from 0 MPa to 150 MPa with the increase in axial strain in both rocks. After the peak stress, the axial stress in Tianhu granite decreases slowly and gradually with the increase in axial strain, while a rapid stress decrease in Tianhu granite occurs with no perceptible increase in axial strain, is obtained by dividing the displacement closure between the two ends of the rock specimen by its length. In summary, the brittle Tianhu granite is clearly more brittle than the Tianhu granite, based on the slope of the post-peak stress-strain curves.

![Figure 2.7. Stress-strain and stress-time curves of the Tianhu granite and brittle Tianhu granite under LSS of infinity.](image_url)
2.7.2 Energy transformations

The elastic strain energy in the rock is the only stored energy source of dynamic rock failure during a very short timeframe under LSS of infinity, since gravitational energy is not considered in this study. In Figure 2.8, both rock specimens have the same maximum elastic strain energy of 1.2 kJ at the peak strength, when the plastic strain work is close to zero. After the peak point, the elastic strain energy gradually decreases to zero and plastic strain work slowly increases to its maximum value for the Tianhu granite specimen, showing that its failure develops and progresses with solution time. The total strain energy, as the sum of elastic strain energy and plastic strain work in the rock, increases until the residual state of the Tianhu granite specimen is achieved, representing no perceptible released kinetic energy during this rock failure. However, the Brittle Tianhu granite specimen has a rapid decrease in elastic strain energy and increase in plastic strain work immediately after the rock peak strength, resulting from the dissipation of kinetic energy. The changes of these three energy components with solution time indicate that the Tianhu granite failure is a stable rock failure, while the Brittle Tianhu granite failed in an unstable manner.

Figure 2.8. Curves of energy components with solution time in the Tianhu granite and Brittle Tianhu granite specimens under LSS of infinity.

Although both rock specimens have elastic strain energy of 1.2 kJ at the same peak strength, their brittleness, affected by the strain-softening parameters, results in different rock failure modes due to the amount of plastic strain work for the residual state of the rocks. The plastic strain work at residual state is 1.8 kJ for the Tianhu granite and 1.0 kJ for the Brittle Tianhu granite. Clearly, the elastic strain energy is larger than the plastic strain work after residual state for the Brittle
Tianhu granite, while elastic strain energy at rock peak strength is smaller than the magnitude of plastic strain work at residual state for the Tianhu granite. For unstable rock failure to occur, the elastic strain energy should be able to supply enough energy to induce the rock failure spontaneously. The relatively magnitudes of available elastic strain energy before rock failure and plastic strain work at residual state determines the Tianhu granite failure has to be a stable rock failure, while the Brittle Tianhu granite can be an unstable rock failure. Therefore, brittle rock tends to fail violently, for it produces less amount of plastic strain work for the residual state of rock.

2.8 Discussions of numerical results using graphical and numerical methods

The fundamental theory of unstable rock failure considers the LSS and post-peak rock stiffness, whose relative magnitudes were thought to determine the rock failure modes (Cook 1965; Salamon 1970; Manouchehrian and Cai 2015; Xu and Cai 2017a). The post-peak rock stiffness can be represented by the slope of the post-peak stress-strain curves of the rock during its strain-weakening process (Figure 2.9).

In a UCS test, the stress within the contact between the rock and the loading system is always in equilibrium. After the peak stress, the rock begins its strain weakening stage while the loading system goes to the unloading stage. In Figure 2.9, the stress in the stiff loading system is smaller than that in the rock at the strain of $\epsilon_x$, which implies that the strain in the rock cannot increase to $\epsilon_x$ without external loading work. However, the stress in the soft loading system is larger than that in the rock at a strain magnitude of $\epsilon_x$, and therefore this strain magnitude in the rock will continue to increase to the equilibrium point spontaneously. The enclosed area between the curves of soft loading system and post-peak behavior of rock is said to be the amount of released kinetic energy during unstable rock failure (Cook 1965; Salamon 1970, 1984; Rice 1983).
Based on Cook (1965), the enclosed area between the curves of post-peak behavior of a rock and an infinitely stiff loading system should always be zero, which also means unstable rock failure cannot occur in a rock specimen under LSS of infinity. However, unstable rock failures are observed in Brittle Tianhu granite specimen under LSS of infinity and Tianhu granite specimen with Young’s modulus of 20 GPa and 25 GPa under LSS of infinity in this study. The numerical results seem to contradict the fundamental theory of unstable rock failure which is theoretically based on the relative magnitude between post-peak stiffness of rock and the LSS. However, the unstable rock failure theory proposed by Cook (1965) was based on the theoretical assumption that the rock specimen would have the isotropic elastic and plastic strain within the entire rock. In the laboratory, it is not practical to measure the average axial strain values within every zone of a rock specimen. The commonly used axial strain, equivalent to the shortening distance of the rock specimen divided by its length, is not always able to reflect the change of axial strain within the rock specimen. Strain gauges can measure the axial strain on a specimen well before rock failure, but its measurement is localized on the surface of rock specimen.

The average axial strain by zone can be obtained by accessing to axial strain values in each zone of the rock specimen model in UDEC. During the strength-weakening process, elastic strain in some zones begins to decrease while large plastic strain increase will be observed in some localized zones of the rock. The decrease in average axial strain would be observed after the peak
strength if the decrease in average elastic strain was larger than the increase in average plastic strain within the entire rock, while the axial strain obtained with rock specimen shortening divided by rock length cannot decrease with the constant loading velocity from the boundary. Under LSS of infinity and using the average axial strain in each zone, the curves of average stress-strain in the Tianhu granite specimen with Young’s modulus values of 20 GPa, 25 GPa, 30 GPa, and 40 GPa are also obtained and plotted in Figure 2.10.

![Figure 2.10 The curves of average stress-strain in the Tianhu granite specimen with Young’s modulus values of 20 GPa, 25 GPa, 30 GPa and 40 GPa.](image)

With infinite LSS, the curve of the loading system is a vertical line passing through the peak point in each curve, so the failures of the rock specimens having Young’s modulus of 20 GPa and 25 GPa correspond to unstable rock failures. Some area to the left of vertical line passing through the peak point exists for the case of 30 GPa, but its failure modes cannot be observed directly from Figure 2.10. Stable rock failure can be observed directly for the case of 40 GPa. Just from the average stress-strain curve of the 30 GPa case, the rock failure modes for the case of 30 GPa cannot be classified directly in this study. Under LSS of infinity, negative post-peak rock stiffness indicates unstable rock failure while positive post-peak rock stiffness represents stable rock failure. If the curve of loading system intersect with the post-peak stress-strain curve, a second criterion is required to identify the rock failure modes, which is beyond the current research scope. In Figure 2.11, the average stress-strain curves of the Tianhu granite and Brittle Tianhu granite under LSS of infinity also follows the above characteristics.
Figure 2.11 The average stress-strain curves of the Tianhu granite and Brittle Tianhu granite under LSS of infinity.

Unstable rock failure would not occur until the value of LSS is larger than that of post-peak rock stiffness. The relative magnitude between post-peak rock stiffness and LSS will determine whether or not the stored elastic strain energy in the loading system can be spontaneously transferred into the rock specimen and result in unstable rock failure. As shown in Figure 2.10 and Figure 2.11, the post-peak rock stiffness is not a constant value, even if the model geometry is simple. When the LSS is not infinity, the post-peak rock stiffness must consider the geometry and elastic modulus of the loading system. The geometry of loading system and the rock specimen is different in this study, the comparison between the magnitudes of LSS and post-peak rock stiffness of the Tianhu granite rock specimen is not discussed.

Compared with the graphical or theoretical method, the numerical approach developed is able to study unstable rock failure in different model geometries (e.g. fault-slip around a circular excavation). Moreover, the distributions of various energy components such as plastic strain work density within each zone of a large model can also be obtained to benefit our understanding of the underlying energy mechanisms.

2.9 Conclusions

The equations of different energy components, including the elastic strain energy, plastic strain work and total strain energy, were integrated into the UDEC software in this study. This new
energy approach was demonstrated to be capable of tracking the energy transformations between different energy components during rock failure in numerical models, independent of the geometry and complexity of the models. It was found that the rapid stress decrease immediately following the attainment of peak strength can be used as an indicator of potential unstable rock failure in UCS tests. In addition, rapid and large changes in total strain energy, elastic strain energy and plastic strain work during rock failure could be utilized as indicators of unstable rock failure, and the magnitudes of these rapid changes could also represent the unstable rock failure intensity. Unstable failure is based on the criterion that more than 50% of elastic strain energy from its peak to residual value within a target rock should be released instantaneously during the rock failure process. The occurrence of unstable rock failure depends on the available elastic strain energy for plastic strain work during rock failure, while the damped kinetic energy results from the difference between the available elastic strain energy and the induced plastic strain work.

Unstable rock failure tends to occur in a rock with large brittleness and small stiffness under a soft loading system. The elastic strain energy within the entire system is the energy source of plastic strain work and released kinetic energy during unstable rock failure, and it plays a significant role in governing rock failure modes. A larger potential for unstable rock failure in a system with smaller LSS and a rock stiffness can be attributed to the fact that the magnitude of elastic strain energy in the entire model at rock peak strength increases with decreasing rock stiffness and LSS. It also shows that brittle rock requires less elastic strain energy for plastic strain work than ductile rock during the rock failure when other parameters such as rock stiffness and LSS are held constant.

Both graphical and numerical methods can be used to identify the rock failure modes and calculate the released kinetic energy in a UCS test. It is also found that the fundamental theory of unstable rock failure by Cook (1965) and Salamon (1970) is based on the assumption that the rock has the same elastic and plastic strain values within each zone of the entire rock during its strength-weakening process. The energy-based approach in numerical models is independent of model geometries, and can thus provide much more details about different energy components during the rock failure process, providing a unique strength relative to analytical methods.
CHAPTER 3

STUDY OF UNSTABLE FAULT-SLIP FAILURE MECHANISMS THROUGH ANALYSIS OF ENERGY TRANSFORMATIONS IN NUMERICAL MODELS

3.1 Abstract:

Energy components including elastic rock strain energy, joint shear strain energy, joint friction work, and damped kinetic energy can be tracked in numerical models to study the energy transformations that occur during slip failure along pre-existing discontinuities in rock. This approach is applied to study mechanical stability in the context of a direct shear test model considering multiple values of Young’s modulus of the intact rock material, joint shear stiffness of the discontinuity and normal stress acting on the discontinuity. Rapid changes from elastic strain energy and joint shear strain energy to joint friction work and kinetic energy are observed in the cases with lower rock stiffness and higher joint shear stiffness, indicating unstable slip failure. It was found that the increases in both the joint friction work and the damped kinetic energy could be used to quantify the intensity of unstable slip failures. The results of this study suggest that the energy tracking approach applied herein represents a valuable means by which to study potential slip failures that could ultimately be applied to analyze complex mine-scale fault-slip cases.

3.2 Introduction

With the depletion of shallow mineral resources and advances in excavation technology, underground mines and tunnels have been extended to increasingly great depths in recent decades. For example, some infrastructure tunnels have even been excavated at more than 2000 m below the ground surface (Zhang et al. 2012a; Mazaira and Konicek 2015). In these deep mining and civil engineering works, rockbursts represent a major hazard due to high magnitudes of pre-mining stresses (White et al. 2002; Whyatt, J, W. Blake 2002). During a rockburst event, a large amount accumulated energy stored in overstressed rock is rapidly released, resulting in a large volume of rock being violently ejected into underground openings, potentially causing severe damage to underground openings, mine equipment, and/or the safety of workers (Whyatt, J, W. Blake 2002; Kaiser and Cai 2012). As a result, many studies have been conducted to understand rockburst failure mechanisms, and various influencing factors including seismic events, lithologies,
geotechnical properties, and mining activities have been studied both in the field and in numerical models (Kaiser and Cai 2012; Zhang et al. 2012b, 2012a; Gu and Ozbay 2014; Manouchehrian and Cai 2015). In the current study, a hypothetical slip failure along a pre-existing discontinuity was simulated to identify the factors influencing unstable slip failures from an energy perspective. The energy components in both the intact rock material and the discontinuity are tracked to study how the Young’s modulus of the intact rock material and stiffness of the discontinuity influence unstable slip failure, and how the changes of energy components reflect the slip failure intensity. Finally, the energy transformations between stored energy and dissipated energy are compared during unstable slip failures, indicating that this approach has great potential to become an important approach to analyze complex and mine-scale fault-slip cases.

3.2.1 Types of rockburst

Based on the source mechanisms of rockbursts, some research studies have classified these ground failures into five types: strain burst, buckling burst, pillar burst, shear rupture and fault-slip (Ortlepp and Stacey 1994; Ortlepp 2005). Others have considered a “buckling burst” as a sub-class of strain burst, and a “shear rupture” as a sub-class of fault-slip (Müller 1991; Kaiser and Cai 2012). Strain bursts involve the development of fractures through intact rock followed by the violent ejection of rock fragments in the vicinity of openings (Whyatt, J, W. Blake 2002). A pillar burst involves the failure of the loading-carrying capacity of a pillar, though it otherwise has many similarities with a strain burst (White et al. 2002; Whyatt, J, W. Blake 2002). Fault-slip rockburst events result from slip movement along pre-existing discontinuities and/or shear ruptures within intact rock, which could correspond to earthquake-scale fault-slip or sliding of a bedding plane in the vicinity of an opening (Whyatt, J, W. Blake 2002; Sainoki and Mitri 2014). Since rockburst is a complex phenomenon, one rockburst event can be associated with more than one type of classification, such as a slip movement along a bedding plane within a pillar. Furthermore, rock damage associated with a fault-slip event can be either due to slip movement close to excavations or dynamic loading of near-excavation rock resulting from a remote fault-slip event. The involvement of structural weakness planes in rockburst events increases the potential for rockbursts to occur, so considerable research has been conducted to analyze these types of rockbursts (White and Whyatt 1999; Whyatt, J, W. Blake 2002; Ortlepp 2005; Zhou et al. 2015a).
3.2.2 Unstable slip failure along a pre-existing discontinuity

Many studies have been conducted to examine rockburst processes from an energy perspective (He et al. 2010; Kias and Ozbay 2013; Levkovitch et al. 2013; Kaiser and Kim 2014; Fakhimi et al. 2016; Khademian et al. 2016; Cai and Manouchehrian 2017; Kim et al. 2017; Leveille et al. 2017; Xu and Cai 2017b). Rockbursts can be studied from the perspectives of energy release, stress concentration, and/or seismic activity, but all of these fundamentally relate to a change in energy in the rockburst source (intact rock and/or fractures). Cook (1965) and Salamon (1970, 1984) initially hypothesized that in a uniaxial compressive strength (UCS) test, the rock failure mode depends on the stiffness of the loading system and post-peak behavior of the rock sample. A soft loading system combined with a brittle rock sample will lead to the occurrence of unstable rock failure, because the stiffness of loading system impacts the amount of elastic strain energy in the loading system before rock failure, while the rock brittleness will influence the amount of energy dissipated by the rock during its failure. This original concept, while limited to rock free of discontinuities, forms the fundamental basis for a broader consideration of rockmass failure stability from an energetic perspective.

Discontinuities such as faults, bedding planes, and dykes in the vicinity of underground openings can increase the potential for rockbursting (White and Whyatt 1999; Jiang et al. 2010; Manouchehrian 2016). Using an energy-focused approach, Rice (1983) explained the prerequisites of fault-slip by using a one-degree-of-freedom fault model, where slip failure modes depend on the stiffness of the intact rock material and slip-weakening behavior of a fault. In a fault-slip event, the stiffness of the rock material surrounding a discontinuity influences the amount of stored energy before slip failure, while the slip weakening behavior affects the amount of friction work during the slip failure. Gu and Ozbay (2014, 2015) tested this fault-slip hypothesis with a direct double-shear test in numerical models, where stable and unstable slip failures were observed with extremely large and small stiffness of the rock material surrounding a discontinuity, respectively. Sainoki and Mitri (2014, 2016) studied the dynamic behavior of mining induced fault-slip in a mine-scale numerical model, and they concluded that different factors such as mining depth and friction angle of the fault could have a large influence on rockburst severity. Gu and Ozbay (2014, 2015) also simulated the unstable shear failure of a large discontinuity plane within a long-wall coal mine model, and showed mining activities could change the loading stiffness and stress condition of a discontinuity, resulting in an increased likelihood of unstable slip failure.
Manouchehrian and Cai (2015, 2017) used the ejected speed of failed rock and kinetic energy density as indicators to distinguish between stable and unstable rock failures in a continuum numerical model, and also demonstrated that an excavation adjacent to a discontinuity would experience a more violent failure than in a case without the discontinuity.

