ASSESSMENT OF ROOF STABILITY IN A ROOM AND PILLAR COAL MINE IN
THE U.S. USING THREE-DIMENSIONAL DISTINCT ELEMENT METHOD

by

Taghi Sherizadeh

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As members of the Dissertation Committee, we certify that we have read the dissertation prepared by Taghi Sherizadeh, titled “Assessment of Roof Stability in a Room and Pillar Coal Mine in the U.S. Using Three-dimensional Distinct Element Method” and recommend that it be accepted as fulfilling the dissertation requirement for the Degree of Doctor of Philosophy.

Date: (06/08/2015)

Pinnaduwa H. S. W. Kulatilake

Date: (06/08/2015)

Moe Momayez

Date: (06/08/2015)

Craig A Wissler

Date: (06/08/2015)

Jaeheon Lee

Final approval and acceptance of this dissertation is contingent upon the candidate's submission of the final copies of the dissertation to the Graduate College.

I hereby certify that I have read this dissertation prepared under my direction and recommend that it be accepted as fulfilling the dissertation requirement.

Date: (06/08/2015)

Dissertation Director: Pinnaduwa H. S. W. Kulatilake

STATEMENT BY AUTHOR

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Taghi Sherizadeh

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Abstract

Roof falls and accumulation of dangerous gasses are the most common hazards in any underground coal mine. Different mechanisms can jeopardize the stability of the roof in underground excavations and successful roof control can only be obtained if the failure mechanism is identified and understood properly.

The presence of discontinuities, the inherent variability of the rock mass and discontinuity properties, and the uncertainties associated with directions and magnitudes of the in-situ stress makes the rock engineering problems challenging. The numerical modeling can assist the ground control engineers in designing and evaluating the stability of the underground excavations. If extensive geological and geotechnical data are available, then detailed predictions of deformation, stress and stability can be accomplished by performing numerical modeling. If not, still the numerical modeling can be used to perform parametric studies to gain insight into the possible ranges of responses of a system due to likely ranges of various parameters. The parametric studies can help to identify the key parameters and their impact on stability of underground excavations. The priorities of the material testing and site investigation can be set based on the selected key parameters from parametric studies.

An underground coal mine in western Pennsylvania is selected as a case study mine to investigate the underlying causes of roof falls at this mine. The immediate roof at the case study mine consists of laminated silty shale, shale, or sandstone that changes from area to area, and the floor is shale or soft fireclay. This study was mainly focused in the stability analysis of the roofs with the laminated silty shale rock type, where the majority of roof falls had taken place in the roof with this type of roof material. Extensive laboratory tests

were performed on the core samples obtained from the case study mine to estimate the intact rock and discontinuity properties of the materials that occur in large extent at the selected interest area of the case study mine.

In this research, the three-dimensional distinct element method was used to investigate the stability of the roof in an underground room-and-pillar coal mine. The implemented technique was able to accurately capture the failure of the major discontinuities and rock masses which consist of intact rock and minor discontinuities. In order to accurately replicate the post failure behavior of the rock layers in the immediate roof area, the strain-softening material constitutive law was applied to this region.

Extensive numerical parametric studies were conducted to investigate the effect of different parameters such as the variation of immediate roof rock mass strength properties, variation of discontinuity mechanical properties, orientations and magnitudes of the horizontal in-situ stresses, and the size of pillars and excavations on stability of the excavations. The distribution of post failure cohesion along with other measures such as accumulated plastic shear strain, distribution of Z-displacements at the roofline, failure state (joint slip and tensile failure) and displacement (normal and shear displacements) of discontinuities were used to accurately assess the roof stability in this case study.

The research conducted in this dissertation showed that the bedding planes play an important role on the behavior of roof in underground excavations. Therefore, an appropriate numerical modeling technique which incorporates the effect of discontinuities should be employed to simulate the realistic behavior of the discontinuous rock masses such as the layered materials in roof strata of the underground coal mines. The three-dimensional distinct element method used in this research showed the clear superiority of this technique

over the continuum based methods.

Chapter 1

Introduction

1.1 Introduction

Roof falls have been one of the most common ground control complications affecting the safety and economy of underground coal mines. Although, over the last decade there have been major improvements in the way which coal mines have managed the roof fall hazards through the application of safety measures, technological improvements, alternative mining methods and extensive support systems, still, roof fall accidents remain a leading cause of coal mining injuries. During 2013, 265 miners were injured in rib and roof falls in the USA according to the U.S. Department of Labor's Mine Safety and Health Administration (MSHA). Therefore, it is obvious that a tangible advancement should take place in the ground control area before the mining industry targets a chance at zero incidents.

Roof stability is associated with the ability of the roof rock to span the openings and is dependent, in part, on the size of the opening, different rock mass material strengths and deformability, geologic discontinuity geometry and their mechanical properties, in-situ stress

system, possible groundwater conditions and possible dynamic loading. Previous studies which are mainly based on empirical methods or limited numerical modeling, attributed the most of the roof fall to the existence and adverse orientation of the high horizontal stress and the mechanical properties of the rock masses.

An underground coal mine in western Pennsylvania is selected as a case study mine to investigate the underlying causes of roof falls at this mine. The case study mine uses the room-and-pillar mining method and over the last ten years more than 40 roof falls have occurred in this mine. The immediate roof in this mine consists of laminated silty shale, shale, or sandstone that changes from area to area, and the floor is shale or soft fireclay. This study is mainly focused in the stability analysis of the roofs with the laminated silty shale rock type, where the majority of roof falls had taken place in the roof with this type of roof material.

Based on the previous studies and our intuitive conclusions, the magnitude and orientation of the in-situ horizontal stresses, the geo-mechanical properties of the discontinuities and the geologic strata associated with the roof strata, and the size of the rooms and pillars are the dominant factors affecting the stability of the roof. In this research, which is based on advanced three dimensional discontinuum numerical modeling, not only the effect of magnitude and direction of in-situ horizontal stress, but also the effect of different parameters such as, the size of excavations and pillars, and the variation of rock mass and discontinuity mechanical properties are examined.

1.2 The Need and Significance of Current Research

The negative effects of the roof failures on underground coal mines has been known as an important issue for many years, but the progress made towards solving this issue was not enough to successfully resolve the problem. The shortcomings of applied empirical and numerical techniques to address the issue, and inappropriate methods used to estimate the mechanical behavior of the rock mass and discontinuities are the main reasons underlying this impasse. In brief, the success of the outcomes of a numerical model is governed by two main factors, namely, the quality of the input and the suitability of the selected method.

The literature review of past studies showed that the empirical techniques and continuum based numerical methods that have limited capability are the most commonly used techniques for addressing the roof stability problem in underground excavations. In the last decade, the Coal Mine Roof Rating (CMRR) method became the most popular empirical technique for evaluating and designing the underground excavations in coal mines in USA. The CMRR is a revised version of the rock mass classification systems and is developed empirically from a database of coal mines in the USA. This technique weighs some of the geotechnical factors that may affect the competency of mine roof and combines them into a single number rating on a scale of 0 to 100. This technique inherits all the shortcomings and disadvantages of its parent system, known as the rock mass rating (RMR) system. The author believes that the simplicity and ease of use are the main reasons that in recent years the US underground coal industry has wholeheartedly embraced this system as the most reliable method of geotechnical characterization. However, the blind use of this technique on its own could potentially lead to imperfect geotechnical characterization and design. The drawbacks of the rock mass classification systems have been known to respected rock

mechanics experts for a long time. Brady and Brown (1985) stated, “Although the use of this approach is superficially attractive, it has a number of serious shortcomings and must be used only with extreme care. The classification scheme approach does not always fully evaluate important aspects of a problem, so that if blindly applied without any supporting analysis of the mechanics of the problem, it can lead to disastrous results”. As a result of random variation in the point load test data and variability in the methodology, the value of CMRR can vary up to 41 points. This extreme variability alone can result in totally different understanding of rock mass strength. Inadequacy of de-rating of low UCS roof, ignoring the stiffness of materials and the deficiency of inclusion of the frequency of weak un-fractured planes are other downsides of the CMRR method ([Calleja, 2008](#)).

FLAC3d (finite difference element based software developed by ITASCA) and ABAQUS (finite element based program developed by Dassault Systems) are the most widely used programs in academia to address the roof stability problem in underground coal mines. It is well known that the discontinuities not only are weakening the rock structure but also they are changing the mechanical behavior of rock masses. In essence, the slip or separation of existing discontinuities in roof strata can amplify the negative effects of stress concentrations around the excavations. Laminated or thinly bedded shale is one of the most frequently seen overlaying strata above the coal deposits. It is well understood that the bedding planes and interfaces have very low or close to zero tensile strength in the direction perpendicular to the bedding planes and their shear strength is much lower than of the rock layers. As a result, the bedding planes are much weaker than the rock layers. Therefore, due to the stress concentrations caused by excavations, slippage and separation can easily happen along the bedding planes before initiation of failure within the rock layers. Both of the aforementioned computer programs are developed based on continuum

mechanics concepts. The continuity of deformations is the intrinsic assumption of the continuum theory and clearly it is not necessarily valid for discontinuous materials such as layered rock masses. In the presence of discontinuities, deformations can be discontinuous across the discontinuities; these displacements can be large. Also, rotation of rock blocks is possible. Therefore, all continuum based methods cannot capture the real behavior of jointed rock masses. The aforementioned two computer programs have limited capability for simulating the discontinuities using the interface (FLAC3D) and contact elements (ABAQUS). In both of these computer programs, implementing the stated techniques for incorporating the discontinuities in the model has major shortcomings. The interface elements first should be defined geometrically and then meshed separately. This process is rather complicated and only can be applied to limited number of interfaces. It is clear that inappropriate selection of interface elements will lead to erroneous results. The ubiquitous joint model is another available technique in FLAC3D for incorporating the discontinuities but is limited to only one joint set and also it cannot capture the anisotropy in elastic properties due to the existence of discontinuities. The validity limits and the concerns over the computational aspects of continuum based methods for handling the discontinuous materials were the key factors convinced the author to avoid using this technique for serving the objectives of the current research. It is clear that due to the discontinuous nature of the rock mass, discontinuum based models can provide a more accurate description of the physical behavior of jointed materials. In this research, a three-dimensional distinct element program (3DEC), which is based on the distinct element method, was used for numerical analysis. In contrast with continuum based programs, in this program the discontinuities are represented explicitly and it does not have the emphasized limitation of the continuum based programs. To the best of the author's knowledge, this is the first time that this

technique is applied to address the roof stability problem (stability of crosscuts, entries and intersections) in room-and-pillar coal mines.

The potential deformation and failure mechanisms of the jointed rock masses are predominantly governed by their response to the induced stresses resulted from excavation and their associated strength and deformational behavior. The failure around underground excavations is a result of failure in both the rock mass part and major discontinuities. In order to realistically predict the failure of rock mass part, the implemented material constitutive law in the numerical model should be able simulate the failure mechanism as closely as possible. Out of the different available material constitutive models, the Mohr-Coulomb elastic perfectly plastic model is the most commonly used constitutive law in rock engineering designs. In the Mohr-Coulomb model the strength properties of the materials are assumed to remain constant after the onset of plastic failure. This assumption means that the material behaves in elastic-perfectly plastic manner by maintaining its strength after failure and is able to support a stress equal to the failure strength. In reality, this stress should get transferred to the surrounding zones. Therefore, the Mohr-Coulomb model is most suitable for the problems where the post failure response of the material is less important. The roof failure in underground excavations is most likely a progressive failure that starts at the weakest part of the rock mass where the stress concentrations are higher than the roof rock strength and then propagates in the least resistance path within the rock mass. Because of the aforementioned limitations of the standard Mohr-Coulomb constitutive law, it is impossible to simulate this process using this model and the strain-softening constitutive law is well-suited for this purpose. The Mohr-Coulomb strain softening model is based on the Mohr-Coulomb model with possible degradation of cohesion, friction angle, tensile strength and dilation after the onset of plastic yield, while in

the standard Mohr-Coulomb model those parameters are assumed to remain constant. The majority of the previous numerical studies for simulating the roof stability in underground coal mines are based on elastic material behavior, only a few of them are based on elastic perfectly plastic constitutive laws and the strain softening constitutive law is rarely used. In this research, a five-segment piecewise-linear strain softening model was employed for the material at the immediate roof area and for the rest of the materials the standard Mohr-Coulomb constitutive model was assigned. The distribution of post failure cohesion along with other indicators such as accumulated plastic tensile and shear strains, and yield state of elements were used to accurately assess the occurrence of failure in roof rock strata.

The accuracy of the results predicted by a numerical model is highly dependent on the quality (aspect ratio, orthogonality, and skewness), size and type of the elements used to mesh the model. However, this important aspect of numerical modeling has not received enough attention from researchers in the rock mechanics literature. The four-node constant strain tetrahedral element is the most commonly used element type in three-dimensional distinct element analysis. In spite of its wide usage, this type of element is known to possess some crucial shortcomings such as lower quality stress prediction, overly stiff behavior, and volumetric lock in nearly incompressible cases. In this research, the higher-order elements are used to accurately capture the post-failure response of the rock mass. The laminated nature of the coal mine roof strata makes it crucial to consider the bending and buckling behavior of the thinly bedded layers properly in the numerical modeling. To accurately capture the bending and buckling behavior of the thinly laminated materials at the roof strata, each individual layer is meshed such that at least five overlaying elements are created across the section of each thin layer. The required number of elements across the section width of thinly laminated layers is calculated from performed sensitivity analysis in 3DEC.

Similar to other engineering problems, the ‘nonsense in, nonsense out’ (NINO) principle holds true for rock mechanics modeling too. The high variability in quality, having been subjected to the disruption of nature over millions of years, is one of the main characteristics of the rock mass. The small rock samples may be tested to any level of accuracy, but the rock quality a few meters away from where the specimen is taken might be different. In this research, to account for the uncertainties associated with the rock mass and major discontinuity properties a sensitivity analysis model has been developed to incorporate such uncertainties in the established numerical model. Since most of the coal mines are located in sedimentary environments, it is usually sufficient to start the numerical simulations based on average rock mass and discontinuity properties. Therefore, in this study a base model was developed based on the average rock mass and discontinuity properties and then it was followed up by different parametric studies to examine the impact of variation of each individual parameter on stability of the roof. In addition to the uncertainties associated with the rock mass and discontinuity properties, the variability of the orientation and magnitude of horizontal stresses is another important factor that impacts the stability of the underground excavations. Since no horizontal stress measurement was conducted at the case study mine, the magnitudes and orientations of the in-situ principal stresses were not known. Therefore, in this research several parametric studies were performed for different orientations and magnitudes of the horizontal in situ stresses. The size of the pillar and roof span are the other factors that highly impact the stability of the roof in underground coal mines and their influence on overall performance of the excavations is researched in detail in this study.

1.3 Organization of the Dissertation

This dissertation is organized into five chapters, including this introduction (Chapter 1). In addition to the main chapters, there is an appendix that covers the lab test results and is referred to as Appendix-A in the text.

Chapter 2 covers a brief review of the literature related to the studies done in the past to address the roof stability problem in underground coal mines. Different numerical techniques adopted in the past to address the stability of excavations in underground coal mines along with their shortcomings are discussed. Some numerical modeling related issues are explained in brief and then sufficient details on selection of a most suitable numerical method to use in this work are provided.

Chapter 3 reviews the standard laboratory testing procedures for estimating the physical and mechanical properties of intact rock and discontinuities. The summary of the laboratory test results estimated from the standard tests such as Brazilian tensile test, unconfined compressive strength (UCS) test, triaxial compression test, direct shear test of discontinuities, as well as the specially designed test to measure the joint normal stiffness are covered in this chapter.

Chapter 4 presents the numerical modeling undertaken to examine the parameters that broadly impact the stability of excavations in room-and-pillar coal mines. Initially the three-dimensional distinct element code, 3DEC, is introduced and some of its significant features are discussed briefly. The procedure of building the numerical model and adoption of the appropriate constitutive models, and mechanical properties of intact rock and discontinuities are also covered in this chapter.

Chapter 5 covers the summary and general conclusions of the undertaken research. This chapter also includes the suggestions given for further research work on the topic dealt with.

Chapter 2

A Review of Numerical Modeling of Roof Instability in Room-and-Pillar Coal Mines.

2.1 Introduction

The rock masses are best described as discontinuous, inhomogeneous, anisotropic and non-elastic (DIANE) materials (Harrison and Hudson, 2000). Unlike the manufactured materials such as metals or plastics, the engineering and physical properties of rock masses cannot be defined or established easily. Therefore, the rock masses are complex materials for mathematical modeling in closed forms, and numerical modeling becomes inevitable for the design and assessment of rock engineering projects ([Jing and Stephansson, 2007](#)).

Under the action of in situ stresses, the rock masses are in equilibrium state prior to any underground excavations. The equilibrium state will be impaired once the excavations are introduced within the rock masses. Generally, due to the heterogeneous and the discontinuous nature of the rock masses, different types of deformation and failure could happen around the excavations. To estimate the location and severity of the failures, and to calculate the magnitude of displacements around the excavations, it is necessary to perform

numerical or analytical analysis.

It has been proven that the numerical techniques are powerful and effective tools for analysis and designs of rock engineering structures. The most applied numerical methods for solving the rock engineering problems can be categorized in three main groups ([Elmo, 2006](#)):

- Continuum methods: Finite Element Method (FEM), Boundary Element Method (BEM) and Finite Difference Method (FDM).
- Discontinuum methods: Discrete/Distinct Element Method (DEM) and Discontinuous Deformation Analysis (DDA).
- Hybrid methods: Hybrid FEM/BEM, Hybrid BEM/DEM, Hybrid FEM/DEM and other hybrid methods.

In this chapter, different numerical methods and their applications for modeling of roof behavior in underground coal mines are discussed briefly. From literature review for each numerical method some case studies are covered from previous research.

2.2 Continuum Methods

In continuum models, the problem domain is divided into smaller elements where their behavior can be estimated using simplified numerical techniques with finite degrees of freedom. This procedure is known as discretization and allows to model the continuum domain with infinite degrees of freedom using the discrete system with finite degrees of freedom ([Helm, 2011](#)). The continuum hypothesis implies that the deformation of all points in a problem region should be continuous. Therefore, all the nodal points forming the mesh

are always shared with other points and during the deformation process they should still stay in the same neighborhood. Thus, in continuum method the rotation and detachment of the mesh is not allowed (Gao, 2013). The main problem with the continuum method is that the assumption of a continuum is not always realistic for simulating the large deformations and the failure surface is not very distinct. Several developments have been proposed to improve the shortcomings of this method. New constitutive models are introduced to better replicate the real behavior of rock masses. Some other developments such as strain softening model have been suggested to simulate the localization of shear bands in the material (Sjoberg, 1999).

Common continuum methods include the Boundary Element Method (BEM), Finite Element Method (FEM), and Finite Difference methods (FDM), which are briefly explained in the following sections.

2.2.1 Boundary Element Method (BEM)

In the Boundary Element Method (BEM) only the boundaries of the problem geometry, such as excavation surfaces, and the material interfaces, are discretized and the interior of the domain is represented mathematically as an infinite continuum. This method is more suitable to solve the problems that the material is homogenous and has linear elastic behavior. The BEM is not efficient in dealing with non-linear and heterogeneous materials. In comparison to other numerical methods, BEM has a fast computing speed and easy mesh generation. The major application of this method is to evaluate the stress distributions around the underground openings. The boundary element method also can be used to perform simple deformation and failure analysis. The system of this method is elastic and therefore the estimated deformations are only the elastic component of the deformation.

Generally, in this method for identifying the failure zone around the excavations the ratio of the rock mass strength to the stress is used as the criterion.

[Gale and Blackwood \(1987\)](#) conducted a series of three-dimensional stress analysis using the boundary element method to determine the stress distribution in the vicinity of a rectangular mine face. From numerical modeling results, they concluded that the stability of the roof above the roadway is significantly influenced by the orientation of the entries with respect to the in situ principal stresses. Their numerical modeling results showed that the entries aligned in the direction of the maximum horizontal stress would be more stable than those oriented in the direction of the minimum horizontal stress.

Based on two-dimensional boundary element analysis, [Ahola et al. \(1991\)](#) and [Kripakov \(1982\)](#) suggested that driving an arched entry in adjacent of rectangular entries and allowing it to cave will significantly reduce the high shear stresses generated by excessive horizontal stresses at the upper corners of the entries. In this study the effect of caving height and slip along the bedding planes were both considered. It has been shown that the slippage of immediate roof along the horizontal bedding planes into the caved entry reduces the magnitude of the shear stresses near the adjacent entries.

[Clifford \(2004\)](#) applied the boundary element method to evaluate the three-dimensional stress redistribution around a coal longwall panel. Then he used the stress output of the boundary element method as input for the subsequent two-dimensional finite difference modeling of the roadway. He suggested that the results from the combination of different modeling methods would provide useful insight for studying a particular problem.

2.2.2 Finite Element Method (FEM)

The Finite Element Method is the most commonly used technique for analyzing the geotechnical problems. The FEM has been applied extensively to solve practical problems. Therefore a lot of experience is available for different type of problems. In this method the physical problem is modelled numerically by discretizing the problem domain into small sized and standard shaped elements. Compared to the boundary element method, FEM has a good flexibility for dealing with heterogeneous and non-linear materials. But this method is generally not suitable for analysis of the rock masses for which the behavior is mainly governed by randomly oriented joint sets.

[Wang et al. \(1974\)](#) used two-dimensional finite element modeling to study the effects of changes in mechanical properties of roof and coal layers on the stress distribution around a single rectangular coal mine opening in a multilayered rock system. They concluded that the variability in the stiffness and relative thickness of the immediate roof layers produce a significant difference in the stress distribution around the opening as compared with that of homogenous material ([Wang et al., 1974](#)) .

[Kripakov \(1982\)](#) conducted further two-dimensional finite element analysis to investigate the roof failure in underground coal mines. He suggested that high shear stress concentration at the upper corners of the rectangular excavations causes cutter type failure in the immediate roof area, as earlier suggested by previous researchers ([Peng, 1978](#); [Nicholls, 1978](#)).

[Su and Peng \(1987\)](#) performed extensive numerical modeling using two-dimensional finite element methods to identify different factors effecting the roof failures in underground coal mines. They examined the effect of high horizontal and vertical stresses, relative stiffness between the coal and its immediate roof, bed separation and geological anomalies

on the behavior of roof strata. They found that the high vertical stress is the major factor controlling the behavior of immediate roof at entry corners, while the direction and magnitude of horizontal stresses are controlling the nature and location of roof failure occurrences in the coal mine entries (Su and Peng, 1987).