Outside these works, most other previous studies of rockburst from an energy perspective have mainly focused on estimating the amount of elastic strain energy stored prior to failure using analytical back-analysis methods, which cannot provide information about energy transformations during rock failure (Gu and Ozbay 2014; Xu and Cai 2017a; Khademian and Ozbay 2018). For example, elastic strain energy distributions can be analytically calculated using formulae for underground circular or rectangle excavations (Andrianopoulos and Manolopoulos 2014; He et al. 2016a; Chen et al. 2017). The amount of released energy can also be back-analyzed with the energy theory proposed by Cook and Salamon (Cook 1965; Salamon 1970), who suggested that the area between the curves of rock’s stress-strain behavior and the loading system stiffness was the amount of release energy during unstable rock failure (Cook 1965; Salamon 1970; Sainoki 2014; Khademian et al. 2018). Both of these approaches are limited, however, by the model geometries allowed and their inability to consider the effect of discontinuities on failure processes.

3.2.3 Study scope and contribution

The energy studied in this paper is the mechanical energy stored in and dissipated by intact rock material and a rock discontinuity. The change of mechanical energy reflects both changes in stress and in strain, so any analysis of rockbursting from an energy perspective is by definition considering both stress and strain. Previously, Wang and Kaunda (2019) simulated a hypothetical pillar burst in Universal Distinct Element Code (UDEC) software, and the increments in plastic strain work and damped kinetic energy during unstable rock failure were proposed as indicators of rockburst severity. To calculate the energy transformations that occur during unstable slip failures, FISH code in UDEC is used to implement various energy equations (see section 2) for every zone and contact within the numerical models. Mechanical energy components can be tracked as rockburst severity indicators in numerical models regardless of their geometry and complexity. Energy components in a small volume of rock in a large model also can be obtained, but in this study, only a single discontinuity loaded in a laboratory shear test configuration is considered.
3.3 Energy theory and calculation

3.3.1 Energy balance

The studied energy components in jointed rock include elastic strain energy, joint normal strain energy, joint shear strain energy, joint friction work, and damped kinetic energy in either rock material or joint, if plastic strain work (damage to intact rock) and gravitational energy are neglected (as in this study). Elastic strain energy, joint normal strain energy and joint shear strain energy are forms of stored energy associated with elastic deformation, while joint friction work and damped kinetic energy are the energy dissipated during rock failure by joint shear slip and movement of intact rock zones. External work for a volume of rock can result from external forces such as excavation or mining activities, but such work must take the form of stored elastic strain energy with an intact rock volume or rock discontinuity before it can be dissipated (Andrianopoulos and Manolopoulos 2014). Based on the energy methodology proposed by Salamon (1984), the energy balance in Eq. (3.1) summarizes the conversion of external work (W), elastic strain energy (W_e), joint shear strain energy (W_js), joint normal strain energy (W_jn), joint friction work (W_jf), and damped kinetic energy (W_d) when its failure develops and progresses from one state to another. The left- and right-hand sides of Eq. (3.1) correspond to the states of energy components within a volume of rock before and after rock failure, respectively.

\[ \Delta W + W_e + W_{jn} + W_{js} + W_{jf} + W_d = W_e' + W_{jn}' + W_{js}' + W_{jf}' + W_d' \]  

After rearranging the Eq. (3.1), the energy transformations during rock failure can be represented as follows:

\[ \Delta W + \Delta W_e + \Delta W_{js} + \Delta W_{jn} = \Delta W_{jf} + \Delta W_d \]  

The left- and right-hand sides of Eq. (3.2) are the changes in stored energy and dissipated energy when the rock changes from one state to another.

Elastic strain energy is reversible energy stored in a volume of rock material that may ultimately be transformed to other forms of energy such as joint friction work during slip failure. It should be noted that inelastic damage to the intact rock matrix is not considered in this study, as it was assumed that failure was fully localized in the rock discontinuity; accordingly, the elastic constitutive model was applied to the intact rock material. The shearing of asperities associated
with slip along the joint plane can be represented by the joint properties of the Continuously Yielding (CY) model, in particular the peak and residual friction angles.

In this study, elastic strain energy only refers to the reversible energy in rock material; elastic strain energy stored in a discontinuity is referred to as joint normal strain energy and joint shear strain energy. The joint normal strain energy is not considered further in the simulated direct shear tests, as its magnitude is kept stable by controlling the model geometry and boundary conditions (the details of which can be found in the following section). The energy components associated with shear force and displacement include joint shear strain energy and joint friction work. Joint shear strain energy is elastic and reversible, while joint friction work corresponds to irreversible energy dissipation due to the relative shear displacement along the joint plane.

### 3.3.2 Elastic strain energy and damped kinetic energy with elastic constitutive model

According to Jaeger et al. (2007), the elastic strain energy density in a volume of rock can be determined by assuming that the rock has an isotropic elastic behavior. The elastic strain energy density of linear isotropic materials undergoing small strains can be written as:

$$W_e = \frac{1}{2E} \left[ \sigma_x^2 + \sigma_y^2 + \sigma_z^2 - 2\nu(\sigma_x\sigma_y + \sigma_x\sigma_z + \sigma_z\sigma_y) + 2(1 + \nu)(\tau_{xy}^2 + \tau_{xz}^2 + \tau_{zy}^2) \right]$$

(3.3)

Under plane strain conditions, the strain in the third direction (z direction) is always zero, so the shear stress components $\tau_{xz}$ and $\tau_{xz}$ become zero. Equation (3.3) can then be rewritten as

$$W_e = \frac{1}{2E} \left[ \sigma_x^2 + \sigma_y^2 + \sigma_z^2 - 2\nu(\sigma_x\sigma_y + \sigma_x\sigma_z + \sigma_z\sigma_y) + 2(1 + \nu)\tau_{xy}^2 \right]$$

(3.4)

Damped kinetic energy, also called mass damping work, is the amount of kinetic energy removed from the system, which is heavily influenced by the mass damping coefficient in numerical models (Itasca Consulting Group 2014). In this study, the default local damping coefficient ($\alpha$) of 0.8 in UDEC was used. The damped kinetic energy ($W_{d-gridpoint}$) over a time step ($\Delta t$) at a gridpoint in numerical models is a function of the damping coefficient, the time step and the kinetic energy in the gridpoint ($W_{k-gridpoint}$), which can be expressed as

$$W_{d-gridpoint} = 2 \times \alpha \times \Delta t \times W_{k-gridpoint}$$

(3.5)
The damped kinetic energy is the sum of damped kinetic energy for all gridpoints and time. Therefore, damped kinetic energy can be considered as a kind of cumulative kinetic energy, and it can be several orders of magnitude larger than that of the total kinetic energy within a model at any given point of time. The damped kinetic energy after an equilibrium state has been achieved in a model essentially accounts for the monitored release energy or seismic energy in underground mines (Poeck 2017; Khademian and Ozbay 2018; Khademian et al. 2018).

3.3.3 Joint shear strain energy and joint friction work with CY joint model

The CY joint model was originally developed to simulate the mechanism of progressive damage of joints under continued shearing (Cundall and Lemos 1990). It accounts for some nonlinear behavior observed in laboratory tests, which include joint shear damage, normal stiffness dependence on normal stress, and a decrease in dilation angle with plastic shear displacement (Itasca Consulting Group 2014). The joint shear strength in CY joint model is influenced by normal stress and plastic shear displacement, meaning it accounts for non-linear post-peak softening behavior. The dilation angle is assumed to be equal to the difference between the current friction angle and residual friction angle, where the current friction angle is set to the peak friction angle before any plastic shear displacement occurs.

The joint normal stiffness \(k_n\) of a contact element controls the normal displacement between two blocks, and the joint shear stiffness \(k_s\) determines the portion of elastic shear displacement out of the total shear displacement between two blocks. In addition, a contact element in a discontinuity exhibits elastic behavior in its normal direction and elastic-plastic behavior along its shear direction.

In Figure 3.1, the current friction angle \(\phi\), initially set as peak friction angle \(\phi_{\text{peak}}\), gradually decreases with the continued accumulation of plastic shear displacement in the discontinuity. Plastic shear displacement in a contact element will be triggered once the shear stress \(\tau_s\) reaches a certain magnitude during the shear loading process under the normal stress \(\sigma_{\text{normal}}\). When the induced shear stress is larger than the weakened shear strength \(\tau_{\text{target}}\), the discontinuity will enter the softening stage, exhibiting a shear strength weakening behavior. The current friction angle gradually decreases to the residual friction angle with the continued accumulation of plastic shear displacement in the joint. Several researchers have simulated joint
slip failure using the CY joint model in UDEC (e.g. Gu and Ozbay 2014, 2015; Khademian et al. 2018).

Figure 3.1 Flowchart of the interactions between plastic shear displacement and friction angle.

The change of plastic shear displacement is controlled by a yield factor ($F$), which is a function of the current shear stress and “target” shear strength in the CY joint model. A detailed description and formulations of the yield factor can be found in a previous publication (Cundall and Lemos 1990). The change of plastic shear displacement ($\Delta u_s^p$) can be defined with respect to the yield factor ($F$) and the change of total shear displacement ($\Delta u_s$), which therefore also defines the change of elastic shear displacement ($\Delta u_s^e$):

$$\Delta u_s^e = F \times \Delta u_s \tag{3.6}$$

$$\Delta u_s^p = (1 - F) \times \Delta u_s \tag{3.7}$$

Based on equations (3.6) and (3.7), the change of joint shear strain energy ($\Delta W_{js}$) can be expressed as the product of the average joint shear force ($\bar{f}_s$) and the corresponding change of elastic shear displacement ($\Delta u_s^e$) – see equation (3.8). Similarly, the change of joint friction work ($\Delta U_{jf}$) is equal to the average joint shear force ($\bar{f}_s$) multiplied by the change of plastic shear displacement ($\Delta u_s^p$) – see equation (3.9).
\[ \Delta W_{js} = -\bar{f}_s \times \Delta u_s^e = -F \times \bar{f}_s \times \Delta u_s \]  
\[ \Delta W_{jf} = -\bar{f}_s \times \Delta u_s^p = -(1 - F) \times \bar{f}_s \times \Delta u_s \]  

The yield factor also can be interpreted as the percentage of total shear displacement that is elastic in nature – see equation (3.6). Initially, the yield factor is one before the onset of plastic shear displacement. As the yield factor decreases from one to zero due to the increase in plastic shear displacement, the rates of change of joint shear strain energy and joint friction work will decrease and increase, respectively. The maximum joint shear strain energy exists at a joint’s peak strength, as the yield factor is zero at this point. The yield factor becomes negative after the joint’s peak strength, since the relative plastic shear displacement is larger than total shear displacement during the shear strength weakening process, resulting in a decrease of joint shear strain energy. Finally, the yield factor becomes zero again when the joint reaches a constant residual strength and plastic shear displacement is equal to total shear displacement, resulting in a constant magnitude of joint shear strain energy from this point onwards.

By integrating the formulae for energy increments presented in this section, the energy transformations within a rockmass, including the energy in both intact rock material and joint planes, can be analyzed by tracking the change of energy components during failure in UDEC.

### 3.4 Model setup

Slip failure along a pre-existing discontinuity can result from a decrease in normal stress or an increase in shear stress, and many factors can affect the stress distribution across a discontinuity, such as excavation advance, blasting vibration and water effects. A hypothetical and simplified slip failure induced by an increase in shear stress along a discontinuity is studied, where the discontinuity is assumed to be highly persistent and to exist in a high stress condition, as is common in many deep mining scenarios (Figure 3.2). The two planes of the discontinuity move horizontally in opposite directions corresponding to the occurrence of discontinuity slip, and the bed rock of the discontinuity is assumed to be good quality brittle rock such as granite. Because the simulated discontinuity is embedded in stiff hard rock at depth, the slip failure can be assumed to occur with a constant normal stiffness boundary condition, and no vertical displacement or
rotation of the rock surrounding the discontinuity is allowed (Bewick et al. 2014; Thirukumaran et al. 2016).

This simplified numerical model is based on a hypothetical slip failure that might occur in an in-situ setting. Accordingly, the numerical model is set to be 1 m in height and 10 m in width, and is effectively of infinite length in the out of plane direction due to the plane strain condition used. The entire model is discretized using a square mesh with 0.05 m edge length, and each square zone has four internal triangular zones. Hydrostatic in-situ field stress of 20 MPa roughly corresponding to a depth of 800 meters is assigned to the base model. This depth is usually the onset depth of mines experiencing rockburst hazards, and a much wider range of depths are selected to investigate the influence of normal stress on slip failure modes as described in Section 3.7. With the internal isotropic zone stress set to 20 MPa, normal stresses of 20 MPa are applied to the left and right boundaries, but the top and bottom boundaries are fixed in the vertical direction (effectively an infinite normal stiffness boundary). This results in a normal stress of 20 MPa in the normal direction of the discontinuity at initial equilibrium of the numerical model prior to the application of any displacements at the boundaries.

A constant horizontal velocity of $2.5 \times 10^{-4}$ m/s was applied to both the top and bottom boundaries of the model, and a time step of $2.0 \times 10^{-7}$ seconds was used. Although this would be
a relatively large shear rate in a laboratory shear test, one must recall that the solution time in numerical models is different from actual physical time. In this case, one second represents five million steps due to the time step of $2.0 \times 10^{-7}$ second. Therefore, the total shear rate can be expressed as $5.0 \times 10^{-11}$ m/step, which is sufficiently small to force the shear stress in the discontinuity to its peak and residual strengths in a quasi-static manner.

The fixed boundary on the top and bottom boundaries in vertical direction minimizes the moment influence in the numerical model to keep a small variation of normal stress distribution in the discontinuity. The large width-height ratio of the model is also designed to reduce the moment influence on the stress distribution in the discontinuity. The moment arm is equal to half of the model height, and the force results from the constant horizontal velocity on the top and bottom boundary. The moment can cause variations of normal stress distribution in the discontinuity, but this effect has been minimized with the above boundary conditions and model geometry. Moreover, the change of joint normal strain energy during rapid slip failure is minimal and more than one magnitude smaller than all other energy components, so the joint normal strain energy is assumed constant and negligible for a single discontinuity loaded in a laboratory shear test configuration.

The rock material is simulated with an elastic constitutive model such that slip failure will occur fully along the pre-existing discontinuity. To simulate a “good quality hard rockmass” such as granite, the Poisson’s ratio was set to 0.2, and the Young’s modulus was set to 40 GPa (Hoek and Brown 1997). The CY joint model input parameters used are listed in Table 3.1. The selection of joint roughness and stiffness is based on some previous research conducted using the CY joint model in UDEC (Gu and Ozbay 2014, 2015; Khademian and Ozbay 2018). The joint normal stiffness can be several times larger than the joint shear stiffness, and their ratio depends on numerous factors including the host rock’s Poisson’s ratio and Young’s modulus (Barton 1972; Bandis et al. 1983; Małkowski 2015). Because the rock material is simulated as elastic and joint infilling is assumed to minimal, a theoretical joint normal and shear stiffness as high as 100 GPa/m can be considered reasonable in deep hard rockmass (Wines and Lilly 2003). In addition, joint shear stiffnesses from 50 GPa/m to 150 GPa/m are also simulated in the Section 3.6. Since it is already well understood that increasing discontinuity brittleness (e.g. as influenced by the peak to residual friction angle drop) contributes to potential instability of slip failures, the friction angle values were held constant in this study (Salamon 1974; Rice 1983; Linker and Dieterich 2008; Gu and Ozbay 2014; Khademian and Ozbay 2018).
Table 3.1 Parameters of the CY joint model for simulating the slip failure along discontinuity.