Stankus and Guo (1997) conducted three-dimensional finite element analysis to study the horizontal stress effect on roof stability. They identified the high horizontal stress field as an important factor contributing to the roof instability. Based on numerical modeling results, they provided a guideline for orienting the panel in a favorable direction to avoid the roof instability resulting from high horizontal stress concentration (Stankus and Guo, 1997).

Morsy and Peng (2005) used the three-dimensional finite element software (ABAQUS) to analyze the stress distribution at the headgate of a longwall panel subjected to high horizontal stress at various angles. They found that the directions of induced principal stresses at the headgate and longwall face are independent from the panel orientation.

2.2.3 Finite Difference Method (FDM)

The main difference between the finite difference and finite element method is the implemented explicit solution scheme to solve the weak form of the differential equations and other than that these two methods are mathematically identical. In the finite difference method (FDM) the continuum domain is represented by a series of discrete grid points where the displacements, velocities, and accelerations are calculated. With implementation of the explicit solution scheme, the finite difference method reduces the processing time and the memory requirement by avoiding the solution of large sets of equations. The process of

linear simulations with the finite difference method is slower than that of the finite element method. Therefore the FDM is well suited for solving nonlinear or large-strain problems, or to solve physically unstable problems ([Itasca, 2013](#)).

[Meyer et al. \(1999\)](#) used FLAC3D, a three dimensional finite difference software, to study the effects of high horizontal stress direction on roadway stability of United Kingdom coal mine roadways. From numerical modeling results they deduced that three-dimensional modeling can provide a significant contribution to understand the failure mechanisms around coal mine roadways. They used elastic-perfect plastic material behavior for performing the modeling. Their modeling results confirmed the previous research results regarding the pronounced effects of high horizontal stress on stability of roadways ([Meyer et al., 1999](#)).

[Sharpe \(1999\)](#) applied FLAC code to model the deformation and failure of roadways in an underground coal mine. He introduced the bedding joints in the model using the interface and ubiquitous joint logic. Parametric studies were performed to examine a range of factors influencing the stability of roadways ([Sharpe, 1999](#)).

[Gadde and Peng \(2005\)](#) performed three-dimensional finite difference analysis using FLAC3D software with strain softening material behavior to simulate the cutter roof in an underground coal mine. They suggested that strain softening material model can help to simulate the cutter roof failure up to a satisfactory level ([Gadde and Peng, 2005](#)).

Three-dimensional finite difference modeling was used by [Ndlovu and Stacey \(2007\)](#) to investigate the roof cutter problem in a South African underground coal mine. They used FLAC3D code with various constitutive material models such as elastic, elastic-perfect plastic, and strain-softening with and without an interface at the coal-shale contact. But they concluded that the applied constitutive material models are not as efficient as

“extension strain criterion” to accurately predict the location and depth of the cutter roof (Ndlovu and Stacey, 2007).

In the most recent study, Ray (2009) performed finite difference numerical modeling to investigate the influence of cutting sequences on cutter development and roof fall in underground coal mines. He assigned the strain softening material constitutive model to the roof material and elastic model to the pillars. He suggested that assigning the strain-softening constitutive behavior to the roof materials results the most realistic cutter pattern. He also found that the relative magnitude of the in situ stresses has significant influence on the severity of the cutter roof. He also concluded that the sequence of excavation is another important factor influencing the cutter roof pattern.

Other researchers (Zipf et al., 2006; Esterhuizen and Mark, 2009; Mark and Pierce, 2009; Helm, 2011) also used the finite difference method with different type of constitutive material models to investigate the stability of underground excavations and they ended up with almost the same findings.

2.3 Discontinuum Methods

The continuum methods described so far are limited to problems where the behavior of the rock mass is not governed by discontinuities. As mentioned before, due to the fundamental assumptions of continuum mechanics, the deformation of all points in a continuum region should be continuous. Therefore, continuum-based numerical methods have limited applicability for simulating the underground openings where opening or sliding along pre-existing discontinuities as well as fracturing of intact rock plays a dominant role. Most of the boundary element, finite element and Lagrangian finite difference programs

are capable of representing a limited number of discontinuities using “interface element” or “slide lines” logics. However, they cannot handle large number of intersecting discontinuities. Also, they are limited to only small deformation and rotation calculations. Finally, they are unable to automatically detect the creation of new contacts between blocks that develop during the simulation ([Itasca, 2013](#)).

Different numerical modelling programs have been developed to overcome the mentioned limitations of continuum methods. The most common discontinuum methods include the Distinct Element Method (DEM) and the Discontinuous Deformation Analysis (DDA) Method which are briefly explained in the following sections.

2.3.1 Distinct Element Method (DEM)

In the distinct element method, the discontinuous medium is represented as assemblages of discrete blocks which might be considered as rigid or deformable and discontinuities are treated as interfaces (boundary conditions) between distinct blocks. The motion along the discontinuities is governed by linear and non-linear force-displacement relations for movements in both the normal and shear directions. In continuum methods, the motion of individual element is not independent but constrained by other neighboring elements connected within its boundaries. In discrete system, the distinct blocks are allowed to move independently according to the force (or stress) constraints on their boundary surfaces and other external loads, according to the equations of motion ([Itasca, 2013](#)). The contact forces and displacements at the interfaces of blocks are found from a series of calculations which traces the movements of blocks. The calculations are based on time-stepping algorithm and the size of the time-step is selected such that the velocities and accelerations within the time-step could be assumed constant and during a single time-step the disturbances cannot

propagate from one discrete element further than its immediate neighbors. The time-step restriction applies to both blocks and contacts. For non-deformable (rigid) blocks, the block mass and interface stiffness between blocks define the time-step limitation. For deformable blocks the time-step limitation comes from the zone size, and the stiffness of the system includes the contributions of both the intact rock modulus and stiffness at the contacts.

The calculations sequence in DEM is a dynamic procedure that alternates between application of Newton's second law of motion at the blocks and force-displacement law at the contacts. The contact forces are calculated using the applied force-displacement law from contact displacements. Newton's second law of motion gives the motion of the blocks resulting from the forces acting on them. For deformable blocks, the motion is calculated at grid-points of the constant-strain elements within the blocks. Then, from prescribed block constitutive model the new stresses are calculated within the elements ([Hart et al., 1993](#)).

The UDEC and 3DEC, two- and three-dimensional codes from ITASCA, are the most common distinct element based software for modeling the rock mechanics problems. [Agapito and Gilbride \(2002\)](#) used UDEC to study the influence of high horizontal stress on roof stability in underground coal mines. The roof material represented as laminated rock mass with vertical crossing joints. Different types of roof ground support were tested with the model. They concluded that under some stress conditions, roof instabilities might be avoided if appropriate stress measurements are taken into account in support design (see [Figures 2.1 and 2.2](#)).

[Alejano et al. \(2008\)](#) used UDEC to study the roof bed failure in stratified rock masses (see [Figure 2.3](#)). They showed that how relaxation of confining stress can play an important role in roof bed deformation ([Alejano et al., 2008](#)).

Gao (2013) used 3DEC (3-Dimensional Distinct Element Code) to simulate the cutter roof failure in underground coal roadways. He adopted the three-dimensional Trigon approach for representing the immediate roof as an assembly of tetrahedral blocks bonded together through the contact surfaces between them. He concluded that by applying the three-dimensional Trigon logic it is possible to capture the cutter roof failure caused by oblique horizontal stress with respect to the roadway direction (see Figure 2.4).

Figure 2.1: Stable roof support for 2:1 horizontal to vertical stress ratio (Alejano et al., 2008).

Figure 2.2: Unstable roof support for 3:1 horizontal to vertical stress ratio (Alejano et al., 2008).

Figure 2.3: Simulation of roof failure in stratified rock masses ([Alejano et al., 2008](#)).

Figure 2.4: Vertical displacement contours from 3DEC model results ([Gao, 2013](#)).

2.3.2 Discontinuous Deformation Analysis Method (DDA)

Discontinuous deformation analysis (DDA) is a form of discrete element method, originated by [Shi \(1988\)](#). The original formulation of DDA was based on the first order polynomial displacement function. Therefore, the stresses and strains inside the blocks

were constant. However, for the cases that the displacements inside the blocks are high and cannot be ignored, hybrid methods are suggested by [Chang \(1994\)](#) and [Jing \(1998\)](#) which allow to add finite element meshes to the blocks so that the stress variation within the block can be allowed. Three-dimensional model of this method is still under development.

[Yeung \(1993\)](#) studied the effects of loading, joint friction angle and rock bolting on the stability of a cross-jointed mine roof using the discontinuous deformation analysis (DDA). His numerical modeling results showed that under low vertical loads, the roof becomes stable by formation of a flat arch inside it and under higher vertical loading the roof fails by the formation of hinges in the supporting arch developed inside the roof (see [Figure 2.5](#)). He also studied the effect of friction angle of the block surfaces on the stability of the roof. He concluded that for lower values of the friction angle the central part of the roof fails as a single unit into the opening (see [Figure 2.6](#)) while for higher friction angle values the roof fails by buckling (see [Figure 2.7](#)).

Figure 2.5: DDA results for the roof under high vertical load ([Yeung, 1993](#)).

Figure 2.6: DDA results for the case with small friction angle (Yeung, 1993).

Figure 2.7: DDA results for the case with high friction angle (Yeung, 1993).

2.4 Conclusion and scope of current research

This chapter has reviewed different numerical techniques used for modeling the roof behavior in underground coal mines. This review has shown that most of the past modeling was done using the continuum based methods and the ones that are done based on the distinct element method were two-dimensional except Fuqiang's (Gao, 2013) research, who used the three-dimensional distinct element method. However, he did not consider the discontinuities explicitly in the roof and also his simulation was limited to a single roadway.

FLAC3d and ABAQUS are two software programs which have been used widely in the past for three-dimensional modeling of roof failure. As mentioned before these two codes are based on the continuum mechanics concepts. So, they cannot capture the real

behavior of the discontinuities. ABAQUS is a software suite for finite element analysis and it allows simulation of interfaces using interface and contact elements. FLAC3D is based on explicit finite difference method that provides both the explicit interface technique and the ubiquitous joint model. In both of these programs, implementing the mentioned techniques for incorporating the discontinuities in the model has major shortcomings. The interface elements first should be defined geometrically and then meshed separately. This process is rather complicated and only can be applied to limited number of interfaces. It is clear that inappropriate selection of interface elements will lead to erroneous results. The ubiquitous joint model in FLAC3D is limited to only one joint set and also it cannot capture the anisotropy in elastic properties due to the existence of discontinuities. According to author's point of view, the ubiquitous joint technique in FLAC3D does not have any physical basis and would lead to incorrect interpretation of the numerical modeling results.

It is well known that discontinuities not only are weakening the rock structure but also they are changing the mechanical behavior of rock masses. In essence, the slip or separation of existing discontinuities in roof strata can amplify the negative effects of stress concentrations around the excavations.

Analysis of the problems which can be assumed as plane strain or plain stress, is more appropriate to be conducted using two-dimensional modeling. Where a simplification to plane strain or planes stress condition is not justifiable, such as in the case of room-and-pillar geometries since the effect of end-walls cannot be ignored, three-dimensional analysis is required. In reality the discontinuities are three-dimensional features and are represented by orientations, locations, shapes and dimensions in space. Therefore, discontinuities cannot be represented properly in two-dimensional modeling.

In conclusion, this review showed that three-dimensional distinct element methods can provide a more accurate and realistic description of the physical behavior of jointed materials. In this study, the 3DEC ([Itasca, 2013](#)) software was employed for numerical analyses. The main objective of this research is to investigate the influence of discontinuities on behavior of roof in room-and-pillar coal mines. Other parametric studies also were conducted to investigate the effects of different parameters such as, variation of rock mass properties, geometry of excavation and direction and magnitude of horizontal stresses on behavior of roof in underground coal mines.

Chapter 3

Estimation of Geomechanical Parameters for Different Intact Rock and Discontinuities from Laboratory Tests

3.1 Introduction

In the field-scale, the rock masses can be assumed as composite materials that are composed of discontinuities and blocks of intact rock between the discontinuities. The properties of the intact rock and discontinuities govern the behavior of rock masses in a very complicated manner. The rock mechanical properties of intact rock and discontinuities, as the components of rock mass, can be estimated in the laboratory. However, the direct measurement of rock mass properties using in-situ tests is expensive and time consuming. In addition the estimated values of rock mass properties from in-situ tests are not reliable due to operational and interpretation problems. Laboratory testing is the most common technique to obtain the required properties of intact rock and discontinuities for estimating the rock mass properties. The International Society for Rock Mechanics (ISRM) and the American Society for Testing and Materials (ASTM) have published suggested standards for conducting a variety of laboratory tests. These standards provide meaning to rock

mechanical properties which are obtained from laboratory tests.

In this study, the laboratory tests are performed on the core samples obtained from an eastern U.S. coal mine, at the University of Arizona Rock Mechanics Laboratory. Brazilian tensile test, unconfined compressive strength (UCS) test, triaxial compression test, direct shear test of discontinuities, and a specially designed test for measuring the joint normal stiffness are conducted on the prepared samples to obtain the intact rock and discontinuity properties of different rock types from the case study mine. Up scaling of the intact rock properties to rock mass properties is not covered in this section and will be discussed later in this dissertation. The summary of all laboratory test results and a brief description of each test are covered in the following sections. For each test the results are listed based on the naming convention of the geological units at the case study mine.

3.2 Estimation of Intact Rock Physical and Mechanical Properties

3.2.1 Tensile Strength

Tensile strength of the rock is defined as the maximum tensile stress that the rock material can withstand. Tensile strength of the rock can be determined by direct pulling test, bending, and indirectly from Brazilian test. Brazilian test is the most common method for tensile strength measurement. Figure 3.1 shows the schematic of apparatus for Brazilian test. The device consists of upper and lower support rings, which apply a compressive load diametrically on a disk shaped sample. Normally the thickness of the sample is half of the sample diameter. The load on the sample generates a tensile stress along the central loading plane of the disk in a direction perpendicular to the direction of applied load. The

result of this loading is a tensile failure in or close to the loading direction. For a disk shaped sample, the tensile strength is expressed by the equation given below:

$$T_o = \frac{2F_c}{\pi Dt} \quad (3.1)$$

In Equation 3.1, D and t are the sample diameter and thickness in mm; F_c is the yield load in Newtons; T_o is the tensile strength in MPa.

Figure 3.1: Schematic of apparatus for Brazilian test

Twelve disk shaped samples, three for each rock type, with each of 2 inch diameter and 1 inch thickness were prepared and tested to estimate the tensile strength of each rock type. Through statistical analysis the mean, maximum, minimum, standard deviation and the coefficient of variation of tensile strength of each rock type were calculated. The summary of the results are listed in Table 3.1.

Table 3.1: Brazilian tensile strength test results.

	Shale	Sandstone	Sandy Shale	Laminated Silty Shale
Maximum value (MPa)	11.33	12.84	13.72	11.71
Minimum value (MPa)	9.41	11.03	9.12	10.81
Mean value (MPa)	10.51	12.13	11.42	11.31
Standard deviation (MPa)	0.99	0.96	3.25	0.46
Coefficient of variation	0.094	0.080	0.285	0.041

3.2.2 Unconfined Compressive Strength (UCS)

The uniaxial compressive test is one of the key tests performed on rocks for determination of Unconfined Compressive Strength (UCS). In this test the circular cylinders of rock samples are compressed under axial load as shown in Figure 3.2. This test is performed based on methods suggested by ASTM D 5102-09. Young's modulus and Poisson's ratio were calculated by using strain gauges along with lateral and axial strain measurements. ISRM recommends the height to diameter ratio of 2 for samples. The following equation is used for the calculation of the compressive strength of the sample:

$$UCS = \frac{P_{max}}{A} \quad (3.2)$$

In Equation 3.2, Pmax is the ultimate load on the sample, and A is the cross sectional area of the sample. When the sample size differs from the ISRM suggestion, the UCS calculated from Equation 3.2 should be corrected as follows:

$$UCS = 0.889\sigma_c \left[0.778 + 0.222 \frac{H}{D} \right] \quad (3.3)$$

Figure 3.3, , shows typical axial and lateral stress-strain responses measured with strain gauges on a cylindrical rock sample during a uniaxial compression test. In this study, the tangent modulus at 50% of the peak stress value was used to estimate the elastic parameters of the samples.

Figure 3.2: Uniaxial compression test apparatus. Strain gauges are attached to the sample for measuring the axial and lateral strains.

Figure 3.3: Basic elastic properties of rock determined from uniaxial compression test with strain gauges.

Two to three samples were tested to estimate the unconfined compressive strength and elastic properties (Modulus of elasticity and Poisson's ratio) of each lithology. Strain gauges have been attached on two samples of each lithology to determine the Young's modulus and Poisson's ratio values.

The length of the prepared samples was approximately twice the diameter. The diameter and length were roughly 50.8 mm (2.0 in.) and 101.6 mm (4 in.), respectively. Two ends of the samples were grinded to make the bearing surfaces smooth and parallel as much as possible. This helped to create a uniformly distributed load over the bearing surfaces of the samples. The summary of the test results along with some statistics are shown in Tables [3.2](#) – [3.5](#).

Table 3.2: Summary of the estimated densities for each rock type.

Density	Shale	SandStone	Sandy Shale	Laminated Silty Shale
Maximum value (kg/m ³)	2635.30	2560.30	2688.30	2677.40
Minimum value (kg/m ³)	2552.20	2486.20	2614.10	2653.50
Mean value (kg/m ³)	2605.70	2526.73	2649.50	2665.45
Standard deviation (kg/m ³)	46.42	37.54	37.22	16.90
Coefficient of variation	0.018	0.015	0.0140	0.0063
Number of tests	3	3	3	2

Table 3.3: Summary of the measured UCS values for each rock type.

UCS	Shale	SandStone	Sandy Shale	Laminated Silty Shale
Maximum value (MPa)	87.39	112.04	59.31	97.21
Minimum value (MPa)	66.48	130.20	56.53	90.04
Mean value (MPa)	78.41	154.54	57.52	93.62
Standard deviation (MPa)	10.76	23.69	2.13	5.07
Coefficient of variation	0.137	0.153	0.037	0.054
Number of tests	3	3	3	2

Table 3.4: Summary of the estimated Young's modulus values for each rock type.

Young's Modulus	Shale	Sandstone	Sandy Shale	Laminated Silty Shale
Maximum value (GPa)	28.20	35.20	27.40	16.70
Minimum value (GPa)	25.30	31.60	25.60	14.80
Mean value (GPa)	26.75	33.40	26.50	15.75
Standard deviation (GPa)	2.05	2.55	1.27	1.34
Coefficient of variation	0.077	0.076	0.0480	0.085
Number of tests	2	2	2	2

Table 3.5: Summary of the estimated Poisson's ratio values for each rock type.

Poisson's ratio	Shale	Sandstone	Sandy Shale	Laminated Silty Shale
Maximum value	0.204	0.219	0.309	0.228
Minimum value	0.165	0.201	0.155	0.165
Mean value	0.185	0.210	0.232	0.197
Standard deviation	0.028	0.013	0.109	0.045
Coefficient of variation	0.15	0.061	0.47	0.23
Number of tests	2	2	2	2

3.2.3 Triaxial Compression Test

The triaxial compression test is the most frequent and useful test used in rock mechanics to estimate intact rock strength parameters. In this test, the axial load along the axis of the cylindrical specimen is the major principal stress and the all around confining stress applied to the curved surface of the sample by confining-fluid, is the minor principle stress (Peng and Zhang, 2007) . A schematic of triaxial compression test is shown in 3.4. At a predetermined confining pressure (σ_3) the sample is loaded axially (σ_1) until it fails. As it can be seen from Figure 3.5, the rock strength increases as the confining pressure increases.

In total twelve triaxial tests, three for each rock type were conducted at different confining stresses. The results of triaxial and uniaxial compression (mean value for each rock type) tests were combined to determine the angle of internal friction, ϕ , and the cohesion, c , of each rock type. The linear Mohr-Coulomb failure criterion was assumed to perform regression analysis for each rock type. The results of triaxial tests are summarized in Table 3.6. Figures 3.6 through 3.9 show the test results in $\sigma_1 - \sigma_3$ space with the linear fit (Mohr-Coulomb) for each rock type. The Mohr circles and the Mohr-Coulomb failure envelope for each rock type are plotted as shown in Figures 3.10 through 3.13.

Figure 3.4: Schematic of triaxial compression test apparatus ([Aadnoy and Looyeh, 2011](#))

Figure 3.5: Stress-strain curves from triaxial tests for medium-grained sandstone under different confining pressures ([Peng and Zhang, 2007](#)).

Table 3.6: Summary of triaxial test results

Rock type	Cohesion (MPa)	Friction angle (deg.)
Shale	25.73	22.45
Sandstone	32.37	45.35
Sandy Shale	18.97	21.98
Laminated Silty Shale	25.4	22.59

Figure 3.6: Uniaxial and triaxial test data for Shale and Mohr-Coulomb fit.

Figure 3.7: Uniaxial and triaxial test data for Sandstone and Mohr-Coulomb fit.

Figure 3.8: Uniaxial and triaxial test data for Sandy Shale and Mohr-Coulomb fit.

Figure 3.9: Uniaxial and triaxial test data for Laminated Silty Shale and Mohr-Coulomb fit.

Figure 3.10: The Mohr circles and the Mohr-Coulomb failure envelope for Shale.

Figure 3.11: The Mohr circles and the Mohr-Coulomb failure envelope for Sandstone.

Figure 3.12: The Mohr circles and the Mohr-Coulomb failure envelope for Sandy Shale.

Figure 3.13: The Mohr circles and the Mohr-Coulomb failure envelope for Laminated Silty Shale.

3.3 Estimation of Discontinuity Mechanical Properties

In this study the friction angle of the discontinuities were obtained from the linear best-fit plot of the shear stress versus the applied normal stress. The joint shear and normal stiffnesses were determined from the slopes of shear stress versus joint shear displacement and normal stress versus joint normal displacement curves.

3.3.1 Direct Shear Test

The direct shear test is the most commonly used test to determine the mechanical properties of rock discontinuities such as joints and bedding planes. Diagram of a typical shear testing machine which can be used to determine the mechanical properties of discontinuities is illustrated in Figure 3.14. Shear stress-shear displacement plots are more likely to steepen before slip and then flatten as slip begins along the discontinuity. For the rough joints the slope of the shear stress-shear displacement curve may turn negative after reaching an initial peak and then flatten. Such behavior is characteristic of the rough joints sheared under moderately low normal stress. As it can be seen from the test results, the smooth joints usually do not show peak residual behavior (Pariseau, 2011). An idealized plot of the peak and residual behavior is illustrated in Figure 3.15.