<table>
<thead>
<tr>
<th>Parameter symbols</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint normal stiffness</td>
<td>100 GPa/m</td>
</tr>
<tr>
<td>Joint shear stiffness</td>
<td>100 GPa/m</td>
</tr>
<tr>
<td>Joint initial friction angle</td>
<td>55°</td>
</tr>
<tr>
<td>Joint intrinsic friction angle</td>
<td>25°</td>
</tr>
<tr>
<td>Joint roughness</td>
<td>0.1 mm</td>
</tr>
</tbody>
</table>

Energy components in each zone and discontinuity contact of the numerical model are calculated at every time step, but are only stored every 1000 time steps to reduce the number of output points for plotting purposes. Since peak strength is attained after tens of millions of time steps in the presented models, this data capture rate is sufficient. If only a few data points are recorded during the slip failure period in a given model, it means that the slip failure occurs relatively rapidly, considering the tens of thousands of recorded points output from each model.

3.5 Slip failure modes of the discontinuity influenced by rock stiffness

The magnitude of rock stiffness can greatly influence the amount of elastic strain energy stored in intact rock material, where the effective rock stiffness is governed by the Young’s modulus and geometry of the rock. Because specific model geometry is also utilized to reduce the moment influence on the normal stress distribution in the discontinuity, the model geometry of the rock is held constant in this study.

Rock exposed to a specific force condition will have a larger magnitude of elastic strain energy with smaller Young's modulus. Accordingly, different Young’s modulus values for the intact rock material surrounding the discontinuity were tested: 10 GPa, 20 GPa, 40 GPa, 80 GPa, and 160 GPa. This wide range of Young’s modulus values is utilized to study the influence of stored elastic strain energy on rock failure modes, although it is acknowledged that the Young’s modulus of 160 GPa is unrealistically high, even for the stiffest hard rocks.

3.5.1 Shear stress and displacement behavior

The shear stress-displacement curves of the discontinuity surrounded by rock material with different stiffness are shown in Figure 3.3. The only difference between the different models is the Young’s modulus of the rock material, so the shear stress roughly follows the same path from 0
MPa at to 13 MPa with the corresponding increase in shear displacement in the discontinuity. The slightly different peak shear strength is believed to result from the clockwise moment, which is induced by the horizontal velocity condition on the top and bottom boundaries. Associated with these slightly different peak shear strength values, small variations of residual strength of the discontinuity also can be observed, although these minor differences do not affect the findings of the sensitivity analysis presented in this study.

Figure 3.3 Shear stress-displacement curves of the discontinuity embedded in rock materials with different Young’s modulus values.

For modulus values of 80 GPa or 160 GPa, the shear stress decreases slowly and gradually with the increase in shear displacement, which can be regarded as the characteristic behavior of the discontinuity. Linear post-peak shear stress-displacement behavior can be clearly observed through the density of markers for modulus values of 10 GPa, 20 GPa, and 40 GPa, and this type of deviation has previously been recognized as an indicator of unstable slip failure in numerical models by other researchers (Gu and Ozbay 2014; Manouchehrian and Cai 2015). The discontinuity embedded in less stiff material tends to have a longer linear section after its peak strength. More details of the shear stress-displacement curves can be interpreted by plotting the curves of shear stress and displacement as a function of solution time (Figure 3.4), where the time interval between every two adjacent markers represents 1000 time steps in the numerical models. It should be noted that it takes different numbers of solution steps (and therefore different amounts of solution time) for the discontinuity to reach its peak strength in different models.
The small number of markers present as the stress-time curve drops from 13 MPa to 10 MPa for modulus values of 10 GPa, 20 GPa, and 40 GPa, matrix Young’s modulus values indicates that the shear stress reduction occurs within a very short time period. No obvious rapid slip failure can be observed for modulus values of 80 GPa and 160 GPa, and the failure is a gradual and stable process resulting from the continuous loading from the boundary. Similar behaviors can also be observed in the curves of shear displacement with solution time in Figure 3.4. A rapid change in shear displacement is shown by the markers for modulus values of 10 GPa, 20 GPa, and 40 GPa, while no such behavior occurs for the stiffer rock matrix cases. All these curves are roughly parallel to each other after the slip failure, because the models have approximately the same residual shear strength and same boundary velocity of \(2.5 \times 10^{-4} \text{ m/s}\) in the opposite directions from the top and bottom boundaries.

The rapid decrease in shear stress and increase in shear displacement after discontinuity’s peak strength can be utilized as indicators of potential unstable slip failure in numerical models (Gu and Ozbay 2014, 2015), suggesting a higher unstable failure potential in the cases with the softer rock matrix.

### 3.5.2 Energy storage and dissipation during dynamic slip failure

By summing the energy values in each zone and discontinuity contact, the total amount of any energy component within the entire model can be obtained such that the energy
transformations between different energy components can be studied. The potential stored energy for the dynamic slip failures observed is the elastic strain energy and joint shear strain energy in the rock and discontinuity, respectively, since gravitational energy and joint normal strain energy are not considered in this study, and the boundary work over the short timeframe during which slip occurs is negligible.

In Figure 3.5, the initial magnitude of elastic strain energy results from the isotropic initial stress of 20 MPa in numerical models. The post-peak decrease in elastic strain energy is approximately 100 kJ for the 10 GPa Young’s modulus rock material, when the stored elastic strain energy was approximately 570 kJ before the occurrence of slip failure. When the Young’s modulus of the rock material is 80 GPa and 160 GPa, the maximum stored elastic strain energy values are approximately 80 kJ and 40 kJ, respectively, which are both smaller than even the decrease in elastic strain energy in the case with the softest rock material. Since the input parameters of the discontinuity are the same for all the models, the discontinuity has similar maximum joint shear strain energy values of around 9 kJ at its peak shear strength and 4.5 kJ at its residual strength in all cases. Large and rapid decreases of elastic strain energy and joint shear strain energy can be observed for modulus values of 10 GPa, 20 GPa and 40 GPa after the peak points, indicating the stored energy is released rapidly with the weakening of discontinuity shear strength.

To make a distinction between stable and unstable slip failures along discontinuities, a slip failure can be identified as unstable failure when over 50% of joint shear strain energy from its peak to residual value within a discontinuity is released instantaneously during the slip failure process. It should be noted that this criterion is aimed to provide a consistent objective basis to identify the rock failure modes in numerical models. However, the exact numerical threshold adopted was based solely on the author’s judgement. For modulus values of 80 GPa and 160 GPa, No large and rapid joint shear strain energy more than 50% of the difference between the peak and residual values of joint shear strain energy is observed during the shear strength softening process, which indicate that these two rock failures are unstable slip failures.
The energy dissipated during slip failure includes joint friction work and damped kinetic energy in the rock, given that no inelastic energy dissipation in the intact rock material is considered. Joint friction work results from the relative shear displacement within the two walls of the discontinuity. The amount of damped kinetic energy is a cumulative measure that is monotonically increasing as stored energy is converted into kinetic energy during the loading process and then numerically dissipated to ensure a quasi-static model condition.

In Figure 3.6, a rapid increase in joint friction work and damped kinetic energy is observed for modulus values of 10 GPa, 20 GPa and 40 GPa, while no such change is observed for modulus values of 80 GPa and 160 GPa. These curves of joint friction work as a function of solution time have a very similar trend to the shear displacement-time curves shown in Figure 3.4; this makes sense, since joint friction work is induced by relative shear displacement. The rapid joint friction work increase during failure can be as much as 83 kJ (i.e. the 10 GPa modulus case), where the relatively high rate of the friction work can be observed from the small number of recorded points during slip failure. The rapid joint friction work increase is approximately 18.7 kJ for the modulus value of 40 GPa, and no perceptible increase occurs for modulus values of 80 GPa or 160 GPa. The damped kinetic energy stays approximately constant before and after the slip failure in all cases, while a rapid increase in damped kinetic energy can be observed during slip failure in the cases with softer rock material. Therefore, increases in joint friction work and damped kinetic energy.
energy can be utilized as indicators of slip failure intensity, as larger energy dissipation magnitudes indicate more stored energy is released during rock failures.

Figure 3.6 Changes in joint friction work-time and damped kinetic energy with solution time in models with different Young’s modulus values.

As described above, the changes of energy components with solution time are indicators of potential instability that are consistent with previously proposed and more conventional approaches for studying unstable rock failure under simple geometric or loading conditions (e.g. the appearance of the shear stress-displacement curve). Unlike some of the previously proposed approaches, however, the energy approach presented here is independent of the geometry and complexity of the models. This means that the approach used here could be extended to understand the broader spatial trends in energy change at the mine scale, whereas approaches developed specifically for a single discontinuity loaded in a laboratory shear test configuration could not necessarily.

3.5.3 Energy balance during slip failure

The stored energy for slip failure includes elastic strain energy and joint shear strain energy, while the dissipated energy consists of joint friction work and damped kinetic energy in this study. While elastic strain energy and joint shear strain energy decrease over the course of slip failure, joint friction work and damped kinetic energy correspondingly increase. The quasi-statical loading condition can slowly and gradually provide elastic strain energy into the numerical model with
solution time, but the amount of elastic strain energy induced by the boundary work over the short timeframe during which slip occurs is negligible.

To obtain the energy changes during the rapid slip failures, the difference between every ten adjacent recorded points among the tens of thousands of recorded points in each of these energy curves was calculated, and the maximum or minimum result from each curve corresponds to the rapid increase or decrease of each energy component during slip failures. The time window for every ten adjacent recorded points is $2.0 \times 10^{-6}$ seconds due to the time step of $2.0 \times 10^{-7}$ seconds. Since the velocity boundary can be assumed to be a “static” loading condition within a very short period, the stored energy decrement will always equals to dissipated energy increment when the loading work from the boundary is assumed zero. The selection of the $2.0 \times 10^{-6}$ second time window was based on a visual inspection of zoomed-in images of the energy component versus time curves; in all cases, the selected time window was found to approximately correspond to the range of time over which there was a perceptible change in the density of recorded points in the energy curves. Although this approach admittedly is somewhat imprecise, the rate of energy component change outside the time window where slip failure occurs is small enough that the errors introduced into the calculated energy values are minimal. This reflected in the relatively small scale of the discrepancies in the energy balance calculations presented as follows.

The changes of each energy component during the slip failures are shown in Table 3.2. The elastic strain energy change ($\Delta W_e$) and joint shear strain energy change ($\Delta W_{js}$) are labeled as negative (loss of stored energy) while the joint friction work change ($\Delta W_{jf}$) and damped kinetic energy change ($\Delta W_d$) are positive (increase in dissipated energy). The percentage difference between the stored energy decrease and dissipated energy increase in the system in each case can be regarded as a representation of the error in the assumed energy balance. Considering the quasi-static loading boundary conditions and neglected normal stress variations in the discontinuity, the maximum discrepancy between these energy components is just 2%, illustrating that the overall energy balance approach considered is reliable.
Table 3.2 The changes in energy components in numerical models with different Young’s modulus values during slip failures.

<table>
<thead>
<tr>
<th>E (GPa)</th>
<th>ΔW_c (J)</th>
<th>ΔW_{js} (J)</th>
<th>ΔW_{jt} (J)</th>
<th>ΔW_d (J)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>-91100</td>
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<td>84300</td>
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</tr>
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<td>160</td>
<td>-20</td>
<td>-26</td>
<td>46</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

3.6 The influence of joint shear stiffness (and joint roughness) on discontinuity failure stability

In the CY model in UDEC, slip failure along discontinuity is controlled by the initial joint friction angle, the intrinsic joint friction angle, the joint shear stiffness and the joint (surface) roughness. In this simulated direct shear test, the joint normal stiffness has minimal influence on the change of energy during rapid slip failures, so the joint normal stiffness is kept at 100 GPa/m. The joint shear stiffness is defined as the ratio of joint shear stress to shear displacement in the discontinuity, and its magnitude can influence the peak shear strength in the CY joint model. Joint roughness represents the height of joint asperities, and it significantly influences post-peak joint shear stress-displacement behavior. When the joint roughness is larger, the weakening of shear strength occurs over a larger range of post-peak shear displacements.

Changing the joint shear stiffness will influence the peak shear strength, but peak shear strength can directly influence the amount of elastic strain energy in the rock material. Therefore, joint shear stiffness and joint roughness were adjusted concurrently to obtain the same peak shear strength to study the combined influence of joint shear stiffness and joint roughness on rock failure modes at the same time. While keeping the Young’s modulus of the rock material at 40 GPa, the joint shear stiffness values of 50 GPa/m, 75 GPa/m, 100 GPa/m, 125 GPa/m, and 150 GPa/m were tested, with corresponding CY joint roughness values of 0.2 mm, 0.13 mm, 0.1 mm, 0.08 mm and 0.067 mm, respectively.
3.6.1 Shear stress and displacement behaviors

The shear stress-displacement curves of the discontinuity with different joint shear stiffness values are illustrated in Figure 3.7. Similar peak and residual shear strengths can be observed in all cases. Based on the density of recorded points in these curves, we can observe that rapid slip failure is more likely to occur in the discontinuities with higher joint stiffness (i.e. low roughness), when they have the same peak strength.

![Shear stress-displacement curves](image)

Figure 3.7 Shear stress-displacement curves of the discontinuity with different joint shear stiffness values.

The changes in shear stress and displacement with solution time are plotted, and a rapid decrease in shear stress and a corresponding increase in shear displacement can be clearly seen in these curves (Figure 3.8). More rapid drops in shear stress can be seen to occur for higher joint stiffness (low roughness) cases. Even in the lowest joint stiffness or highest roughness case, however, a small vertical section of the shear stress with solution time curve can be observed. This suggests that the difference between stable slip failure and unstable slip failure may be a continuous one, such that some cases (e.g. joint shear stiffness in the 50 to 75 GPa/m range) can be considered as transitional (Manouchehrian and Cai 2015). Even the 75 GPa/m and 100 GPa/m cases only show a partial discontinuity strength loss during rapid slip (unlike the 125 GPa/m and 150 GPa/m cases, which definitively correspond to unstable behavior due to the fact that the full peak-residual strength drop occurs during rapid slip).
3.6.2 Stored energy and energy dissipation during dynamic slip failure

At a solution time of zero, the same magnitudes of elastic strain energy (~90 kJ) and joint shear strain energy (0 kJ) in all models results from the in-situ stress of 20 MPa and zero initial shear stress in the models, respectively (see Figure 3.9). A maximum elastic strain energy of approximately 145 kJ can be observed in all cases, and the minor discrepancies that exist are caused by the minor variations of the peak shear strength as shown in Figure 3.8. After the attainment of peak strength, a rapid decrease in elastic strain energy can be observed, especially when the discontinuity is very stiff.

Unlike elastic strain energy, very different maximum joint shear strain energy values are observed, though the discontinuity in all cases has similar peak strength. Based on Hooke’s law, the joint shear strain is not only a function of joint stiffness ($k_s$) and elastic shear displacement ($\Delta u_s$), but a function of shear force ($f_s$) and joint shear stiffness ($k_s$):

$$\Delta W_{js} = \frac{1}{2} \times k_s \times \Delta u_s^2 = \frac{1}{2} \times \frac{f_s^2}{k_s}$$

(3.10)

Based on Equation (3.10), the discontinuity having larger joint stiffness will have less joint shear strain energy at the same shear stress. As can be seen in Figure 3.9, the discontinuity with stiffness of 50 GPa/m has the largest maximum joint shear strain energy, while the joint shear strain energy is smallest for the joint shear stiffness value of 150 GPa/m. A rapid decrease in joint
shear strain energy can be observed in all these five curves, though it is not very obvious for the modulus value of 50 GPa/m. Among these five cases, it is interesting that the largest joint shear strain energy decrease occurs in the discontinuity with a joint stiffness of 100 GPa/m during the rapid slip failures. Based on the criterion that unstable slip failure should have an instantaneous joint shear strain energy more than the 50% of the difference between peak and residual value of the joint shear strain energy, only the discontinuity with modulus value of 50 GPa/m has a stable slip failure.