In total, fifteen direct shear tests were conducted to estimate the discontinuity mechanical properties. Whenever it was not possible to perform the test on a natural joint, the core samples were sawn in half to create artificial planar joints. The results of these tests have been used to determine the friction angle and shear stiffness values of discontinuities. The direct shear tests have been carried out at four different normal loads for each sample. The plot of shear stress versus shear displacement is shown in Figure 3.16 for sample # 2 of

Laminated Silty Shale as an example and the rest of the plots are illustrated in Figures A.1 - A.15 in Appendix A. The shear strength values were plotted against the normal stress values, and then a straight line was fitted through these points to determine the friction angle of the discontinuity. The test data and linear fit of the data are illustrated as an example in Figure 3.17 for sample # 2 of Laminated Silty Shale and the rest of the plots are shown in Figures A.16 - A.34 in Appendix A. The summary of the test results for each sample is presented in Table 3.7. For the sawn samples the estimated friction angle is the basic friction angle and for the samples with natural joints both the residual and peak friction angle were estimated. For Laminated Silty Shale samples, all of the tests were conducted on natural joints. In some of the samples the discontinuities were very smooth and therefore the estimated peak friction angles are approximately equal to the residual friction angles. Non-zero cohesion and pseudo-cohesion values are artifacts of the curve-fitting and as it can be seen all cohesion values are close to zero. This indicates that as it was expected, the shear strength only has the frictional component. The statistical summaries of the test results for rock types are shown in Tables 3.8 through 3.11.

Figure 3.14: Diagrammatic section through a shear testing machine (Hoek, 2000).

Figure 3.15: Peak and residual shear stress–shear displacement behavior for a rough joint.

Figure 3.16: The plot of shear stress versus shear displacement for sample # 2 of Laminated Silty Shale under four different normal stresses.

Figure 3.17: Direct shear test data and linear fit for sample # 2 of Laminated Silty Shale.

Table 3.7: The results of the direct shear tests for all rock types.

Rock Type	Joint Type	Cohesion (KPa)	Friction angle (°)	
			Basic/Residual	Peak
Shale 1	Natural joint	21.18	20.21	21.1
Shale 2	Sawn	40.1	15.51	-
Shale 3	Sawn	26.6	18.33	-
Sandstone 1	Sawn	22.45	18.79	-
Sandstone 2	Sawn	10.16	21.26	-
Sandstone 3	Sawn	1.8	20.21	-
Sandstone 4	Natural joint	57.9	20.3	25.3
Sandy Shale 1	Sawn	39.9	16.43	-
Sandy Shale 2	Sawn	6.47	18.58	-
Sandy Shale 3	Sawn	13.3	15.48	-
Sandy Shale 4	Sawn	17.12	19.09	-
Laminated Silty Shale 1	Natural joint	15.16	18.94	18.94
Laminated Silty Shale 2	Natural joint	16.5	16.53	16.53
Laminated Silty Shale 3	Natural joint	32.14	16.63	19.97
Laminated Silty Shale 4	Natural joint	35.95	18.02	19.39

Table 3.8: The statistical summary of direct shear test results for Shale.

Shale	Cohesion (KPa)	Friction angle (°)	
		Basic/Residual	Peak
Maximum value	40.10	20.21	-
Minimum value	21.18	15.51	-
Mean value	29.29	18.02	-
Standard deviation	9.74	2.37	-
Coefficient of variation	0.33	0.13	-

Table 3.9: The statistical summary of direct shear test results for Sandstone.

Sandstone	Cohesion (KPa)	Friction angle (°)	
		Basic/Residual	Peak
Maximum value	57.90	21.26	21.10
Minimum value	1.80	18.79	-
Mean value	23.08	20.14	-
Standard deviation	24.72	1.02	-
Coefficient of variation	1.07	0.05	-

Table 3.10: The statistical summary of direct shear test results for Sandy Shale.

Sandy Shale	Cohesion (KPa)	Friction angle (°)	
		Basic/Residual	Peak
Maximum value	39.90	19.09	-
Minimum value	6.47	15.48	-
Mean value	19.20	17.40	-
Standard deviation	14.49	1.72	-
Coefficient of variation	0.75	0.10	-

Table 3.11: The statistical summary of direct shear test results for Laminated Silty Shale.

Laminated Silty Shale	Cohesion (KPa)	Friction angle (°)	
		Basic/Residual	Peak
Maximum value	35.95	18.94	19.97
Minimum value	15.16	16.53	16.53
Mean value	24.94	17.53	18.71
Standard deviation	10.64	1.16	1.51
Coefficient of variation	0.43	0.07	0.08

3.3.2 Estimation of Joint Stiffnesses

The normal and shear stiffnesses relate the joint stress to relative displacement across and along the joint. Thus,

$$\sigma = k_n v, \text{ and } \tau = k_s u \quad (3.4)$$

where σ , τ , v , u , k_n , and k_s are joint normal stress, joint shear stress, joint normal displacement, joint shear displacement, joint normal stiffness, and joint shear stiffness, respectively.

The units of joint stiffnesses are defined as stress over displacement units, i.e., psi/in. or GPa/m. The normal and shear displacements of the joints are highly nonlinear. Therefore, any stress displacement relation for the joints should be considered in differential form as shown in Equations 3.5 .

$$d\sigma = k_n dv, \text{ and } d\tau = k_s du \quad (3.5)$$

This type of formulation defines the joint stiffnesses more realistically as local slopes of stress displacement plots ([Pariseau, 2011](#)).

Joint Normal Stiffness from Uniaxial Joint Compression Test

There is neither a published standard procedure in American Society for Testing and Materials (ASTM) nor in the International Society of Rock Mechanics (IRSM) for rock joint testing of the type that was performed in this study. The sample preparation procedure

for this test was like the preparation of sample for unconfined compressive test. Then the sample was sawn in half to create a single horizontal joint perpendicular to the axis of the cylindrical sample. To estimate the joint normal stiffness of bedding planes, joints among the same materials, and interfaces of different rock materials, joints among two different materials, the tests were conducted on all possible combinations of the available rock types.

The total deformation obtained from the test includes deformation of the intact rock parts as well as the joint deformation. Figure 3.18 shows the total deformation-axial stress plot obtained for sample #2 of Laminated Silty Shale, as an example. The same figure shows how the deformation-axial stress plot was obtained for the intact rock portion by drawing a line parallel to the straight line portion of the total deformation-axial stress curve. Figure 3.19 depicts the joint deformation-axial stress curve obtained for Laminated Silty Shale joint by subtracting intact deformation from the total deformation. The joint deformation - normal stress plot from a joint testing is usually non-linear and generally it shows a decreasing slope with increasing normal stress, but ultimately reaches a constant value (see Figure 3.19). As it can be seen from Figure 3.20, the exponential curve fitting was performed to mathematically relate the joint deformation to axial stress. In this plot, the joint normal stiffness for any given axial stress is equal to the slope of the line tangent to the curve at that given point. Mathematically, the first derivative of the derived equation from curve fitting will result as the joint normal stiffness for any given normal stress. As an example, for Laminated Silty Shale, from exponential curve fitting the relation between normal stress and joint normal deformation can be expressed by Equation 3.6. The first derivative of this equation will give the joint normal stiffness at any given normal stress. Therefore, for Laminated Silty Shale the derived equation of joint normal stiffness can be

expressed by Equation 3.7.

$$\sigma_n = 0.2624e^{16.49v} \quad (3.6)$$

$$k_n = \sigma'_n = 16.491(0.2624e^{16.49v}) = 16.491\sigma_n \quad (3.7)$$

Figure 3.21 shows the plot corresponding to Equation 3.7. This technique for estimating the joint normal stiffness was suggested by Professor Kulatilake and was followed to estimate the joint normal stiffnesses of all samples.

The summary of the test results are listed in Table 3.12. The plots of the total deformation versus axial stress, joint normal deformation versus axial stress, and axial stress versus the joint normal deformation with exponential curve fit, the plot of joint normal stiffness for different values of normal stresses are depicted in Figures A.35 – A.44, A.45 – A.54, A.55 – A.64, and A.65 – A.74 in Appendix A, respectively.

Figure 3.18: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Laminated Silty Shale.

Figure 3.19: Plot of joint normal deformation versus axial stress for Laminated Silty Shale.

Figure 3.20: The plot of axial stress versus the joint normal deformation and exponential curve fit for Laminated Silty Shale joint.

Figure 3.21: The plot of joint normal stiffness values for different values of normal stresses for Laminated Silty Shale joints.

Table 3.12: The summary of uniaxial joint compression test results.

Rock type	Joint type	Derived equation for K_n (GPa/m)	K_n (GPa/m)	K_n (GPa/m)	K_n (GPa/m)
			at $\sigma_n=1.72$ MPa	at $\sigma_n=0.83$ MPa	at $\sigma_n=0.42$ MPa
Laminated Silty Shale	Bedding plane	$16.491\sigma_n$	28.36	13.68	6.92
Sandstone	Bedding plane	$13.053\sigma_n$	22.45	10.83	5.48
Sandy Shale	Bedding plane	$15.945\sigma_n$	27.42	13.23	6.7
Shale	Bedding plane	$22.872\sigma_n$	39.3	18.98	9.6
LSS* – Sandstone	Interface	$17.337\sigma_n$	29.82	14.39	7.3
LSS – Sandy Shale	Interface	$15.74\sigma_n$	27.1	13.07	6.6
LSS – Shale	Interface	$18.881\sigma_n$	32.47	15.67	7.93
Sandstone – SS**	Interface	$30.386\sigma_n$	52.26	25.22	12.76
Shale – Sandstone	Interface	$26.461\sigma_n$	45.5	21.96	11.12
Shale – SS	Interface	$23.023\sigma_n$	39.6	19.1	9.66

LSS = Laminated Silty Shale

SS = Sandy Shale

Estimation of Joint Shear Stiffness from Direct Shear Test

Shear stiffness represents the resistance of the discontinuity to shear displacements under an applied shear stress before reaching the peak shear strength (ASTM D 5607 - 95). As it can be seen from the direct shear test results, the shear stress versus shear displacement curve is approximately linear up to the yield point and the slope is a function of the normal stress. In this study, the joint shear stiffness was estimated from the slope of the shear stress versus shear displacement curve prior to reaching the peak shear strength. The relation between the shear stiffness and normal stress can be expressed by an equation similar to equation 3.7. The summary of the test results is listed in Table 3.13.

Table 3.13: The summary of estimated joint shear stiffnesses.

Rock type	Joint type	Derived equation for Ks (GPa/m)	Ks (GPa/m) at $\sigma_n=1.72$ MPa	Ks (GPa/m) at $\sigma_n=0.83$ MPa	Ks (GPa/m) at $\sigma_n=0.42$ MPa
Laminated Shale 1	NJ	$0.652\sigma_n$	1.07	0.6	0.21
Laminated Shale 2	NJ	$0.660\sigma_n$	1.11	0.65	0.23
Laminated Shale 3	NJ	$0.563\sigma_n$	0.95	0.47	0.38
Laminated Shale 4	NJ	$0.826\sigma_n$	1.44	0.81	0.47
Average (NJ)	-	$0.6752\sigma_n$	1.14	0.63	0.30
Sandstone 1	SW	$1.58\sigma_n$	2.91	1.37	1.10
Sandstone 2	SW	$1.174\sigma_n$	1.97	0.98	0.46
Sandstone 3	SW	$1.355\sigma_n$	2.23	1.16	0.68
Sandstone 4	NJ	$0.782\sigma_n$	1.52	0.71	0.46
Average (SW)	-	$1.223\sigma_n$	2.16	1.05	0.67
Sandy Shale 1	SW	$0.996\sigma_n$	1.79	1.10	0.52
Sandy Shale 2	SW	$0.871\sigma_n$	1.84	1.07	0.85
Sandy Shale 3	SW	$0.659\sigma_n$	1.15	0.52	0.40
Sandy Shale 4	SW	$0.864\sigma_n$	1.68	1.26	0.64
Average (SW)	-	$0.798\sigma_n$	1.56	0.98	0.63
Shale 1	SW	$0.862\sigma_n$	1.68	1.26	0.64
Shale 2	NJ	$0.953\sigma_n$	1.62	0.83	0.53
Shale 3	SW	$0.975\sigma_n$	1.65	1.25	0.38
Average (SW)	-	$0.918\sigma_n$	1.65	1.25	0.51

Chapter 4

Numerical Modeling of Roof Stability in Underground Room-and-pillar Coal Mines

4.1 Introduction

The 3DEC numerical code was used to setup three-dimensional models for the stability analysis of underground excavations in room-and-pillar mines. The numerical analysis presented in this chapter focus on the following aspects:

- Parametric studies on the influence of the magnitude and orientation of the horizontal stresses on stability of the immediate roof.
- Parametric studies on the effects of rock mass properties of the immediate roof on stability around excavations.
- Parametric studies on the influence of variation of bedding plane properties on behavior of the immediate roof.
- Effect of variation in room and pillar size on roof stability.

Laminated or thinly bedded shale is one of the most frequently seen overlaying strata above the coal deposits. Of particular importance in underground coal mining is the lithology of immediate roof with respect to the underground opening. The bedding planes and interfaces have very low or close to zero tensile strength in the direction perpendicular to the bedding planes and their shear strength is much lower than of the rock layers. As a result, the bedding planes are much weaker than the rock layers. Therefore, due to the stress concentrations caused by excavations, slippage and separation can easily happen along the bedding planes before initiation of failure within the rock layers. The intensity of slippage and bed separation in underground openings depends on the rock types as well as the bonding strength of the beds and magnitudes of the loads.

The slippage (shear-displacement) and separation (normal displacement) of bedding planes can amplify the adverse effects of stress concentrations around the excavation. Therefore, the bedding planes play an important role on the behavior of roof in underground excavations and the thinly bedded rock should be considered as a problematical roof in underground excavations.

The failure of underground excavations is a result of failure in both the rock mass part and major discontinuities. Bedding planes and other large scale discontinuities are considered as major discontinuities. The rock mass part, considered as an equivalent continuum, consists of intact rock and minor discontinuities. The small scale discontinuities approximately perpendicular to the bedding planes are considered as minor discontinuities. In contrast with continuum numerical methods, the discontinuum techniques can capture the aforementioned both types of failures and their interaction. In most of the past research on numerical modeling of underground excavations, the continuum methods, which cannot effectively simulate the significant influence of discontinuities on the behavior of jointed

rock mass, have been used.

In this study, 3DEC, a three-dimensional distinct element code was used to investigate the stability of excavations in underground room-and-pillar coal mines. The main interest was the stability of immediate roof with Laminated Silty Shale rock type. In order to accurately investigate the failure in the rock mass part of the roof strata, small sized six-sided polyhedral type of zones were allocated to the immediate roof layer. Also, to replicates the post failure behavior of the rock layers in the immediate roof area, the strain softening material constitutive law was implemented to this region.

In this chapter the three-dimensional distinct element code, 3DEC, is introduced and some of its significant features are discussed briefly. Following this the results of numerical modeling are discussed to investigate the influence of different parameters on stability of the underground excavations in room-and-pillar coal mines.

4.2 Introduction of Three-dimensional Distinct Element Code (3DEC)

3DEC is a three-dimensional numerical program based on distinct element methods for discontinuum analysis. This software is based on a Lagrangian calculation scheme which is well-suited for modelling the large deformations and movements of blocky systems. The response of jointed rock masses subjected to either static or dynamic loading can be simulated using this software. The discontinuous medium is represented as assemblages of discrete blocks which might be considered as rigid or deformable and discontinuities which are treated as interfaces (boundary conditions) between distinct blocks. In contrast with continuum methods, large displacements along discontinuities and rotations of blocks are allowed. Deformable blocks are divided into finite difference elements, and each element

reacts according to an assigned linear or non-linear stress-strain law. Linear or non-linear force-displacement relations are used to govern the relative motion of discontinuities for movements in both the normal and shear directions. 3DEC is primarily intended for rock engineering applications and particularly is well-suited to study the failures directly related to the existence of discontinuous features. 3DEC has a built in programming language called Fish, which was used in this research extensively for developing the model such as building the geometry, assigning the in-situ stresses, etc. (Itasca, 2013).

4.2.1 The Problem Solution Scheme in 3DEC

3DEC is based on a dynamic algorithm (time-domain) that solves the equations of motion using the explicit finite difference method. At each time-step, the law of motion and constitutive equations are applied. The sub-contact force-displacement relations are subscribed for both rigid and deformable blocks. The new block positions and contact displacement increments (or velocities) are calculated from integration of the law of motion. The new sub-contact forces are calculated from the new block positions using force-displacement law and then applied to the blocks in the next time-step. The cycle of mechanical calculations is illustrated in Figure 4.1 (Itasca, 2013).

Figure 4.1: The calculation cycle in 3DEC.

In comparison to other available numerical modeling programs, 3DEC is more complex with capability of modeling the three-dimensional problems in great detail. This software is best-suited for analysis of three-dimensional discontinuous systems where the response of the system is predominately governed by behavior of randomly oriented joint sets. 3DEC can be used to perform continuum analysis if so desired, though the code is not efficient as conventional continuum based codes such as FLAC3D.

4.2.2 Numerical Modeling Procedure in 3DEC

In order to setup and run a model in 3DEC, it is necessary to specify the following three main components of the problem:

- (1) A distinct-element model that matches the problem geometry;
- (2) Constitutive behavior and material properties; and

(3) Boundary and initial conditions.

The model block is used to define the geometry of the problem. The constitutive behavior and assigned material properties control the response of the model to disturbance. The boundary and initial conditions define the in-situ state (i.e., the condition before excavation). After these conditions are defined in 3DEC, the model is brought into equilibrium state. Then an alteration is made (e.g. excavation of material or changing the boundary conditions). After bringing the model again to equilibrium state, the resulting response of the model is calculated. The general solution procedure for a static analysis with 3DEC is illustrated in Figure 4.2. Therefore it is convenient to follow this procedure because it appropriately represents the sequence of processes that occur in the physical model.

Figure 4.2: The general solution procedure for static analysis in 3DEC.

4.3 Parametric Study of Roof Stability by Three-dimensional Numerical Modeling

The reference model was built as much as possible with symmetrical geometry to allow for easy assessment of the influence of each individual variable in parametric studies.

A set of material and discontinuity properties was adopted for numerical modeling based on the results of laboratory tests performed in this study and literature review. To account for the influence of the presence of minor discontinuities in the rock masses, the strength and deformability properties of the rock masses used in the numerical models were estimated by reducing the mean values of test results obtained on intact rock samples. In order to assess the effect of in-situ horizontal stress magnitude on stability of the roof in underground excavations, several parametric studies were performed for different horizontal-to-vertical stress ratios. In the eastern U.S. coalfields, the maximum horizontal stress is generally two or three times the vertical stress and 76% of the maximum horizontal stress direction fall within N25°E and N75°E ([Agapito and Gilbride, 2002](#)). For the reference model this ratio was assigned as 2 and 1 for the maximum horizontal-to-vertical stress and the minimum horizontal-to-vertical stress, respectively.

To investigate the influence of the maximum horizontal stress orientation on roof stability, different numerical models were solved for different orientations of the maximum horizontal stress. In the reference model the direction of the maximum horizontal stress was aligned in the direction of the crosscuts (perpendicular to the direction of the entries). The effect of variability in major discontinuity and rock mass properties on roof stability also has been

investigated.

4.3.1 Geometry and Lithology of the Model

A system of coordinate axes was defined with the origin at the center of the model and mid-height of the central pillar; the x-axis, y-axis and z-axis were set along the directions of the cross cuts, the entries and upwards, respectively. The reference model has 25 pillars (5 in x-direction and 5 in y-direction), six entries and six crosscuts. Both entries and crosscuts were 6.1 m wide whereas pillars were square with side dimension 12.7m. Figure 4.3 shows the plan view of the room-and-pillar mining layout and the locations of the four vertical cross-sections which will be referred to as the cross-sections A1-A1, A2-A2, B1-B1 and B2-B2 for illustrating the numerical modeling results.

Figure 4.3: The plan view of the room-and-pillar mining layout.

The applied lithology for the modeling (see Figure 4.4) is based on a case study mine that has been experiencing severe roof failures at different intensity levels. Some roof falls were at intersections, and some were in entries or crosscuts averaging from 12.2 to 91.4 m long with fall height ranging from 1.5 to 3.7 m. The immediate roof in this mine consists of laminated silty shale, shale, or sandstone that changes from area to area, and the floor is shale or soft fireclay. Most of the roof falls had taken place in the thinly laminated silty shale roof. Figure 4.5 illustrates the locations of three horizontal cross-sections at the immediate roof area which will be referred to as the cross-sections C-C (roof line), D-D (0.65 m above the roof line) and E-E (1.3 m above the roof line) for illustrating the numerical modeling results.

Figure 4.4: Lithology used in the modeling.

Figure 4.5: The locations of horizontal cross-sections.

4.3.2 Meshing of the Model

To study the roof failure in soft rocks, the failure of rock mass must be accommodated in the model along with the discrete features such as faults and joints. The standard 3DEC zones to simulate the continuum materials are 4-node tetrahedral elements with linear displacement interpolation functions. When using the block plasticity in constitutive models, it is important to point out that an overestimation of the collapse load can result for the constant strain tetrahedral elements.

In 3DEC, accurate solutions to plasticity problems can only be achieved if all tetrahedral zones have a grid-point at the centroid of the block and they are diagonally opposed in the block, or if “mixed discretization” is used. “Mixed discretization” and “nodal mixed discretization” are two options available in 3DEC for improving the plasticity solutions. In the “mixed discretization”, the discretization for the isotropic and deviatoric parts of the stress and strain tensors are different. In essence, the deviatoric behavior is defined on an element basis (triangle in 2D or tetrahedral in 3D), while the volumetric behavior is averaged over an assembly of elements, referred to as zones (quadrilateral in 2D or hexahedral in 3D). Nodal mixed discretization (NMD) is a variation on the mixed discretization scheme, in which averaging of the volumetric behavior is carried out on a node, rather than a zone, basis ([Itasca, 2013](#)).

In this case study, the “mixed discretization” and “nodal mixed discretization” were used for meshing the immediate roof area. To increase the accuracy of calculations, three different mesh sizes were used as follows: (a) fine size mesh in the mining area (0.37 m for “mixed discretization” and 0.5 m for “nodal mixed discretization”); (b) an intermediate size mesh (1.5 m) to some extent around the mining area and (c) a coarser mesh (8 m) in

the rest of the model. The aspect ratios of the elements were kept near one. Figures 4.6 illustrates the different mesh types used in the numerical model.

Figure 4.6: Different mesh types used in the numerical model.