![Figure 3.9 Change of elastic strain energy and joint shear strain energy with solution time in models with different joint stiffness values.](image)

The dissipated energy includes joint friction work and damped kinetic energy, which results from the decrease of stored energy during slip failure. The changes in joint friction work and damped kinetic energy as a function of solution time are plotted in Figure 3.10. It can be clearly seen that the joint friction work represents the main part of the dissipated energy, and distinct indications of slip failure can be clearly identified in the curves of joint friction work and damped kinetic energy as functions of solution time. After the slip failure, the damped kinetic energy remains constant, indicating the quasi-static boundary condition induces nearly no kinetic energy in the models. Due to the opposite loading velocity from the boundary, a gradual increase in joint friction work with solution time can be observed even after the slip failures in the discontinuities.
Figure 3.10 Change of joint friction work and damped kinetic energy with solution time in models with different joint stiffness values.

### 3.6.3 Energy balance during rapid slip failure

Similarly to the previous cases where various Young’s modulus values were tested, the differences in each of the energy components between every ten adjacent points in each of these energy curves was calculated to identify the magnitude of change in each of the energy components during slip failure. The resulting values are listed in Table 3.3.

<table>
<thead>
<tr>
<th>Joint Stiffness (GPa/m)</th>
<th>ΔW_e (J)</th>
<th>ΔW_{js} (J)</th>
<th>ΔW_{jf} (J)</th>
<th>ΔW_d (J)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>-1860</td>
<td>-815</td>
<td>2700</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>75</td>
<td>-11100</td>
<td>-3160</td>
<td>14200</td>
<td>214</td>
<td>1</td>
</tr>
<tr>
<td>100</td>
<td>-16200</td>
<td>-3240</td>
<td>19000</td>
<td>673</td>
<td>1</td>
</tr>
<tr>
<td>125</td>
<td>-19000</td>
<td>-2940</td>
<td>21010</td>
<td>1130</td>
<td>1</td>
</tr>
<tr>
<td>150</td>
<td>-20700</td>
<td>-2610</td>
<td>22100</td>
<td>1520</td>
<td>1</td>
</tr>
</tbody>
</table>

As the joint shear stiffness increases, the elastic strain energy drop during rapid slip failure increases from 1860 J to 20720 J, but the joint shear strain energy change does not show a consistent trend (as discussed with respect to Figure 3.9). With respect to dissipated energy, the changes in joint friction work and damped kinetic energy both increase as a function of the joint stiffness.
stiffness, but the damped kinetic energy increases are much smaller than the joint friction work increases. The maximum difference between stored energy and dissipated energy is 1%, again confirming the overall reliability of the energy balance calculation.

As joint stiffness increases and joint roughness decreases for the discontinuities having the same peak strength, shear strength will decrease more rapidly with an increase in joint shear displacement, which means less joint friction work increase will be induced with the same magnitude of shear displacement (Figure 3.8). Even though the joint may have more joint shear strain energy at the same peak strength when joint stiffness is smaller, a larger joint roughness will greatly increase the joint shear resistance that occurs during rapid slip failure. The dissipated energy during rapid slip failure increases from 3 kJ to 26 kJ as the joint shear stiffness increases from 50 GPa/m to 150 GPa/m and the joint roughness correspondingly decreases from 0.2 mm to 0.067 mm; this again indicates that the magnitude of dissipated energy can be used to quantify the severity of unstable rock discontinuity slip failure.

3.7 Slip failure modes of the discontinuity influenced by normal stress

Normal stress is also a significant parameter that can greatly influence the stability of discontinuity slip failure. Keeping the joint normal and shear stiffness both as 100 GPa/m and the Young’s modulus of the rock material as 40 GPa, a wide range of hydrostatic in-situ stress values (from 5 to 80 MPa) were tested.

3.7.1 Shear stress-displacement behavior

The shear stress-displacement curves of the discontinuity under different normal stress conditions are shown in Figure 3.11. Both the peak and residual shear strengths increase with the normal stress in the discontinuities, and the magnitudes of shear displacements at the peak shear strength also increase with normal stress. Based on the density of recorded points in these curves, rapid slip failures appear to occur for the discontinuities cases with normal stresses of 10 MPa, 20 MPa, 30 MPa, 40 MPa and 60 MPa. The largest rapid shear stress decrease and shear displacement increase appears to occur for normal stresses of 20 MPa to 40 MPa, while the lowest (5 MPa) and highest (80 MPa) confining stress cases show no indications of rapid failure.
Figure 3.11 Shear stress-displacement curves of the discontinuity under different normal stress values.

Overall, these results suggest that the likelihood and severity of potentially unstable slip failures increase first as confinement increases, then subsequently decreases. A direct shear strength test on a Hungarian granite specimen along discontinuities was carried out in laboratory by Buocz et al. (2014), and the measured change of shear stress with shear displacement under different normal force conditions also confirmed that the larger difference between peak and residual shear strength appears at a moderate normal stress condition (Figure 3.12). The shear stress-displacement curve of the Hungarian granite specimen under different normal stress conditions was plotted in Figure 4, where the normal force values of 0.5 kN, 1 kN, 2 kN and 3 kN were applied on the rock specimen sequentially. As could be seen from Figure 3.12, the shear stress reduction from peak strength to residual strength increased firstly from normal force of 0.5 kN to 2kN, then little difference was observed when the normal force was extended to 3 kN.

We can consider this trend in terms of the different mechanisms of deformation of the joint plane at different normal stresses (Logan and Teufel 1986; Porter et al. 2013): at low normal stress, deformation will be dominated by sliding over asperities (leading to large amounts of dilation); at moderate normal stress, deformation consists of a combination of dilation and brittle shear through asperities; at high normal stress, deformation is non-dilative, and semi-ductile shear through asperities occurs. At low normal stress, there is not enough stress in the system to allow for violent rupture through asperities, whereas a high normal stress, the overall discontinuity behavior is too
ductile (minimal strength loss) to allow for unstable slip to occur; accordingly, the most unstable condition occurs under moderate normal stress.

![Shear stress-displacement curve](image)

Figure 3.12 Shear stress-displacement curve of Hungarian granite specimen under different normal stress conditions (after Buocz et al. 2014).

### 3.7.2 Energy balance during rapid slip failure

Similarly to the previous cases, the differences in each of the energy components between every ten adjacent points in each of these energy curves was calculated to identify the magnitude of change in each of the energy components during slip failure. By considering the individual energy components listed in Table 3.4, we can get a quantitative sense of the relative energy release associated with sudden post-peak slip in the different normal stress cases. As the joint normal stress increases, the elastic strain energy decrease during rapid slip failure increases from 540 J to 27130 J, but it then decreases to 390 J when the normal stress is larger than 40 MPa. The same trend can be observed for the other energy components as well.

At low normal stresses, not enough energy is stored in the system to cause unstable slip failure. Conversely, although the stored energy is high at peak strength for the highest normal stress case, only a small percentage of that energy is dissipated through slip failure due to the relatively low peak-to-residual strength difference. Accordingly, the models with moderate normal stress have the highest total energy release during slip failure.
Table 3.4. Changes in energy components during slip failure for different normal stress values.

<table>
<thead>
<tr>
<th>Normal stress (MPa)</th>
<th>$\Delta W_e$ (J)</th>
<th>$\Delta W_{js}$ (J)</th>
<th>$\Delta W_{jf}$ (J)</th>
<th>$\Delta W_d$ (J)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>-536</td>
<td>-125</td>
<td>662</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>10</td>
<td>-5730</td>
<td>-1130</td>
<td>6760</td>
<td>156</td>
<td>1</td>
</tr>
<tr>
<td>20</td>
<td>-16160</td>
<td>-3240</td>
<td>18960</td>
<td>673</td>
<td>1</td>
</tr>
<tr>
<td>30</td>
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<td>-4810</td>
<td>28300</td>
<td>818</td>
<td>1</td>
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<td>-5480</td>
<td>32500</td>
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<tr>
<td>60</td>
<td>-16800</td>
<td>-3260</td>
<td>20100</td>
<td>62</td>
<td>1</td>
</tr>
<tr>
<td>80</td>
<td>-385</td>
<td>-70</td>
<td>456</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

3.8 Conclusions

Previous studies have shown that changes in the shear stress and displacement can be utilized as indicators of unstable slip failure in numerical models, as the deviation of a simulated shear stress-displacement curve from its characteristic behavior. In this study, it was found that the rapid shear stress decrease and shear displacement increase immediately following the attainment of peak strength are larger for cases with lower intact rock stiffness, or higher joint shear stiffness and lower joint roughness. In addition, it was found that the magnitudes of various energy component changes during slip could be used to quantify unstable slip failure intensity. The use of these energy-based indicators has the advantage over the aforementioned conventional indicators in that it can be generalized to more complex geometries and modeling scenarios. The energy approach method applied in this study was demonstrated to be capable of reliably tracking the different energy components studied, and the overall energy balance in the modeled system was verified.

The increased potential for unstable slip in cases with softer intact rock material surrounding the failing discontinuity can be attributed to the fact that the magnitude of elastic strain energy in rock matrix at failure increases with decreasing rock stiffness. The increase in elastic strain energy leaves more stored energy available to be transferred to the slipping joint during failure. With respect to the influence of joint shear stiffness and roughness, although the amount of joint shear strain energy at the peak strength is larger when the joint stiffness is smaller and the joint roughness is larger (assuming the same peak strength), a discontinuity with smaller stiffness
and larger roughness has a much larger weakening shear displacement from its peak strength to residual strength; this results in a larger amount of joint friction work occurring during this weakening process.

An increase of normal stress acting on a discontinuity can increase the amount of stored energy in the discontinuity and the neighboring rock, but this increased energy storage only translates to an increased propensity towards unstable slip failure up to a point. Beyond a certain normal stress, however, unstable slip failure is suppressed due to the increase in joint ductility that occurs with increased normal stress.
CHAPTER 4

NUMERICAL ANALYSIS OF ROCKBURST POTENTIAL AS A FUNCTION OF THE CRITICAL DISCONTINUITY DISTANCE NEAR UNDERGROUND EXCAVATIONS

4.1 Abstract

The effects of major discontinuities on rockburst proneness near underground excavations in overstressed hard rock are significant for rock engineering activities. An energy tracking approach was developed and applied to investigate different factors influencing rockburst potential using the Universal Distinct Element Code (UDEC) software. A circular excavation near a discontinuity was set up as the base model, and mechanical interpretations of the excavation stability were conducted to study the rockburst damage induced by unstable discontinuity slip failure. The critical discontinuity distance was adopted to quantify the burst-prone potential using the base model, which is the minimum normal distance between the excavation and discontinuity plane required to prevent violent rock slab failure under static stress conditions. In burst-prone rock, a small change in the discontinuity-excitation distance can have a significant influence on the rockmass stability around the excavation. Considering that a large critical discontinuity distance indicates higher rockburst potential, the numerical results indicate that rockburst potential increases under the following conditions: high stress conditions, a large degree of in-situ stress anisotropy when the maximum principal stress is aligned with the discontinuity strike, a small angle between the discontinuity plane and the major principal stress, a large excavation radius (or excavation volume), and a longer discontinuity length (up to a point).

4.2 Introduction

4.2.1 Rockburst types

High magnitudes of pre-mining stress have greatly increased the occurrence of rockbursts in recent decades due to increasing mining depths in brittle hard rock (White et al. 2002; Kaiser and Cai 2012; Mazaira and Konicek 2015; Gale 2018b). A rockburst can be defined as a large volume of ejected rock fragments caused by the instantaneous release of energy from highly stressed surrounding rock in deep underground openings (Cook 1965). It is also defined as damage
to an excavation that occurs in a violent manner and is associated with a seismic event (Kaiser et al. 1996). During a rockburst, a large amount of pulverized rock can be violently ejected into underground openings, resulting in injuries or fatalities of workers and/or disruption of mining activities. Understanding the mechanisms of rockbursts is important in developing risk mitigation measures to enhance workplace safety in deep underground mines and excavations. Several research efforts have been conducted to study the factors influencing rockburst damage, and many critical factors such as the seismic events, geology and mining activity have been identified (Cook 1965; Salamon 1984; Linkov 1996; Kaiser and Cai 2012; Khademian et al. 2018). Unfortunately, the complex mechanisms of rockburst are still not fully understood, partially due to a lack of efficient and practical tools for analyzing rockburst in both the field and in numerical models.

Based on their source mechanisms, previous research studies have divided these ground failures into three major types: strain burst, pillar burst and fault-slip (Müller 1991; Mitri 1999; Kaiser and Cai 2012). Strain bursting involves the development of fractures in overstressed intact rock followed by the violent ejection of rock fragments in the vicinity of openings (White and Whyatt 1999; White et al. 2002; Whyatt, J, W. Blake 2002). Pillar bursting depends on the loading-carrying capacity of a pillar and the amount of released energy during pillar failure, and a pillar with large brittleness under soft overburdens tend to fail more violently (Kias and Ozbay 2013; Khademian et al. 2016; Manouchehrian and Cai 2017; Xu and Cai 2017b). Fault-slip, also called slip burst, is defined as unstable slip failure along pre-existing discontinuities and/or shear within intact rock, resulting in the sudden release of seismic energy (Whyatt, J, W. Blake 2002; Gu and Ozbay 2014; Sainoki and Mitri 2014, 2016). The resulting damage from these three rockburst types can be different in both damage distribution and amount of released energy in rockbursting mines. Overall, strain and pillar bursts resulting in ejected pulverized rock are of most hazards to the safety of mine workers and underground equipment, while fault-slips usually release the largest amount of seismic energy (White and Whyatt 1999; Whyatt, J, W. Blake 2002). However, the potential for rockburst can greatly increase when a major structural weakness plane exists nearby a tunnel, which is common in both small and large rockburst events.

A strain burst induced by slip failure in the vicinity of an opening can involve both unstable rock failure in intact rock material and unstable slip failure along discontinuities, which is a common type of unstable rock failure in the surrounding rock near underground excavations (Cook 1965; Salamon 1970, 1984; Rice 1983; Gu and Ozbay 2014; Manouchehrian and Cai 2015). Due
to the complexity of rockbursting, the underlying energy transformations associated with unstable rock failure in intact rock material and along discontinuities are not fully understood.

### 4.2.2 Unstable rock failure in intact rock and along discontinuities

Unstable rock failures are often accompanied by the ejection of rock fragments, which can be observed in both the laboratory and in rockbursting mines. Research on unstable rock failure began with laboratory testing of the post-peak behaviors of rock samples under a very stiff loading system, when the first stiff laboratory loading machine was designed in 1969 (Wawersik and Fairhurst 1970). The mechanical theory of unstable rock failure was initially proposed by Cook (1965), who stated that the amount of released energy during a uniaxial compressive strength (UCS) test depends on the loading system stiffness (LSS) and the post-peak behavior of rock samples. Unstable rock failure is more likely to occur in brittle rock under a soft loading system, where the LSS will impact the amount of stored elastic strain energy in the loading system before rock failure, while rock brittleness governs the amount of dissipated energy during rock failure (Salamon et al. 2003; Kias and Ozbay 2013; Cai 2016; Xu and Cai 2017b). In recent years, some newly-designed experiments have been conducted to produce unstable rock failure by changing loading conditions in the laboratory. For example, a steel beam, capable of storing a large amount of elastic strain energy, was placed between a rock specimen and its loading platen, causing the violent failure of the rock sample with rejected rock fragment speed as high as 4 m/s (Fakhimi et al. 2016). A true-triaxial rockburst testing platform was designed to compress five faces of a rock sample block in three axial directions to simulate the strain burst around excavations, where fractured rock fragments can be ejected violently from the free face (He et al. 2012; Zhao and Cai 2014).

Using a one-degree-of-freedom fault model, Rice (1983) suggested that the amount of elastic strain energy in the rock material surrounding a discontinuity was the energy source of slip failure, while the occurrence of unstable slip failure was determined by the shear stress-displacement behaviors of the discontinuity and shear stiffness of the surrounding rock. This energy hypothesis was studied by Gu and Ozbay (2014, 2015) using the numerical modeling software UDEC, where they confirmed that unstable slip failure was more likely to occur in a discontinuity surrounded by a softer rock material. Rice (1983) also concluded that the released energy during slip failure was affected by the shear stress-displacement of the discontinuity and the stiffness of the rock material surrounding the discontinuity. The estimation of released energy
with this graphical method was studied with direct shear test models in UDEC by Khademian et al. (2018), who also compared the numerical results with other analytical solutions. Manouchehrian and Cai (2015, 2017) utilized the maximum velocity of the failed rock within numerical models to quantify the rockburst intensity in a circular excavation model, demonstrating that the existence of a discontinuity can increase the occurrence and intensity of rock failure.