4.3.3 Material Properties

As discussed in chapter 3, the material properties of the case study mine were estimated using extensive lab test results. The estimated properties are for samples which are small size and representative of intact part of the rock masses. To account for the scale effect and influence of the presence of small size discontinuities in the rock masses, the estimated strength and deformability properties of the rock materials from lab tests were adjusted to rock mass scale by reducing the mean values of test results. There are several techniques for estimating the rock mass properties from intact rock properties. For instance, based on extensive experimental and numerical modeling, Kulatilake et al. (2001, 2004 and 2006), Wu and Kulatilake (2012) concluded that the ratio of mean rock mass strength / mean intact rock strength could range between 20 and 60%. The value of the internal friction angle found to be more or less the same for both the intact rock and rock mass scales. Therefore, it is suggested to assume the same frictional component and reduction of cohesion

component to represent the equivalent shear strength behavior of the rock mass which consist of intact rock combined with the minor discontinuities. The rock mass modulus was found to be between 50 and 60% of the intact rock modulus value. Also, the rock mass Poisson's ratio was found to be about 11-21% higher than the intact rock Poisson's ratio value. Note that these stated ratios depend on the rock mass discontinuity geometry system and values of discontinuity mechanical properties compared to that of intact rock properties. The reduction ratio of tensile strength was assumed to be the same as the reduction ratio of cohesion to calculate the new tensile strength to represent the tensile strength behavior of the equivalent continuum material that represents the tensile behavior of the intact rock combined with the minor discontinuities ([Kulatilake et al., 2004](#); [Wu and Kulatilake, 2012](#)).

In this case study, the results obtained from previous case studies by Kulatilake et al. (2002 and 2012) were used to estimate the rock mass properties from lab test results . The summary of the estimated rock mass properties which are used as inputs of the numerical models is given in Table [4.1](#). The coal samples were not available for performing the lab tests, therefore the assigned properties for coal are from the literature review and available test results from the case study mine.

Table 4.1: The assigned rock mass property values in 3DEC.

Parameter	Values/[ratios]			
	Massive Silty Shale	Laminated Silty Shale	Coal	Massive Shale
Tensile Strength (MPa)	2.40 [0.21]	2.40 [0.21]	1.0	3.30 [0.31]
Young's Modulus (GPa)	8.2 [0.52]	8.2 [0.52]	3.2	13.91 [0.52]
Poisson's ratio	0.21 [1.07]	0.21 [1.07]	0.3	0.23 [1.25]
Density (Kg/m³)	2665	2665	1300	2605
Cohesion (MPa)	5.1 [0.20]	5.1 [0.20]	3.4	5.54 [0.21]
Friction angle (°)	21 [0.83]	21 [0.83]	19	22.45 [1]

Note: the values in brackets are the ratios of equivalent properties to the corresponding intact rock properties.

4.3.4 Discontinuity Properties

The immediate roof at the mine is a weak laminated silty shale with thin laminations of 2.5 to 52 mm and fails easily along the weak laminations. For modeling, the thickness of laminated silty shale and the thickness of laminations are considered as about 4.5 m and 1.0 cm, respectively. With the assumed value for the thickness of laminations, the number of discontinuities in the immediate roof would be 450. Meshing the model with this many discontinuities that are having very low spacing is an almost impossible task in a large scale model. Therefore, the equivalent continuum concept was used to decrease the number of discontinuities from 450 to 3, by increasing the spacing of the joints from 1 cm to 1.5 m. Based on the equivalent continuum concept, it was necessary to reduce the joint normal stiffness and joint shear stiffness from $61\text{e}9 \text{ Pa/m}$ and $2.48\text{e}9 \text{ Pa/m}$ to $4.07\text{e}8 \text{ Pa/m}$ and $1.65\text{e}7 \text{ Pa/m}$, respectively to obtain the same normal and shear deformations for the laminated silty shale layer with the 3 discontinuities along with the selected new spacing. To be on the conservative side, the shear strength of discontinuities was modeled by a Mohr-Coulomb criterion with a residual friction angle and zero cohesion and tensile

strength. Table 4.2 provides the properties of the discontinuities that were used as input in the numerical modeling.

Table 4.2: The properties of the discontinuities used as input in the numerical modeling.

Parameter	Values for bedding planes	Values for interfaces	
	Laminated Silty Shale	Coal - Laminated Silty Shale	Coal - Massive Shale
Joint Friction Angle (°)	17.5	18.25	18.50
Joint Normal Stiffness (Pa/m)	4.07E+08	3.12E+10	4.28E+10
Joint Shear Stiffness (Pa/m)	1.65E+07	1.94E+09	2.29E+09

4.3.5 Boundary Conditions and In-situ Stresses

Since no horizontal stress measurement was done at the mine site, the magnitudes and directions of the principal stresses are not known. Therefore, in this research several parametric studies were performed for different horizontal-to-vertical stress ratios. For the reference model this ratio was assigned as 2 and 1 for the maximum horizontal-to-vertical stress and the minimum horizontal-to-vertical stress, respectively. To examine the influence of the maximum horizontal stress orientation on roof stability, several numerical models were solved for different directions of the maximum horizontal stress. In the reference model the direction of the maximum horizontal stress was aligned in the direction of the crosscuts. For each material layer the stress gradients and the in-situ stresses were calculated and applied in the numerical model. A vertical stress of 2.75 MPa was applied to the top of the model to simulate the overburden thickness of 110 m up to the ground level.

The horizontal displacements at the four lateral far field boundaries of the model, as well as the vertical displacement at the bottom of the model were set to zero by prescribing

the zero velocity boundary condition.

4.3.6 Material Constitutive Models

The failure around underground excavations is a result of failure in both the rock mass part and major discontinuities. In order to realistically predict the failure of rock mass part, the implemented material constitute law in the numerical model should be able simulate the failure mechanism as closely as possible. Out of different available material constitutive models, the Mohr-Coulomb elastic perfectly plastic model is the most commonly used constitutive law in rock engineering designs. This constitutive law represents the linear yield envelope where the area below the envelope represents the stable state in the material while the area above the envelope represents the failure state. In this criterion the plastic strain is not calculated directly. In the Mohr-Coulomb model the strength properties of the materials are assumed to remain constant after the onset of plastic failure. This assumption means that the material behaves in elastic-perfectly plastic manner by maintaining its strength after failure and is able to support a stress equal to the failure strength. In reality, this stress should get transferred to the surrounding zones. Figure 4.7 depicts the typical stress-strain curve for an elastic-perfectly plastic material. Therefore, the Mohr-Coulomb model is recommended to be applied for the problems where the post failure response of the material is less important. For modeling of the problems such as roof failure where the estimation of plastic strain is required, the strain-softening constitutive behavior is well-suited. The stress-strain curve for a material undergoing strain-softening behavior is depicted in Figure 4.8. The Mohr-Coulomb strain softening model is based on the Mohr-Coulomb model with possible degradation of cohesion, friction angle, tensile strength and dilation after the onset of plastic yield, while in the standard Mohr-Coulomb

model those parameters are assumed to remain constant. 3DEC uses the Mohr-Coulomb failure criterion with strain softening to detect the material failure and implements the cohesion, friction, dilation and tensile strength degradation as a function of plastic strain to implement the strain softening behavior as illustrated in Figure 4.9. These functions also might be approximated by linear segments as shown in Figure 4.10 (Itasca, 2013).

Figure 4.7: Typical stress-strain curve for an elastic-perfectly plastic material.

Figure 4.8: The stress-strain curve for a material undergoing strain-softening behavior.

Figure 4.9: Degradation of cohesion (a) and friction (b) as a function of plastic strain.

Figure 4.10: Degradation of cohesion (a) and friction (b) approximated by linear segments.

In this case study, the strain softening constitutive model was used for the laminated silty shale layer and for the rest of the materials the standard Mohr-Coulomb constitutive model was assigned. Ideally, the strain softening parameters should be estimated from laboratory tests. However, in this study, these parameters were not available and were estimated from a review of parameters used for modeling of similar problems by [Hajiabdolmajid et al. \(2002\)](#), [Badr et al. \(2003\)](#), [Zipf et al. \(2006\)](#), and [Ray \(2009\)](#). In this research, the angle of internal friction was kept constant and the cohesion degradation was applied as per Figure 4.11 for weak laminated rock.

Figure 4.11: Typical degradation of cohesion for different rock types with plastic strain (Ray, 2009).

4.3.7 Roof Failure Detection Criteria

Because the formation of new fractures in rock mass cannot be simulated in 3DEC, the location of the failures must be judged, for example by examining plastic indicators (failure flags in 3DEC), grid-point velocities, displacements, localization of shear strain and maximum unbalanced force. In the case of implementing strain softening behavior, the failure in rock mass damage can be presented as a percentage of cohesion degradation. This allows one to assess the level of the failure that has occurred as a function of accumulated plastic shear strain. A model is considered to be in equilibrium (stable) when the unbalanced force and velocity of grid-points have reached to near zero values. However, non-zero or fluctuating unbalanced force indicates that most likely the block failure or joint slip and plastic flow are occurring in the model. Non-zero unbalanced force only can be used to identify the stability of the model but not the location of failure (Itasca, 2013).

In 3DEC all the elements that are satisfying the failure criterion will be flagged by “plastic indicators” as unstable. However, depending on the amount of strain and confinement, even a yielded element can retain significant amount of post-failure strength. Therefore, the unstable elements flagged by plastic indicators not necessarily represent the collapsed elements.

In this research the distribution of post failure cohesion along with other indicators such as accumulated plastic shear strain, and yield state of elements were used to accurately assess the occurrence of failures in the rock mass.

4.3.8 Numerical Modeling Results of the Reference Case

The reference model has 25 pillars (5 in the x-direction and 5 in the y-direction), six entries and six crosscuts. Both entries and crosscuts are 6.1 m wide whereas pillars are square with side dimension 12.7 m. The applied lithology for the modeling is shown in Figure 4.4. Small sized six-sided polyhedral type of zones were allocated to the immediate roof layer. A vertical stress of 2.75 MPa was applied to the top of the model to simulate the stress due to the overburden thickness of 110 m up to the ground level. In this case study the direction of the maximum horizontal stress was aligned in the direction of the crosscuts. The ratios of the maximum horizontal-to-vertical stress and the minimum horizontal-to-vertical stress applied were 2 and 1, respectively. The assigned material and discontinuity properties are listed in Tables 4.1 and 4.2. The strain softening constitutive model was used for the laminated silty shale layer and for the rest of the materials the standard Mohr-Coulomb constitutive model was assigned. The numerical modeling results were obtained at the cross sections defined in Figures 4.3 and 4.5.

4.3.8.1 Failure State Around the Excavations

The model results obtained at cross sections A1-A1, B1-B1, A2-A2, B2-B2 (see Figure 4.3) and C-C, D-D, E-E (See Figure 4.5) are shown in Figures 4.12a-d and 4.13a-c, respectively. It can be seen from Figures 4.12a-b and 4.13a, at the roofline, the tensile failures take place at roof centers and shear failures take place at roof-centers of the roof line above the central and side entries (in the direction of the minimum horizontal stress) as well as the upper corners of entries and crosscuts.

Figure 4.13b depicts the failure state of the immediate roof at 0.65 m above the roofline. At this level, the tensile failures are more concentrated at the roof centers above the crosscuts in the direction of the maximum horizontal stress. The shear failures take place above the upper edges of the crosscuts as well as the mid-span and on the edges closer to the corners above the entries. The slippage of bedding planes at the immediate roof is capturing a significant amount of the shear stress concentrations in the direction of the minimum horizontal stress and as a result, the shear stress concentrations at the upper edges of the entries away from the corners are not at an optimum to cause shear failure throughout the edges of the entries.

The roof failure state at 1.3 m above the roofline is illustrated in Figure 4.13c. At this level the shear and tensile failures take place above the roof centers and above the crosscuts and entry edges, respectively.