4.2.3 Rockburst damage induced by slip failure along discontinuities

Discontinuities parallel to openings can decrease rockmass strength and increase the rockbursting potential through stress concentration and buckling of the surrounding rock (White and Whyatt 1999; Nygård et al. 2006; Vazaios et al. 2019). Considerable research effort has been made toward understanding the buckling mechanism of rockbursts, where the rockmass was split or separated by discontinuity into layers subparallel to the wall of underground excavations (Fairhurst and Cook 1966; Ortlepp and Stacey 1994; White and Whyatt 1999). Localized stress concentrations result from the shortening of the rock slab between the excavation and discontinuity plane, and this is a critical contributing factor to rockburst initiation. A large rockburst occurred in the drainage tunnel of the Jinping II hydropower station in China on November 28, 2009, which caused seven fatalities and ejected 400 m$^3$ of rock fragments. Most importantly, a large fault dipping sub-horizontally to the tunnel axis was exposed after the rockburst, which had been recognized as the key contributing factor of this rockburst event by many researchers (e.g. Jiang et al. 2010; Li et al. 2012; Zhang et al. 2012a, 2014; Zhou et al. 2015a). A detailed description about the project and rockburst event is given in Chapter 5.

Previous studies indicate that during tunnel excavation in certain geologic conditions, rockburst events decrease significantly when the tunnel advance direction changes from parallel to perpendicular to strike of major discontinuities in some rockbursting mines (White and Whyatt 1999; White et al. 2002; Whyatt, J. W. Blake 2002; Sainoki and Mitri 2014). The discontinuities can change the local stiffness of the rockmass around the openings, resulting in local stress concentration and large rockbursting potential (Gu and Ozbay 2014, 2015). The highly stressed rock slab often results from the slip movement along the discontinuity, which shifts the stress in the deep rock, creating violent buckling failure. Based on experimental data, Duan et al (2019) carried out numerical investigations to consider the influence of principal stress orientation in relation to the pre-existing discontinuity orientation on unstable failure potential through UCS tests.
using the two-dimensional particle flow code (PFC 2D). The numerical results suggest that the excavation-induced fault instability highly depends on the magnitude and orientation of principle stress to the discontinuity during rock failure.

Excavation during mining results in both the in-situ stress redistribution around the openings and decrease of normal loads across nearby discontinuities. The rock stiffness is determined by both elastic properties and geometry of the rock block; therefore different rock stiffness can exist in the rock blocks closer to the openings, even when geological conditions are relatively homogeneous.

4.2.4 Research objective of current study

Given that the stressed rock slab bounded between an excavation and a discontinuity can fail suddenly and violently due to small stress changes or remote seismic events, the present numerical investigation aims to study the role of discontinuities on rockburst occurrence and damage around a circular tunnel. A large-scale tunnel model is used as a base model to identify factors most directly linked to rockburst hazards, where the sensitivity of the model results to horizontal-to-vertical in situ stress ratio, principal stress orientation in relation to the discontinuity plane, discontinuity lengths, and tunnel radius is evaluated. By tracking the evolution of energy storage and dissipation within the numerical models, the energy release mechanisms associated with rockbursting are assessed, leading to further insights into the causes of rockbursting.

4.3 Energy Calculations

4.3.1 Energy balance

The investigated energy components in a large-scale model with a local discontinuity include elastic strain energy, plastic strain work, joint friction work and damped kinetic energy; gravitational energy and joint shear strain energy are negligible in the context of this study, and are therefore ignored. Elastic strain energy ($W_e$) is the energy source for rock failures, while plastic strain work ($W_p$), joint friction work ($W_f$) and damped kinetic energy ($W_d$) are the components of energy dissipated during rock failure. In a large-scale model, the amount of joint shear strain energy can be neglected and its magnitude is minimal compared with the elastic strain energy in rock matrix. The external work ($W$) for a volume of rock can result from externally imposed stress changes or seismicity due to excavation or mining activities. When the state of rock changes from
one form to another, the energy balance within the rock proposed by Salamon (1984) can be summarized as shown in Eq. (4.1), where the left- and right-hand sides of Eq. (4.1) correspond to the energy components within a volume of rock before and after the change of state respectively.

\[
\Delta W + W_e + W_p + W_{jf} + W_d = W_e' + W_p' + W_{jf}' + W_d'
\]  
(4.1)

After rearranging Eq. (4.1), the change of elastic strain energy and external work equals to the energy dissipated through plastic strain work, joint friction work and damped kinetic energy shown in (4.2).

\[
\Delta W + \Delta W_e = \Delta W_p + \Delta W_{jf} + \Delta W_d
\]  
(4.2)

When no external work is applied to the volume of interest, the change of elastic strain energy will be the source of energy during rock failure.

**4.3.2 Energy components**

The investigated energy components within intact rock material include elastic strain energy, plastic strain work, total strain energy and damped kinetic energy in the present study. Elastic strain energy is a form of reversible strain energy resulting from the force applied to elastic materials, where the corresponding elastic deformation can be recovered after the force is removed. According to Jaeger et al. (2007), the elastic strain energy density in a unit volume of isotropic material can be expressed based on stress tensor components, the elastic modulus and Poisson’s ratio as shown in Eq. (4.3), which assumes that the rock material within each rock block has an elastic and linear behavior.

\[
U_e = \frac{1}{2E}\left[\sigma_x^2 + \sigma_y^2 + \sigma_z^2 - 2\nu(\sigma_x\sigma_y + \sigma_x\sigma_z + \sigma_z\sigma_y) + 2(1+\nu)(\tau_{xy}^2 + \tau_{xz}^2 + \tau_{zy}^2)\right]
\]  
(4.3)

Given that under plane strain conditions the shear stress tensors of \(\tau_{xz}\) and \(\tau_{xz}\) are always zero, Eq. (4.3) can be rewritten as:

\[
U_e = \frac{1}{2E}\left[\sigma_x^2 + \sigma_y^2 + \sigma_z^2 - 2\nu(\sigma_x\sigma_y + \sigma_x\sigma_z + \sigma_z\sigma_y) + 2(1+\nu)\tau_{xy}^2\right]
\]  
(4.4)

Also, given that rock is anisotropic and exhibits elasticity at low stress and plasticity near failure, the elastic strain and plastic strain can co-exist in the same rock material. The sum of elastic
strain energy and plastic strain work can be represented by the total strain energy. The increase in total strain energy density can be calculated with the average of each stress tensor by multiplying the corresponding strain tensor increment in Eq. (4.5) (Itasca Consulting Group 2014).

\[
\Delta U_t = \frac{1}{2} \left[ (\sigma_x + \sigma_x') \Delta e_x + (\sigma_y + \sigma_y') \Delta e_y + (\sigma_z + \sigma_z') \Delta e_z + (\tau_{xy} + \tau_{xy'}) \Delta \gamma_{xy} 
+ (\tau_{xz} + \tau_{xz'}) \Delta \gamma_{xz} + (\tau_{yz} + \tau_{yz'}) \Delta \gamma_{yz} \right] \tag{4.5}
\]

Shear strain and stress tensor components are always zero in the third direction in the plane strain condition, and therefore Eq. (4.5) can be simplified to the following:

\[
\Delta U_t = \frac{1}{2} \left[ (\sigma_x + \sigma_x') \Delta e_{xx} + (\sigma_y + \sigma_y') \Delta e_{yy} + (\tau_{xy} + \tau_{xy'}) \Delta \gamma_{xy} \right] \tag{4.6}
\]

Before inelastic damage is induced in the rock material, the initial total strain energy density is assumed equal to the elastic strain energy density. With the initial total strain energy density, the current total strain energy density \(U_t\) at different states can be obtained by adding the current increase in total strain energy density \(\Delta U_t\) to its previous total strain energy density \(U_t'\).

\[
U_t = U_t' + \Delta U_t \tag{4.7}
\]

Since the total strain energy is the sum of elastic strain energy and plastic strain work in a volume of rock, the plastic strain work density \(U_p\) can be expressed as the difference between the total strain energy density and the elastic strain energy density shown in Eq. (4.8).

\[
U_p = U_t - U_e \tag{4.8}
\]

During an unstable rock failure, part of the elastic strain energy will also be transformed into kinetic energy that is ultimately dissipated by the model; this is represented by the damped kinetic energy, which is the amount of kinetic energy removed from the models. A default local damping coefficient \(\alpha\) of 0.8 in UDEC was applied in the present study. The damped kinetic energy \(W_{d-gridpoint}\) over a time step \(\Delta t\) at a gridpoint is a function of damping coefficient, time step and kinetic energy in the gridpoint \(U_k\), which can be expressed as shown in Eq. (4.9)

\[
W_{d-gridpoint} = 2 \times \alpha \times \Delta t \times U_k \tag{4.9}
\]

The increase in damped kinetic energy over a time step equals to the sum of damped kinetic energy in each gridpoint of the model during that time step. The damped kinetic energy at any
given point of time is the sum of damped kinetic energy in all gridpoints during all previous time steps, which can be considered as a measure of cumulative kinetic energy.

The Continuously Yielding (CY) joint model was used to simulate the physical behavior of the discontinuity in this study, because it can represent the progressive damage of a discontinuity under continued shearing (Cundall and Lemos 1990; Itasca Consulting Group 2014). The CY joint model has been previously applied to model slip failures in both laboratory and in-situ discontinuities in UDEC (Gu and Ozbay 2014, 2015; Khademian et al. 2018). The joint shear displacement is the sum of the elastic shear displacement and plastic shear displacement in the CY joint model, where the plastic shear displacement is controlled by a yield factor (F). The yield factor is zero before the onset of plastic shear displacement in the discontinuity, but it begins to decrease from one to zero with an increase in plastic shear displacement. A detailed description and formulations of the yield factor can be seen in the reference (Cundall and Lemos 1990). The plastic shear displacement \( \Delta u_s^p \) makes up a proportion of the total shear displacement \( \Delta u_s \) as determined by Eq. (4.10).

\[
\Delta u_s^p = (1 - F) \times \Delta u_s \tag{4.10}
\]

The joint friction work corresponds to the shear force and plastic shear displacement in the discontinuity, resulting from the dissipation of elastic strain energy. Therefore, the change in joint friction work \( \Delta W_{jf} \) is equal to the product of average shear force \( \bar{f}_s \) and the change in plastic shear displacement \( \Delta u_s^p \) – see Eq. (4.11).

\[
\Delta W_{jf} = - \bar{f}_s \times \Delta u_s^p = \bar{f}_s \times (F - 1) \times \Delta u_s \tag{4.11}
\]

By adding the change in joint friction work to the previous magnitude of joint friction work \( W_{jf} ' \), the current joint friction work \( W_{jf} \) is illustrated in Eq. (4.12).

\[
W_{jf} = W_{jf} ' + \Delta W_{jf} \tag{4.12}
\]

By implementing the above energy equations with FISH code in UDEC, the energy storage and dissipation within a numerical model can be tracked and analyzed to understand the energy mechanisms of rockbursts. With this approach, the energy transformations between different
energy components can be analyzed by assessing to the change and distribution of these energy components within the entire numerical model.

4.4 Model Setup

The base model consists of a horizontal discontinuity placed above a circular tunnel (Figure 4.1). The horizontal discontinuity in the base model, expressed at different orientations in later models, could represent mining-induced fracture, bedding plane, or other pre-existing geological discontinuity. The numerical model has a height and width of 100 m to avoid any influence of the outer boundary on stress redistribution after excavation, and all four sides of the model are fixed with roller constraints. Due to the plane strain condition in UDEC, the lengths of all features (e.g. tunnel and discontinuity) in the out of plane direction are effectively infinite. In the base model, the circular tunnel is assumed to be at a depth of 1200 m with horizontal-to-vertical in-situ stress ratio of 1.5, as in-situ stress ratios larger than one are commonly observed in rockbursting mines, for example 2 in the Canadian Shield or 3.5 at the Junction Mine in Australia (M.F. et al. 2001; Whyatt, J. W. Blake 2002; Ortlepp 2005; Keneti and Sainsbury 2018). Therefore, the in-situ stress of the rock at that depth will be 33.6 MPa and 50.4 MPa in the vertical and horizontal directions, respectively. The tunnel radius is set as 2 m and the discontinuity has a length of 60 m in the base model.

It should be noted that the discontinuity distance is the normal distance between the tunnel and discontinuity plane, which also is equal to the minimum thickness of the rock slab between the discontinuity and tunnel. For example, when the horizontal discontinuity is 3.0 m above the circular tunnel center, the discontinuity distance will be 1.0 m with a tunnel radius of 2.0 m. Though almost all mine excavations are square or rectangular, the distance between a discontinuity and circular tunnel is easier to define given different discontinuity orientations will be analyzed in this study.
Figure 4.1 Geometry and boundary conditions of the base model with an tunnel radius of 2.0 m and a horizontal discontinuity with a starting orientation (dip angle) of 0 degrees.

The anisotropic zone stress is set to 33.6 MPa in the vertical direction and 50.4 MPa in the horizontal direction, and this constant in-situ stress is obtained at the initial equilibrium with the roller constraints outside the model before the tunnel is introduced into the model. Since UDEC uses an explicit solution method, a constant time step of $5 \times 10^{-7}$ s was assigned to all the models in this study, meaning one second of solution time (different from the actual physical time of one second) represents two million steps.

The rock material is simulated using Mohr-Coulomb model with strain-softening behavior, and the hypothetical rock properties are based on practical estimates of properties for a generic good quality hard rockmass from Hoek and Brown (1997). These parameters are listed in Table 4.1. For good quality hard rockmass such as granite, the rockmass behaves in a brittle manner, which means the rock strength drops suddenly after the peak strength. The strain-softening parameters were selected to promote brittleness of the rockmass so that violent rock failure can occur in the models (see Table 4.2).
Table 4.1 Parameters of the Mohr-Coulomb model for simulating good quality hard rockmass.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Granite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2800</td>
</tr>
<tr>
<td>Young’s modulus (GPa)</td>
<td>40</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.2</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>40</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>10</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Table 4.2 Strain-softening parameters of the Mohr-Coulomb model.

<table>
<thead>
<tr>
<th>Cohesion</th>
<th>Tension cut-off</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion yield Stress (MPa)</td>
<td>Shear plastic strain</td>
</tr>
<tr>
<td>10</td>
<td>0</td>
</tr>
<tr>
<td>0.01</td>
<td>0.01</td>
</tr>
</tbody>
</table>

The CY joint model input parameters used for the discontinuity are listed in Error! Not a valid bookmark self-reference.. The values of joint (normal and shear) stiffness, friction coefficient and roughness are based on previous studies conducted using the CY joint model and practical estimates of joint normal and shear stiffness (Swan 1983; Gu and Ozbay 2014; Thirukumaran et al. 2016; Khademian and Ozbay 2018).

Table 4.3 Parameters of the CY joint model for simulating the discontinuity slip movement.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint normal stiffness</td>
<td>50 GPa/m</td>
</tr>
<tr>
<td>Joint shear stiffness</td>
<td>20 GPa/m</td>
</tr>
<tr>
<td>Initial friction coefficient</td>
<td>0.6</td>
</tr>
<tr>
<td>Residual friction coefficient</td>
<td>0.5</td>
</tr>
<tr>
<td>Joint roughness</td>
<td>0.1 mm</td>
</tr>
</tbody>
</table>
The excavation is implemented using a relaxation method, which allows reacting boundary forces within the perimeters of an excavation to be reduced and ramped down to a prescribed force level without causing any plastic damage to the rock. This excavation method can avoid unrealistic tensile failure that is sometimes associated with sudden unloading in numerical models. The prescribed force level in each grid point of the inner boundary is evenly decreased with a step of 0.5 percent of its initial maximum magnitude, which means there are two hundred relaxation steps in total. Each relaxation step is assigned one thousand time steps for the model to converge. The excavation relaxation process occurs over 0.2 million model steps (or a solution time of 0.1 seconds) in each numerical model, which is large enough to avoid unrealistic tensile failure around the tunnel.

The entire model is discretized using a wide range of mesh sizes from 0.04 m to 2 m, where denser mesh is applied near the tunnel with a discontinuity distance of 1.1 m. A mesh optimization study was conducted for the mesh immediately surrounding the tunnel, where the average principal stress in the rock slab close to tunnel center is obtained at equilibrium state of the model. The average principal stress within the rock slab of the tunnel, given different mesh sizes, is presented in Figure 6. As the mesh size value around the tunnel decreases from 0.2 m to 0.08 m, the average principal stress decreases from 77.7 MPa to 75.3 MPa. Then the average principal stress becomes stable when the mesh size decreased from 0.08 m to 0.04 m. According to Figure 4.2, when mesh size values range from 0.04 m to 0.08 m, the average principal stress in the rock slab close to the tunnel center is stable. A mesh size of 0.05 m near the tunnel is then selected to exclude the mesh size influence on rock failure behavior in this large model.

Energy components in each zone and contact within the numerical model are calculated at every 100 time steps to reduce the numerical computing load and number of output points for plotting purposes. It should also be noted that the tracking of different energy components within the entire model begins immediately after the excavation, which implies that the elastic strain energy in the excavated rock material is not counted. Each model takes about one million time steps from the time of excavation to the final equilibrium, resulting in approximately ten thousand recorded points in each energy curve. If sparse recorded points are observed in different energy curves, the implication is that there is a rapid change of energy that occurred in the model.
4.5 Indicators of the rockburst damage induced by unstable slip failure

In this study, the changes in different energy components within the models are utilized to analyze and identify the rockburst behavior induced by unstable slip failure. For the rockburst damage induced by unstable slip failure, distances between the tunnel and discontinuity plane are varied and simulated in steps of 0.1 m for the base model. Therefore, 0.1 m is the maximum precision for the determination of the critical discontinuity distance in the present study. The influence of discontinuity slip movement on rock failure was simulated to better understand the interactions between rockburst damage and discontinuity slip movement.

4.5.1 Physical behavior of the rockmass and discontinuity plane

Different discontinuity distances were simulated with the base model, and a step change in behavior was only noted between the models with discontinuity distances of 1.0 m and 1.1 m. The distribution of the displacement magnitudes around the tunnel in these two cases is plotted in Figure 4.3, where the color bar limits in Figure 4.3a are ten times of these in Figure 4.3b for the same color scheme. Displacements as much as 0.1 m can be observed in the roof and floor of the tunnel, with a discontinuity distance of 1.0 m, while the maximum displacement in the model with a discontinuity distance of 1.1 m is only 0.005 m. The irregular displacement contours and large displacement magnitudes in the case of 1.0 m indicate that the rockmass with large displacement...
has lost its strength in Figure 4.3a. In contrast, regular and smooth displacement contour outlines can be clearly seen around the tunnel, with a discontinuity distance of 1.1 m.

![Figure 4.3 The distribution of displacement magnitudes around the circular tunnel with discontinuity distances of (a) 1.0 m and (b) 1.1 m.](image)

The interactions between the tunnel excavation and discontinuity plane can be analyzed by plotting the distributions of shear displacement, normal stress and shear stress along the discontinuity plane at equilibrium state of the models (Figure 4.4). Before excavation, the initial close-to-zero shear stress in the discontinuity results from the parallel relationship between the discontinuity plane and the major principal stress, while the normal stress in the discontinuity is induced by the minor principal stress being orthogonal to the discontinuity plane. Therefore, the initial normal and shear stress in the discontinuity plane prior to excavation are 33.6 MPa and 0 MPa, respectively.

In Figure 4.4a, the distribution of shear displacement in the discontinuity plane indicates that two parts of the discontinuity plane move towards the opposite directions in both cases, although the slip movement in the case of 1.1 m is very small. Large shear displacements (as much as 2 cm) are observed in the model with a discontinuity distance of 1.0 m, indicating that local slip failure occurred in the discontinuity plane near the excavation. The influence of excavation on the normal stress along the discontinuity plane can be seen in Figure 4.4b, where the normal stress near the excavation is less than 33.6 MPa at the equilibrium state of the model in both cases, while the normal stress drops to zero after the rock failure in the case of 1.0 m. In Figure 4.4c, the shear stress distributions beside the two parts of the discontinuity plane have similar magnitudes, but
opposite directions. The direction of the shear stress indicates that the shear stress is the resisting force of the discontinuity slip movement, and shear stress as large as 16 MPa along the discontinuity can be observed in the model with a discontinuity distance of 1.0 m.

![Figure 4.4 Distribution of (a) shear displacement, (b) normal stress, and (c) shear stress along the discontinuity plane at equilibrium state of the models with discontinuity distances of 1.0 m and 1.1 m.](image)

Both the excavation and slip movement along the discontinuity contribute to the generation of stress concentration, resulting in the shortening of the rock slab between the excavation and discontinuity plane. In addition, the decrease in the normal stress across the discontinuity resulting from the unloading during excavation reduces the shear strength in the part of discontinuity closest
to the excavation. Once the rock slab failure is triggered, a large shear displacement corresponding to slip failure in the discontinuity is obvious. Whether the slab or discontinuity fails first is an interesting question and depends on many factors; however the question is beyond the current research scope in this study. It is clear from these results, however, that a minor change in discontinuity distance can have a significant influence on the rockmass stability around the excavation.

4.5.2 Energy transformations during rockmass failure induced by discontinuity slip movement

By calculating and summing the energy values in each zone or contact of the model, the total amount of elastic strain energy, plastic strain work, joint friction work and damped kinetic energy within each model can be obtained. Therefore, the energy transformation from elastic strain energy to plastic strain work, joint friction work and damped kinetic energy can be studied. The four sides of the model are constrained in roller conditions, indicating that no external mechanical energy can be transferred into the numerical models. However, the stress redistribution after excavation will result in the decrease in elastic strain energy at equilibrium state of the model. The evolution of different energy components in the model as a function of solution time is shown in Figure 4.5.

No perceptible change of energy components can be observed during the first 0.1 seconds of solution time, when the force in grid points along the inner boundary is slowly reduced with relaxation method to simulate the advance of excavation. The elastic strain energy within both models decreases simultaneously with solution time after the excavation, while a rapid decrease in elastic strain energy is observed in the 1.0 m case. Rapid changes of plastic strain work, joint friction work and damped kinetic energy with solution time can also be observed for the 1.0 m case. Much more energy is associated with slip failure (joint friction work) and intact rock damage (plastic strain work) in the model with 1.0 m discontinuity distance than in the model with 1.1 m discontinuity distance. The rapid rock failure also can be characterized by the relatively loose density of recorded points in the curves of joint friction work with solution time. The large and rapid change of joint friction work indicates that a rapid slip failure occurred along the discontinuity (resulting from the failure of the overstressed rock slab). Moreover, this rapid slip movement along the discontinuity also results in significant damage in the intact rock slab that
moves towards the excavation based on the displacement magnitude and vector in numerical models.

Figure 4.5 Changes of (a) elastic strain energy, (b) plastic strain work, (c) joint friction work, and (d) damped kinetic energy as a function of solution time in models with the discontinuity distances of 1.0 m and 1.1 m.

A large magnitude of damped kinetic energy still can be observed in the model with 1.1 m discontinuity distance due to the stress redistribution process, which can be assumed to be the amount of damped kinetic energy for the model to reach equilibrium without failure. Therefore, the difference between the two curves in Figure 4.5d can be assumed to be the damped kinetic energy due to the rock failure relative to the stable equilibrium condition. It is also the same case with the other energy components (elastic strain energy, plastic strain work and joint friction work).
In the model with 1.0 m discontinuity distance, the rock failure results in a large decrease in elastic strain energy density in both shallow and deep surrounding rock (Figure 4.6a). The slip failure resulting from rock failure can transfer a large amount of elastic strain energy from the deep surrounding rock to the dissipation of energy through plastic strain work and damped kinetic energy in the shallow rock around the excavation. However, the distribution of elastic strain energy density in the deep surrounding rock still remains at a relatively large magnitude in the model with discontinuity distance of 1.1 m, where large elastic strain energy density concentrations in the direct roof and floor confirm the stress concentration due to the excavation and discontinuity.

The distributions of plastic strain work density in models with discontinuity distances of 1.0 m and 1.1 m are presented in Figure 4.7, where plastic strain work mainly localizes in the roof and floor of the excavation due to the fact that the discontinuity plane and major principal stress are both horizontal. Areas with larger plastic strain work density represent more elastic strain energy being dissipated into plastic strain work in the form of rock fractures or failure planes (Figure 4.7a). In Figure 4.7b, a very small magnitude of plastic strain work density can be observed in the shallow roof and floor of the excavation with 1.1 m discontinuity distance, resulting from the redistributed stress concentration after excavation.
Unstable rock failure around excavation near a discontinuity can be identified when large amount of elastic strain energy is released due to a small disturbance such as fault distance in this study. It is clear from the results of the base model that a minor change in the discontinuity distance can have a significant influence on the distribution of elastic strain energy density and plastic strain work density around the excavation. Moreover, large and rapid changes in different energy components in the model with discontinuity distance of 1.0 m indicate that the rock failure occurs rapidly and violently after excavation.

The elastic strain energy stored in the rock is the energy source of the plastic strain work, joint friction work and damped kinetic energy in the present study. While the elastic strain energy within the model is decreasing during stress redistribution due to excavation or rock failure, the plastic strain work, joint friction work and damped kinetic energy are correspondingly increasing at the same time. Taking the solution time of 0.4 second as a point of reference, the elastic strain energy decrease ($\Delta W_e$), plastic strain work increase ($\Delta W_p$), joint friction work increase ($\Delta W_{jf}$) and damped kinetic energy increase ($\Delta W_d$) relative to the initial model states are presented in Table 4.4.
Table 4.4 Magnitudes of energy components at a solution time stamp of 0.4 seconds in models with discontinuity distances of 1.0 m and 1.1 m.

<table>
<thead>
<tr>
<th>d (m)</th>
<th>ΔW_e (MJ)</th>
<th>ΔW_p (MJ)</th>
<th>ΔW_{jf} (MJ)</th>
<th>ΔW_d (MJ)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>-4.45</td>
<td>2.62</td>
<td>0.46</td>
<td>1.26</td>
<td>-2</td>
</tr>
<tr>
<td>1.1</td>
<td>-1.1</td>
<td>0.27</td>
<td>0.01</td>
<td>0.82</td>
<td>0</td>
</tr>
</tbody>
</table>

A difference of 2% is observed between the decrease in elastic strain energy and the increase in plastic strain work, joint friction work and damped kinetic energy. The damped kinetic energy observed in the model with 1.1 m discontinuity distance is 0.82 MJ, but this is entirely associated with the redistribution of the in-situ stress after the excavation. If we assume the model with 1.1 m discontinuity distance is the equilibrium state of the excavation before the rock damage within the model with 1.0 m discontinuity distance, the change in energy components due to fault-slip can be calculated with these two model results. The difference between each energy components is -3.35 MJ, 2.35 MJ, 0.45 MJ, and 0.44 MJ for elastic strain energy, plastic strain work, joint friction work and damped kinetic energy, respectively. The approach applied in the present study is capable of reliably tracking different energy components in numerical models, improving current understanding of the energy transformations associated with rockbursts.

### 4.5.3 Critical discontinuity distance

The critical discontinuity distance can be defined as the minimum discontinuity distance to avoid unstable slip failure, while discontinuity distance represents the normal distance between the discontinuity and excavation surface. The critical discontinuity distance is between 1.0 m and 1.1 m for the base model case with a precision of 0.1 m as described above. The occurrence of rock slab failure in the excavation can be triggered by several factors including seismic energy, stress redistribution due to mining activities, and time-dependent effects on rock strength.

A larger critical discontinuity distance represents a higher rockburst-prone potential, for a large safe zone should be kept between the excavation and discontinuity plane. Rockburst induced by discontinuity slip movement will occur at a discontinuity distance less than the critical distance when the discontinuity plane is parallel to the excavation; while the potential of rockburst will be greatly reduced at discontinuity-excitation larger than the critical distance. The parameter of
critical discontinuity distance can be utilized as a good indicator of investigating the influence of other parameters on the occurrence of rockburst damage induced by unstable slip failure.

4.6 Parametric analysis

By adopting the critical discontinuity distance as a parameter for representing rockburst potential, sensitivity analyses were conducted using the base model as a starting point. The factors which were investigated include the principal stress magnitude and orientations in relation to the discontinuity plane, the excavation radius, and the discontinuity length.

4.6.1 Principal stress magnitude and orientations in relation to discontinuity plane

Large horizontal-to-vertical in-situ stress ratios are common in many underground tunneling and mining environments, where the major principal stress is often sub-horizontal (M.F. et al. 2001; Keneti and Sainsbury 2018). When present at orientations subparallel to the direction of the major principal stress, major discontinuities can result in highly stressed rock and burst-prone slabs around excavations.

While keeping the vertical stress as constant, an in-situ stress K ratio ranging from 1.0 to 1.5 and discontinuity dip angle from 0° to 90° are simulated using the base model setup for all other model parameters. A larger in-situ stress ratio also represents a higher major principal stress magnitude due to the constant vertical stress. The discontinuity dip angle is equal to the angle between the major principal stress and discontinuity plane, resulting from the major principal stress in the horizontal direction. The change in critical discontinuity distance with discontinuity dip angle under different in-situ stress ratios (k) using the base model setup is illustrated in Figure 4.8.

The results show that with increasing horizontal-to-vertical in-situ stress ratio, the critical discontinuity distance approximately increases at the same discontinuity dip angles, with a more notable impact in models with a smaller discontinuity dip angle. The critical discontinuity distance in models with 0° dip angle increases notably from 0.8 m to 1.1 m when the in-situ stress ratio changes from 1.4 to 1.5. Therefore it can be inferred that the critical discontinuity distance increases with the magnitude of major principal stress.
Sub-horizontal fracture orientations led to the most instability in the models, due to the concentration of high horizontal stresses \((k > 1)\) between the discontinuity and excavation. No influence of discontinuity dip angle on the critical discontinuity distance can be observed in models under an hydrostatic stress condition \((k = 1.0)\), for the discontinuity is always along one of the principal stress orientations. The general decrease in critical discontinuity distance with increasing dip angle can be seen in all in-situ stress ratios from 1.1 to 1.5.

To study the influence of stress magnitude on critical discontinuity distance, the maximum principal stress values of 33.6 MPa, 37.0 MPa, 40.3 MPa, 47 MPa and 50.4 MPa are applied horizontally in the base model, where one set of models have hydrostatic in-situ stress conditions \((k = 1)\), and the other set have anisotropic stress conditions with a constant vertical stress of 33.6 MPa \((k > 1)\). The changes in critical discontinuity distance with maximum principal stress are plotted in Figure 4.9. In the models having the same maximum principal stress, the critical discontinuity distance is smaller in the hydrostatic stress conditions than that in anisotropic stress conditions.
For both the $k = 1$ and $k > 1$ cases, the critical discontinuity distance positively increases with the maximum principal stress magnitude. At the same maximum principal stress magnitude, the critical discontinuity distance in the model with $k = 1$ is smaller than that with $k > 1$, which indicates that the anisotropic stress can increase the rockburst potential in excavations near a major discontinuity.

4.6.2 Excavation radius and discontinuity length

When the radius of the excavation is large, or when different excavations are close to each other, the rock slab bounded by a major discontinuity is more likely to fail unstably and violently (Kaiser and Cai 2012; Keneti and Sainsbury 2018; Manouchehrian 2018). With all other base model parameters held constant, different excavation radii were assigned to the model to investigate their corresponding critical discontinuity distances. The curve of the critical discontinuity distance as a function of excavation radius is plotted in Figure 4.10, where the critical discontinuity distance increases nearly proportionally to the excavation radius. The results indicate that the model no longer remains stable when the excavation radius is equal to and larger than 2.3 m. The mesh transition boundary is kept constant in the base model, and therefore the distance between the excavation boundary and the mesh transition boundary becomes smaller with
increasing excavation radii, resulting in the tunnel fail due to coarse mesh size effect. In summary, the excavation radius (or excavation volume) can have a significant influence on the stability of the excavation and related rockburst potential.

Figure 4.10 The change of critical discontinuity distance with excavation radius in the base model having dip angle of 0°, in-situ stress ratio of 1.5, and discontinuity length of 60 m.

Figure 4.11 The change of critical discontinuity distance with fault length in the base model having excavation radius of 2.0 m, dip angle of 0°, and in-situ stress ratio of 1.5.
Similarly, the influence of the discontinuity length on the critical discontinuity distance was investigated using the base model setup. For discontinuity lengths above 15 m, the critical discontinuity distance remains at 1.1 m; below this discontinuity length, the critical discontinuity distance decreased from 1.1 m to 0.9 m when the discontinuity length decreased from 15 m to 5 m (Figure 4.11). As clearly observed from Figure 4.6, the existence of a discontinuity can transfer the elastic strain energy in the deep rock to the rock immediately surrounding the excavation. The discontinuity length of 15 m can be regarded as the maximum effective length of the discontinuity for this specific base model.

4.7 Conclusions and future work

The energy-based approach presented in this study is a useful technique to reliably track the change of different energy components from the commencement of excavation to ultimate rock failure. The tracked changes in the different energy components with solution time show that a decrease in the elastic strain energy corresponds to increases in complementary energy components, namely plastic strain work, joint friction work and damped kinetic energy in the studied models. Discontinuity slip movement near an excavation can increase the stress concentration in the bounded rock slab, whose failure will cause a large and rapid increase in shear displacement (or joint friction work) along the discontinuity. Rapid shear displacement towards the excavation within the discontinuity exacerbates the failure intensity in the rock slab. Accordingly, sudden changes in the model energy components, such as joint friction work, can be utilized as indicators of the high intensity of rock failure induced by discontinuity slip in numerical models.

The existence of a major discontinuity parallel to an excavation surface can result in stress concentration development and ultimately a release of the elastic strain energy from the deep surrounding rock, which serves as an energy source for violent rock failure around the excavation. In the present study, no rock failure was observed in models with discontinuity distances of 1.1 m or greater, while a large volume of rock failed rapidly in the base model case with a 1.0 m discontinuity distance. The observation indicates that the distance between the excavation and discontinuity plane plays a critical role in determining burst-proneness. A large critical discontinuity distance indicates that a large distance between the excavation wall and discontinuity plane, is needed to avoid violent rock slab failure under static stress conditions (i.e. a relative high
burst potential). A sensitivity analysis of different rockburst contributing factors shows that the critical discontinuity distance increases with horizontal-to-vertical in-situ stress ratios (or maximum principle stress magnitude), in-situ stress anisotropy and magnitude, excavation radius (or excavation volume) and discontinuity length; however, the critical discontinuity distance decreases as a function of the angle between the discontinuity plane and the major principal stress. The discontinuity length can influence the critical discontinuity distance, but this influence is negligible when the length is above a certain threshold (15 m for the example base model illustrated in this study).

This parametric analysis mainly focuses on the influence of unfavorable stress and geological conditions around a theoretical circular excavation closed by a discontinuity. The impacts of material and joint properties on unstable rock failure, including the rock stiffness, rock brittleness and joint stiffness, were examined in CHAPTER 2 and CHAPTER 3, respectively. The underlying mechanism of rock slab failure, as discussed in this chapter, is believed to be associated with a combination of unstable rock failure within the rock slab and unstable slip failure along the discontinuity. Therefore, future research work following this study could focus on the influence of rock and discontinuity properties on the critical discontinuity distance.
CHAPTER 5

NUMERICAL MODELING OF ROCKBURST DAMAGE INDUCED BY FAULT-SLIP IN THE DRAINAGE TUNNEL OF JINPING II HYDROPOWER STATION

5.1 Introduction

A large rockburst event occurred at the Jinping II drainage tunnel in China in 2009, and this event is believed to have been caused by a major geological structure (Zhang et al. 2012b, 2014; Zhou et al. 2015b). In this Chapter, the previously presented energy approach for studying rockburst hazards in numerical models is applied to this rockburst event and the energy results from the model are verified against field data. Specifically, a numerical simulation of this rockburst event was conducted to understand the mechanical cause of this rockburst hazard, with the underlying assumption that a minor decrease in the fault distance may result in the triggering of the rock slab failure. The research objective was to investigate the hypothesis that the rockburst damage induced by fault-slip mainly resulted from the geological feature, given that the fault plane is sub-parallel to the excavation surface of the tunnel; therefore the rock slab between the fault plane and the excavation would be expected to have a smaller stiffness due to its geometry, and larger stress concentration than the rest of the surrounding rockmass.

5.1.1 Project overview

The Jinping II hydropower station was designed to produce energy by utilizing an elevation drop of 310 m along the Yalong River beside the Jinping Mountain (Li et al. 2012; Zhang et al. 2013). Seven parallel tunnels were constructed to traverse the Jinping Mountain from the upstream to downstream part of the Yalong River, where more than 75% these tunnels have cover depths of 1500 m or greater and the maximum overburden is 2,525 m (Figure 5.1a) (Li et al. 2012; Zhang et al. 2013). During the excavation of these tunnels, a large number of rockburst events from small strain bursts to devastating fault-slip rockbursts were observed, among which the most intense occurred on November 28, 2009, in the drainage tunnel. The drainage tunnel was excavated using a TBM with a diameter of 7.2 m, and the location of this tunnel relative to other excavations is presented in Figure 5.1b.
Figure 5.1 Jinping II hydropower station: (a) map of the project in China, (b) layout and configuration of seven tunnels (Li et al. 2012).

Due to the great depth of the tunnels, 133 large strain bursts and fault-slips occurred during construction (Chen et al. 2013). Rockbursting during the project mostly occurred a few meters behind the tunnel face, and close to many steeply dipping faults which are commonly found in the region. The maximum principal stress can be as much as 63 MPa, and the primary rock type is the Baishan Group marble. The strain burst induced by fault-slip on November 28, 2009, in the drainage tunnel (referred to as the “11.28 rockburst”), resulted in severe consequences and hazards during the project.

5.1.2 The 11.28 rockburst event

The so-called “11.28 rockburst” occurred in the drainage tunnel, releasing material from the tunnel face to approximately 24 m behind the tunnel face, where the overburden is about 2330 m. This disaster caused seven fatalities, one injury, and damaged the TBM. Before this rockburst, the tunnel had been supported with shotcrete, H-section steel sets, and cement-grouted rock bolts. The support system was able to strengthen the rockmass strength and resist the ejected or collapsed
rock during previous strain burst events, before the 11.28 rockburst. However, all support systems were destroyed by the sudden dynamic loading generated by the rockburst, and the main beam of the TBM machine was damaged. It was estimated that a Richter magnitude of 2.0 was recorded by the microseismic monitoring system equipment (Zhang et al. 2012b).

The rockmass mainly consists of medium-fine-grained marble, including hard calcite, pinstripe biotite, and minor quantities of other minerals. Some structural planes with scratches indicated slip failures occurred along these structural planes during the rockburst. A fault with a dip angle of 50° was exposed after this rockburst, and the rockmass between the excavation and fault collapsed in the rockburst section. This fault plane is smooth and sub-parallel to the tunnel axis with no filling. The maximum depth of the failure zone can be as much as 7 m, as shown in Figure 5.2. The failure depth of this rockburst event along the tunnel is a function of the large fault and the local presence other structure planes, however the details about the other structure planes are not provided in the literature. The weak discontinuities can greatly determine the profile of rock damage during rockburst, and many other discontinuities except this large fault were observed after this large rockburst event (Zhang et al. 2012b). Therefore, only the influence of the fault on the rockburst event is investigated herein, but the maximum ejected rock volume after the
rockburst will not be compared with the numerical results due to the lack of geotechnical data on the other geological features. Specifically, the investigation focused on the likelihood that the rockburst damage was mainly due to fault-slip along the fault, which was mapped as sub-parallel to the excavation surface of the tunnel. During the advance of tunnel, the tunnel fault distance decreased, and the rockburst event exposed a length of fault with distances from the original excavation approximately ranging from 0.3 m (close to the face) to 1.4 m (away from the face). Therefore, the average discontinuity distance was assumed to be between 0.6 m and 0.7 m in the rockburst section. The cause of this rockburst event was thought to result from the fault-slip failure induced by the tunnel excavation.

5.2 Model setup

5.2.1 Rockmass parameters

Laboratory test results show that the marble from the project site has a UCS of 80 MPa to 120 MPa, and physical and mechanical parameters of the rock are listed in Table 5.1 (Li et al. 2012; Chen et al. 2013). The rockmass is simulated using the Mohr-Coulomb model with strain-softening behavior in UDEC, where the cohesion of the rockmass is increased from 23.9 MPa to 26 MPa to keep the tunnel stable before rock failure. The cohesion-softening and tension-softening parameters of the rockmass are shown in Table 5.2. Additionally, the parameters in Table 5.2 are utilized to promote relatively high rock brittleness so that unstable rock failure occurs in the model (as was observed in reality).

Table 5.1 The mechanical parameters for simulating the rockmass at the 11.28 rockburst tunnel section (Li et al. 2012).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2780</td>
</tr>
<tr>
<td>Young’s modulus (GPa)</td>
<td>25.3</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.23</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>46</td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>23.9</td>
</tr>
<tr>
<td>Dilation angle (°)</td>
<td>10</td>
</tr>
<tr>
<td>Tensile Strength (MPa)</td>
<td>1.5</td>
</tr>
</tbody>
</table>
Table 5.2 Calibrated parameters of the Mohr-Coulomb strain-softening model for the rockmass.

<table>
<thead>
<tr>
<th>Cohesion yield Stress (MPa)</th>
<th>Shear plastic strain</th>
<th>Tension cut-off stress (MPa)</th>
<th>Tensile plastic strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>26</td>
<td>0</td>
<td>1.5</td>
<td>0</td>
</tr>
<tr>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.001</td>
</tr>
</tbody>
</table>

In Figure 5.1c, the drainage tunnels were excavated ahead of the four headrace tunnels, so minimum influence of re-distributed stress would be expected from them on the 11.28 rockburst section of the drainage tunnel. The distance between the drainage tunnel and transportation tunnel #2 was approximately five times of the tunnel diameter, which is also far enough to influence the drainage tunnel much. Even if the drainage tunnel is influenced by the other excavations, about 3% of increase in the in-situ stress is expected according to Kirsch Equations (Kirsch 1898). The in-situ stress components (compression is negative) of the rockmass at the rockburst sections of the drainage tunnel are listed in Table 5.3 (Zhang et al. 2013).

Table 5.3 The in-situ stress components at the 11.28 rockburst section of the drainage tunnel (Zhang et al. 2013).

<table>
<thead>
<tr>
<th>$\sigma_x$ (MPa)</th>
<th>$\sigma_y$ (MPa)</th>
<th>$\sigma_z$ (MPa)</th>
<th>$\tau_{xy}$ (MPa)</th>
<th>$\tau_{yz}$ (MPa)</th>
<th>$\tau_{xz}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-46.4</td>
<td>-51.7</td>
<td>-61.5</td>
<td>-2.4</td>
<td>-0.6</td>
<td>3.5</td>
</tr>
</tbody>
</table>

A fault near the rockburst damage location was revealed after this 11.28 rockburst, which was straight and smooth with no infilling (Zhou et al. 2015b). Table 5.4 presents the input parameters of the fault with the CY joint model, and the values of these parameters are based on previous research conducted using the CY joint model, and practical estimates of joint normal and shear stiffness (Swan 1983; Gu and Ozbay 2014; Thirukumaran et al. 2016; Khademian and Ozbay 2018).
Table 5.4 Parameters of the CY joint model for simulating the fault-slip movement.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint normal stiffness</td>
<td>50 GPa/m</td>
</tr>
<tr>
<td>Joint shear stiffness</td>
<td>20 GPa/m</td>
</tr>
<tr>
<td>Initial friction coefficient</td>
<td>0.6</td>
</tr>
<tr>
<td>Residual friction coefficient</td>
<td>0.5</td>
</tr>
<tr>
<td>Joint roughness</td>
<td>0.1 mm</td>
</tr>
</tbody>
</table>

5.2.2 Model construction

Elastic analysis results indicate that the in-situ stress around the tunnel increases only about 3 % by the excavation of other tunnels in Figure 5.1, and thus the influence of nearby excavations on the model results is minimal and can be neglected in this study. By simplifying the rockburst in this circular excavation into a two-dimensional problem, the rock failure in the drainage tunnel can be analyzed as a plane strain condition. The two-dimensional model geometry and in-situ stress conditions are illustrated in Figure 5.3, where gravitational acceleration (9.8 m/s²) is downward.

The fault distance was defined as the normal distance between the tunnel wall and the fault plane. Given that a fault with a dip angle of 50° will intersect with square or circular blocks in UDEC and may result in irregular block shapes, this presents difficulty in generating satisfying meshes in UDEC. Accordingly, the model in Figure 5.3 can be rotated so that the fault is oriented in the vertical direction. By rotating the model geometry 50° clockwise around the tunnel center, the new in-situ stress components and gravitational acceleration around the excavation will become as shown in Figure 5.4. The adjusted in-situ stress components and gravitational acceleration are essentially the same as those in Figure 5.3 but represented in a different coordinate system.
Figure 5.3 The geotechnical model and in-situ stress field of the tunnel under gravity of 9.8 m/s².

Based on the equivalent geotechnical model in Figure 5.4, a large-scale UDEC model with a vertical fault close to the tunnel is presented in Figure 5.5. The numerical model has a height and width of 200 m to avoid any influence of the model boundaries on the results near the excavation,
and roller constrains are used for the model boundaries. Given that the full fault length was not directly identified in the field, a fault distance of 60 m was assumed. The mesh used is based on the mesh optimization results in discussed in Chapter 4, where the mesh size of 0.05 m was used in the immediate vicinity of the tunnel. Mesh sizes ranging from 0.05 m to 5 m are applied to the numerical model in Figure 5.5, where denser mesh is assigned to the blocks closer to the tunnel. Since UDEC uses the explicit method to get the solutions, a constant time step of $5 \times 10^{-7}$ second is assigned to all the models in the current case study.

![Figure 5.5 Geometry and boundary conditions of the drainage tunnel based on the equivalent geotechnical model.](image)

The relaxation method describe in Chapter 4 was used to model the excavation process. A total of one hundred relaxation steps were assigned, and each relaxation step was allotted with one thousand time steps in the numerical models, resulting in a total of one hundred thousand time steps or solution time of 0.05 second for the tunnel excavation. The energy components within the entire model are calculated at every 1000 time steps to reduce the numerical computing load. The tracking of energy starts immediately after the removal of the rock material within the tunnel; therefore the initial elastic strain energy of the model does not include the energy in the excavated rock material.
5.3 Numerical simulation results

The fault distance was varied in increments of 0.1 m to identify the critical fault distance, which implies that the maximum precision of the analysis of critical fault distance is 0.1 m in this study. In reality, as shown in Figure 5.2, there is not a fixed fault distance in this rockburst section with a length of 24 m due to the slight angle between the tunnel axis and fault plane strike. The fault distance decreased during advance of the tunnel, and the rockburst event exposed a length of fault with distances from the original excavation ranging from 0.3 m (closest to the face) to 1.4 m (away from the face). Given this information, a critical fault distance between 0.6 m and 0.7 m was selected as a target for the simplified 2D model considered in this study. Additionally, the Richter magnitude 2.0 event measured during the rockburst is utilized to verify the numerical modeling results.

5.3.1 Mechanical cause of the rockburst damage induced by fault-slip

Through an iterative manual back analysis process, the strain-softening parameters were adjusted (ultimately to those shown in Table 5.2) such that rock failure is observed in models with fault distances equal to or less than 0.6 m, while there is no perceptible rock damage for fault distances of 0.7 m and larger. The rockmass behaviors of the model with fault distances of 0.6 m and 0.7 m are presented to reveal the mechanisms of rockburst damage induced by fault-slip.

The distributions of the normal stress, shear stress and shear displacement in the fault plane at equilibrium of the models with fault distances of 0.6 m and 0.7 m are presented in Figure 5.6. The tunnel excavation caused large normal stress reductions in the fault plane close to the tunnel for d = 0.7 m, resulting in small slip movements, and redistributed shear stress of as much as 22 MPa towards the opposite directions in the fault plane. However, the normal and shear stress in the fault plane close to the tunnel become zero, while large shear displacement (as much as 5 cm) in the opposite directions can be observed in two different locations of the fault plane for d = 0.6 m, indicating a total loss of rock strength.
Figure 5.6 Distribution of (a) normal stress, (b) shear stress, and (c) shear displacement along the fault plane at equilibrium of the models with fault distances of 0.6 m and 0.7 m.

The distributions of the displacement magnitudes around the excavation for both models are plotted in Figure 5.7, where a larger contour range is used in the 0.6 m fault distance case to highlight the large displacement within the rock slab between the tunnel wall and fault plane (Figure 5.7a). In the case of the 0.7 m fault distance, the smooth displacement distribution with relatively small magnitudes shown in Figure 5.7b indicates that the displacement mainly results from the elastic deformation of the rockmass after the tunnel excavation. It can be concluded that the rockmass between the tunnel wall and the fault plane failed for \( d = 0.6 \) m, while the tunnel has no perceptible failure at all at \( d = 0.7 \) m.
The tunnel excavation process causes stress redistribution around the openings and slip movement along the fault, while the stress magnitude within the rock slab would be much smaller if there was no fault close to the tunnel. Once the stress within the rock slab exceeds its strength, the rock slab deforms significantly and loses all its strength, in part due to the slip movement in the fault. Rockmass damage induced by nearby fault-slip is a complex process associated with a combination of unstable rock failure within the rock slab and unstable slip failure along the discontinuity. Whether the slab or fault fails first depends on their corresponding contributing factors, however this question is beyond the current scope of this study.

5.3.2 Analysis of the rockburst event using an energy approach

The elastic strain energy, plastic strain work, joint friction work and damped kinetic energy within the entire model are tracked from the first time step after the removal of rock material within the excavation (Figure 5.8). Due to the roller boundary conditions on the four sides of the model, there is no major external work entering the model during the simulation process. Gravity is considered in the numerical models, however the change in gravitational potential energy is only 4.5 kJ during the rock slab failure process. Therefore, the change in gravitational potential energy with solution time is not plotted and analyzed in the energy balance of the entire model.

In Figure 5.8, the changes in each of the energy components follow similar trends before a solution time of 0.24 seconds for the fault distance of 0.6 m, when a large and rapid decrease in
elastic strain energy and increases in the other energy components appear suddenly. Although the solution time in the numerical models cannot be compared directly with physical time, it can still be used to indicate the relative rates of change of stress and strain in the rockmass. It should be noted that the tracked energy components are recorded at every one thousand time steps to reduce the number of plot points, which means that there are one thousand points in each curve during a solution time period of 0.5 seconds due to a constant time step of $5 \times 10^{-7}$ seconds. About twenty recorded points during these sudden changes can be clearly identified in the curves of plastic strain work and joint friction as functions of solution time, indicating that the rock failure occurs during a short period of time.

Figure 5.8 Changes in (a) elastic strain energy, (b) plastic strain work, (c) joint friction work, and (d) damped kinetic energy as a function of solution time.
After the relaxation phase at the solution time of 0.05 s, the unbalance force within the models is still high, so a very large decrease in elastic strain energy and increase in damped kinetic energy can be observed for both models in Figure 5.8. By assuming that both models had the same energy changes before the occurrence of the rock slab failure in the 0.6 m fault distance case, the difference of each energy component between the two models would be equal to the change of energy due to the occurrence of violent rock failure caused by the fault. By subtracting each energy component in the model with a fault distance of 0.6 m from the model with fault distance of 0.7 m at each recorded point, it is possible to obtain the changes in the different energy components with solution time that are specifically associated with the rockburst event (Figure 5.9).

Figure 5.9 Changes in difference of (a) elastic strain energy, (b) plastic strain work, (c) joint friction work, and (d) damped kinetic energy as a function of solution time.
In Figure 5.9, rapid changes in all four energy components can be observed at the solution time-stamp of 0.24 seconds due to the rock slab failure. In addition, the magnitude of the energy transformations associated with the sudden rock slab failure can be directly obtained from these curves. A small magnitude of energy components can exist between these two models before the occurrence of rock failure at solution time of 0.24 seconds, which results from the fault distance difference before the rapid rock slab failure.

When the fault distance difference between these two models is smaller (such as 0.05 m), the energy magnitudes are smaller at the solution time of 0.24 seconds. Ideally, each energy component difference between these two models should be close to zero before the rock slab failure. With the assumption that the changes in difference of different energy components result from the fault distance in these two models, we can approximately estimate that the induced changes in the elastic strain energy, plastic strain work, joint friction work and damped kinetic energy by rock slab failure are about -3.1 MJ, 0.9 MJ, 1.5 MJ and 0.7 MJ, respectively.

Given that that the approximate length of the rockburst damage zone along the tunnel axis was estimated as 24 m after it occurred, the induced changes in elastic strain energy, plastic strain work, joint friction work and damped kinetic energy by the rockburst are about -74.4 MJ, 21.6 MJ, 36 MJ, and 16.8 MJ respectively (24 times the direct numerical results from the two-dimensional numerical results in Figure 5.9). The damped kinetic energy in the numerical models corresponds to the amount of released energy during a seismic or rockburst event in the underground excavations. The conversion from released energy (E) by an earthquake into Richter magnitude (M) of an earthquake is defined as: $M = (\log (E) - 4.4)/1.5$ (Gutenberg and Richter 1956), which translates into a Richter magnitude of 1.9 for the simulated rockburst in this study.

To illustrate the energy source of the rock failure, the distributions of the elastic strain energy density around the excavation with fault distances of 0.6 m and 0.7 m at equilibrium state of the models are plotted in Figure 5.10. The elastic strain energy is concentrated in the rock slab and left tunnel wall in the model with a 0.7 m fault distance, while the elastic strain energy density within the rock slab is close to zero after rock slab failure for the model with a 0.6 m fault distance. The close-to-zero elastic strain energy within the rock slab also means the rock slab has failed and lost its strength. The plot shows that the total elastic strain energy in Figure 5.10b is larger than...
that in Figure 5.10a, and this is due to the release of elastic strain energy during rock failure in the 0.6 m fault distance case.

Figure 5.10 The distribution of elastic strain energy density around the excavation with fault distances of (a) 0.6 m and (b) 0.7 m at equilibrium state of the models.

By subtracting the elastic strain energy density in the model with fault distance of 0.6 m from that with fault distance of 0.7 m, the change in elastic strain energy density within the entire model can be observed (Figure 5.11). The positive and negative elastic strain energy density differences represent the stored and released elastic strain energy in that zone during the rock failure, respectively. The largest release of elastic strain energy for rock failure is from the stressed

Figure 5.11 The elastic strain energy density difference between the models with fault distances of 0.6 m and 0.7 m.
rock slab, and a release of elastic strain energy can also be observed in the upper-left and lower-right of the tunnel in this model. It should be noted that the upper-left and lower-right sections of the tunnel in the numerical models correspond to the roof and floor of the tunnel wall in the drainage tunnel, considering that this model was rotated 50° in the clockwise direction.

The plastic strain work density in the model with a fault distance of 0.6 m is plotted in Figure 5.12, where the maximum work density is $2.5 \times 10^6$ J/m$^2$. During rock failure, the plastic strain work mainly centers on rock slab between the tunnel wall and fault plane, which may represent newly-induced shear plane or fractures in rockmass. The maximum plastic strain work density for the model with a fault distance of 0.7 m is only approximately 300 J/m$^2$.

![Figure 5.12 The distribution of plastic strain work density around the excavation with fault distance of 0.6 m.](image)

The rapid and large changes in elastic strain energy, plastic strain work, joint friction work and damped kinetic energy in the model with a fault distance of 0.6 m indicate that the rock failure can be classified as a rockburst, and the critical fault distance for this tunnel is 0.7 m with 0.1 m precision. Since the average exposed fault plane after rockburst in the drainage tunnel was estimated to lie between 0.6 m and 0.7 m from the excavation, we can be reasonably confident that the model used represents an appropriate approximation of observed field conditions.
5.4 Conclusions

The 11.28 rockburst which resulted in the ejection of large volume of rock fragments in the Jinping drainage tunnel caused seven fatalities and destroyed one TBM machine. This rockburst is thought to have resulted from the slip failure of a major fault sub-parallel to the tunnel axis. When the average fault distance in a simplified 2D model decreases from 0.7 m to 0.6 m, violent rock failure was observed in the rock slab wedged between the tunnel wall and the fault plane in the numerical models. The tunnel excavation and fault plane increase the stress magnitude within the rock slab, resulting in the shear stress and displacement moving in opposite directions in two sections of the fault plane. When the stress in the shortened rock slab exceeds the rockmass strength, the unstable rock slab failure occurs.

The rapid and large changes in elastic strain energy, plastic strain work, joint friction work and damped kinetic energy indicate that the rock failure can be classified as a rockburst event. Further, the release of elastic strain energy in the surrounding rock, especially in the rock slab, is the primary energy source for violent rock failure. Moreover, the Richter magnitude and fault distance from the numerical result are similar to the field measurements for the 11.28 rockburst event. The similarities indicate that the energy technique developed and applied in UDEC can be used to reliably simulate rockburst events and to back-analyze rockburst occurrence in deep tunnels near major discontinuities.
CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

In this thesis, the indicators and contributing factors of unstable rock failure in intact rock and along a discontinuity were analyzed to gain an insight into the mechanisms of rockburst. The unstable failure in intact rock and along discontinuity is based on laboratory compressive strength test and direct shear test configurations, but the contributing factors from these numerical results can help mining and geotechnical engineers gain insights into the question on what kinds of mechanical parameters promote rockburst potential. The stability of a circular tunnel near a major discontinuity was analyzed in different scenarios to understand the interaction between the rock slab failure and discontinuity slip failure. The parameter of critical discontinuity distance is proposed to quantify the rockburst potential of deep excavations near major discontinuities. The influence of unfavorable stress and geological conditions can be interpreted directly from the values of critical discontinuity distance in different scenarios.

With a better understanding of the rockburst mechanisms, a documented rockburst event occurred at the Jinping II drainage tunnel in China was simulated. The Richter magnitude and critical fault distance output from the developed approach were utilized to compare with the field measurements. This Chapter summarizes the key findings obtained from each chapter of the thesis, and some suggestions for future work are provided.

6.1 Conclusions

To make a distinction between stable and unstable rock failures in numerical models, a rock failure is defined as unstable failure when over 50% of elastic strain energy (or joint shear strain energy) from its peak to residual value within a target rock is released instantaneously during the rock failure process under displacement boundary conditions. For models with roller or fixed constraints such as model of excavation near a discontinuity in Chapter 4 & 5, the rock failure can be identified as a rockburst when a large amount of elastic strain energy is released due to a small disturbance.
In Chapter 2, the equations of different energy components were integrated into UDEC to enable the tracking of the energy transformations between different energy components during rock failure. Unstable rock failure during uniaxial compression was simulated in UDEC, where rapid and large changes in total strain energy, elastic strain energy and plastic strain work during rock failure were identified as reliable indicators of unstable rock failure. The magnitudes of these rapid changes can also be used to represent unstable rock failure intensity. From an energy perspective, the occurrence of unstable rock failure depends on the available elastic strain energy for plastic strain work during rock failure, while released kinetic energy results from the energy balance between available elastic strain energy and induced plastic strain work. Although a simplified UCS test configuration is different from the traditional UCS test with loading machines in laboratory, this hypothetical model still can be utilized to analyze the fundamental contributing factors of unstable rock failure. A smaller loading system stiffness (LSS) and rock stiffness can increase the magnitude of stored elastic strain energy available to be transferred to the plastic strain work and kinetic energy, while a more brittle rock dissipates less energy through plastic strain work during its failure. Therefore, unstable rock failure tends to occur in brittle rocks that are relatively soft under a soft loading system.

In Chapter 3, unstable slip failure along a pre-existing discontinuity under high stress was analyzed by tracking the energy in both the rock matrix and discontinuity in a simulated direct shear test to gain insight into the affecting factors of fault-slip. This model configuration is based on the laboratory direct shear test configuration, however internal isotropic stress is assigned to the model to simulate the stress conditions in deep rock. Slip failure modes along pre-existing discontinuities depend on the available elastic strain energy in the rock matrix and discontinuity for joint friction work, if inelastic damage to the intact rock matrix is not considered. The magnitude of elastic strain energy in the rock matrix at failure increases with decreasing rock stiffness, leaving more stored energy available to be transferred to the discontinuity undergoing slip during failure. The amount of joint shear strain energy at the peak strength is larger when the joint stiffness is smaller and the joint roughness is larger (assuming the same peak strength); however a discontinuity with a smaller stiffness and larger roughness has a much larger weakening shear displacement from its peak strength to residual strength, resulting in a larger amount of joint friction work occurring during this weakening process. An increase in normal stress acting on a discontinuity can increase the amount of stored energy in the discontinuity and the neighboring
rock, although this increased energy storage only translates to an increased propensity towards unstable slip failure up to a point. Beyond a certain threshold of normal stress, unstable slip failure is suppressed due to the increase in joint ductility that occurs with increased normal stress. In summary, unstable slip failure tends to occur along a discontinuity with large shear stiffness embedded in a soft rock matrix, and tends to be restricted if the normal stress is too large.

In Chapter 4, a circular excavation near a major discontinuity was simulated to study the rockburst damage induced by slip along a nearby geological discontinuity. The slip along the discontinuity can increase the stress concentration in the rock slab bounded by the discontinuity and the excavation, and the failure of this slab results in a large and rapid increase in shear displacement along the discontinuity. In turn, the rapid shear displacement within the discontinuity exacerbates the stressed rock failure intensity in the rock slab, causing the ejection of rock fragment into the excavations as a result of the release of the elastic strain energy from the surrounding rock. This work indicates that the distance of the discontinuity plays a critical role in affecting burst potential, and a larger critical discontinuity distance represents a relatively higher burst potential. The sensitivity analysis of different rockburst contributing factors shows that the critical discontinuity distance increases with the horizontal-to-vertical in-situ stress ratios, stress magnitude, tunnel radius (and hence excavation volume) and discontinuity length; moreover, the critical discontinuity distance decreases with the angles between the discontinuity plane and the major principal stress. The discontinuity length can influence the critical discontinuity distance, but this influence is negligible when the length is above a certain magnitude.

In Chapter 5, the application of the modeling approach developed in this thesis is illustrated with the 11.28 rockburst that occurred in the Jinping II drainage tunnel. In the case study model, when the excavation to fault distance decreases from 0.7 m to 0.6 m, a large and violent rock failure is observed in the excavation, which is consistent with the field observation that the advance of the tunnel face (decreasing fault distance) caused the rockburst in the drainage tunnel. In addition, the Richter magnitude and fault distance from the numerical result is close to the field measurement of the fault-slip event, which suggests that the energy approach can be used to reliably simulate large-scale rockbursts in deep mines and tunnels.
6.2 Recommendations for future work

The energy approach developed and applied herein can greatly benefit current understanding of the underlying mechanisms of rockbursts. However, as discussed in Chapter 1, rockbursting in deep mines and tunnels is such a complex phenomenon that caution should be exercised when analyzing mine-scale rockburst cases using any approach. There are still many unresolved issues which should be addressed before a comprehensive understanding of rockburst occurrence in deep mines and tunnels can be achieved. In particular, some recommendations for future studies are as follows:

• The influence of the interaction between rock discontinuities may be critical to the occurrence of rockbursting. Multiple discontinuities such as joint sets and bedding planes could be integrated in numerical models to study the interaction between rock structures and their influence on rockburst in deep mines and tunnels.

• The influence of rock support systems in reducing the rockburst damage can be analyzed to benefit mining or geotechnical engineers to take preventative measures.

• The research in this dissertation is based on either quasi-static loading conditions or static stress conditions; in other words, potential rockburst triggering factors such as seismic activities were not simulated and analyzed.

• The strain-softening constitutive model and CY joint model were selected to simulate the post-peak behaviors of rock material and discontinuities in this dissertation respectively. Some other models such as the cohesion-weakening-friction-strengthening strength model also may be utilized to simulate rockbursts in deep hard-rock mines and tunnels.

• Simulations in the current work were conducted using the 2D numerical software UDEC. However, three-dimensional discontinuity, excavation, and stress geometries are likely to influence rockburst potential and should be studied in the future.
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