(a) Failure state around the excavations on cross section A1-A1 (entries).

(b) Failure state around the excavations on cross section B1-B1 (crosscuts).

(c) Failure state around the excavations on cross section A2-A2 (entries).

(d) Failure state around the excavations on cross section B2-B2 (crosscuts).

Figure 4.12: The extent of unstable roof at entries and crosscuts.

(a) Failure state on cross section C-C at the roofline.

(b) Failure state at the immediate roof on cross section D-D, which is located at 0.65 m above the roof line.

(c) Failure state at the immediate roof on cross section E-E, which is located at 1.3m above the roofline.

Figure 4.13: The extent of unstable roof at the immediate roof.

4.3.8.2 Displacement Distribution at the Immediate Roof

The distributions of z-displacement are illustrated in Figures 4.14a-c. As shown in Figure 4.14, the z-displacements are higher above the entries, which are perpendicular to the direction of the maximum horizontal stress, compared to that above the crosscuts and reach a maximum above the intersections.

(a) Z-displacement distribution at the immediate roof on cross section A2-A2 (entries).

(b) Z-displacement distribution at the immediate roof on cross section B2-B2 (cross-cuts).

(c) Z-displacement distribution at the roofline on cross section C-C.

Figure 4.14: Z-displacement distributions at the immediate roof.

4.3.8.3 Distribution of Accumulated Plastic Shear Strain at the Immediate Roof

The magnitude of the accumulated plastic shear strain can be considered as a measure of damage intensity (inelastic deformations). The distributions of accumulated plastic shear strain at the immediate roof area are illustrated in Figures 4.15a-b and 4.16a-c. As can be seen from Figures 4.15b and 4.16a-c, shear failures take place above the edges of the crosscuts in the direction of the maximum horizontal stress and continuing vertically (cutter roof) up to the first bedding plane.

The Figures 4.15a and 4.16a-c show how the formation of cutter roof is different above the entries compared to that above the crosscuts. Above the entries, the cutter roof type failures do not take place at full length of the entry edges and are more concentrated at the vicinity of the corners (at the intersections between the entries and crosscuts) up to some extent. Also, above the entries in the vicinity of the bedding planes the slippage of discontinuity causes shear failures at the roof centers. The occurrence of shear failure along the bedding planes (in the direction of the minimum horizontal stress) and formation of cutter roof increases the possibility of roof collapse at the entries.

(a) Distribution of accumulated plastic shear strain at the immediate roof on cross section A2-A2 (entries).

(b) Distribution of accumulated plastic shear strain at the immediate roof on cross section B2-B2 (crosscuts).

Figure 4.15: Distribution of accumulated plastic shear strain at the immediate roof.

(a) Distribution of accumulated plastic shear strain at the roofline on cross section C-C.

(b) Distribution of accumulated plastic shear strain on cross section D-D, which is 0.65 m above the roof line.

(c) Distribution of accumulated plastic shear strain on cross section E-E, which is 1.3 m above the roof line.

Figure 4.16: Distribution of accumulated plastic shear strain at the immediate roof.

4.3.8.4 Distribution of Post Failure Cohesion at the Immediate Roof Area

The damage in the rock mass can be presented as degradation of cohesion. In this case study, the friction component of the shear strength was kept constant and the cohesion was reduced as a function of plastic strain. Thus, the rock mass damage associated with post-yield deformation is reflected as cohesion degradation until reaching a residual value and the elements with lower post failure cohesion will have less strength than the higher ones.

The distributions of post failure cohesion at the immediate roof area are shown in Figures 4.17a-b and 4.18a-c. In the plots, the darkest blue elements are representing the elements

with the lowest cohesion values and can be treated as failed elements. The locations of failed elements coincide with the locations of elements which have high accumulated plastic shear strain values and the same interpretation is applicable for detecting the shear failures.

(a) Distribution of post failure cohesion at the immediate roof on cross section A2-A2 (entries).

(b) Distribution of post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts).

Figure 4.17: Distribution of post failure cohesion at the immediate roof.

(a) Distribution of post failure cohesion at the roofline on cross section C-C.

(b) Distribution of post failure cohesion on cross section D-D which is 0.65 m above the roof line.

(c) Distribution of post failure cohesion on cross section E-E which is 1.3 m above the roof line.

Figure 4.18: Distribution of post failure cohesion at the immediate roof.

4.3.8.5 Normal and Shear Displacements of Bedding Planes at the Immediate Roof

Normal and shear displacements of the bedding planes at the immediate roof area are plotted in Figures 4.19a-c and 4.20a-c, respectively. As can be seen from Figure 4.19, the joint normal displacements are higher at the roof centers compared to other locations. The magnitudes of joint normal displacements at the roof centers above the entries are higher than those above the crosscuts. The maximum joint normal displacements take place above the intersections between the entries and crosscuts. The maximum normal displacement obtained above the intersection is around 1.58 cm. The maximum shear displacement obtained is around 1.95 cm.

(a) Joint normal displacement distribution on cross section A2-A2.

(b) Joint normal displacement distribution on cross section B2-B2.

(c) Joint normal displacement of closest bedding planes to the roofline.

Figure 4.19: Joint normal displacement distributions of joints at the immediate roof.

(a) Joint shear displacement distribution on cross section A2-A2 (entries).

(b) Joint shear displacement distribution on cross section B2-B2 (crosscuts).

(c) Joint shear displacement of closest bedding planes to the roofline.

Figure 4.20: Joint shear displacement distributions of joints at the immediate roof.

4.3.8.6 Failure State of Bedding Planes

Figure 4.21 depicts the failure state of the closest bedding planes at the roofline. The bedding planes fail in tension above the intersections. This finding matches with Figure 4.19c that shows the highest joint normal displacements at these locations.

Figure 4.21: Failure state of the closest bedding planes to the roofline.

4.3.9 Effect of Horizontal Stress Orientation on Roof Stability

The orientation of the maximum horizontal stress has a significant influence on roof stability. The objective of this parametric study was to investigate the influence of the horizontal stress direction on stability of weak roofs in underground excavations. In this study, the angle between the maximum horizontal stress and the crosscut axis is referred to as the maximum horizontal stress angle (β) and was measured with respect to the crosscut

axis as shown in Figure 4.22. Assuming an increment of 15° for the maximum horizontal stress angle, 7 cases with angles of 0° , 15° , 30° , 45° , 60° , 75° , and 90° were required to cover the different possible orientations of the maximum horizontal stress. The geometry of the model is perfectly symmetrical around $\beta = 90^\circ$ and approximately symmetric around $\beta = 0^\circ$. The effect on roof stability due to slight non-symmetry around $\beta = 0^\circ$ may be little. The highest effect on roof stability happens for β between 0 and 45° . Therefore, the number of the cases was reduced to 4. Because the numerical modeling results of the maximum horizontal stress angle of 0° (the maximum horizontal stress was in the direction of the crosscuts) were covered as the reference case in section 4.3.8, only three numerical modeling cases with the maximum horizontal stress angles of 15° , 30° and 45° were performed. The numerical modeling results are illustrated in Figures 4.23 – 4.24.

Figure 4.22: The maximum horizontal stress angle (β) with respect to x-axis.

From the numerical modeling results it can be seen that the variation of the maximum horizontal stress orientation influences the location and severity of the roof failure.

With increasing β the z-displacement decreases at the roofline but still the maximum

z-displacement occurs above the intersections. When the maximum horizontal stress is perpendicular to the direction of entries, $\beta=0^\circ$, the z-displacements at the roof centers above the entries are higher than those above the crosscuts. The increase in β angle increases the z-displacement at the roof centers above the crosscuts and decreases the z-displacement at the roof centers above the entries (see Figure 4.23).

From Figures 4.24 – 4.31 it can be seen how the orientation of the maximum horizontal stress changes the pattern and intensity of failures in the immediate roof area. The intensity of cutter roof failures above the crosscuts and above the entries, decreases and increases, respectively, with increasing β angle. The intensity of shear failure at roof centers above the entries and above the crosscuts, decreases and increases, respectively, with increasing β angle. In addition, with increasing β angle the cutter roof failures concentrate more at the roof area above the upper corners of the intersections. The results for $\beta=30^\circ$ and 45° show that the shear failures are likely to snake in-between the pillars and the orientation of shear failures at top of the four-way intersections change with the orientation of the maximum horizontal stress and are nearly in the direction of minimum horizontal stress (see Figures 4.26 and 4.29). Offsetting the pillars may help to avoid this type of failure.

The magnitudes of normal and shear displacements along the bedding planes decrease with increasing β angle and for $\beta=45^\circ$ they reach to their minimum values. But still the maximum joint separation takes place above the intersections. The direction of slippage along the bedding planes changes with the direction of maximum horizontal stress (see Figure 4.32 – 4.33).

As can be seen from Figure 4.34, the intensity of failure along the bedding planes at immediate roof decreases with the increase of β , and above the intersections the type of

failure in bedding planes changes from tensile to shear.

The numerical modeling results of this parametric study suggests that reorienting the entries to a direction 45° from either the minimum or the maximum horizontal stress direction can improve the roof condition.

Figure 4.23: Z-displacement distribution at the roofline on cross section C-C, for different values of β angle.

Figure 4.24: Distribution of accumulated plastic shear strain at the roofline on cross section C-C, for different values of β angle.

Figure 4.25: Distribution of accumulated plastic shear strain at the immediate roof on cross section D-D, for different values of β angle.

Figure 4.26: Distribution of accumulated plastic shear strain at the immediate roof on cross section E-E, for different values of β angle.

Figure 4.27: Distribution of post failure cohesion at the roofline on cross section C-C, for different values of β angle.

Figure 4.28: Distribution of post failure cohesion at the immediate roof on cross section D-D, for different values of β angle.

Figure 4.29: Distribution of post failure cohesion at the immediate roof on cross section E-E, for different values of β angle.

Figure 4.30: Distribution of post failure cohesion at the immediate roof on cross section A2-A2 (entries), for different values of β angle.

Figure 4.31: Distribution of post failure cohesion at the immediate roof on cross section B2-B2 (entries), for different values of β angle.

Figure 4.32: Joint normal displacement of closest bedding planes to the roofline at different values of β angle.

Figure 4.33: Joint shear displacement of closest bedding planes to the roofline at different values of β angle.

Figure 4.34: Failure state of the closest bedding planes to the roofline at different values of β angle.

4.3.10 Effect of the Maximum Horizontal Stress Magnitude on Roof Stability

According to the reference case description a constant vertical normal compressive stress of 2.75 MPa was applied at the top of the model; the minimum horizontal stress-to-vertical stress ratio was kept as one. To assess the influence of the maximum horizontal stress magnitude on stability of the roof, the maximum horizontal-to-vertical stress ratio, K , was increased from $K=1$, to 2 in steps of 0.5. The numerical modeling results obtained are illustrated in Figures 4.35 – 4.36.

With increasing K value, the z -displacement increases at the roofline and for all values of the K ; the maximum z -displacement occurs above the intersections. For K values greater than 1, the z -displacements above the entries are higher than that above the crosscuts at the roof centers (see Figure 4.35).

From Figures 4.36a, 4.37a and 4.38a, it can be seen that even for low values of horizontal stresses, $K=1$, the cutter roof failures take place in laminated weak roof. The intensity of cutter roof and shear failure at roof centers above the entries, decreases and increases, respectively, with the increase of maximum horizontal stress magnitude in the direction of crosscuts. The increase of maximum horizontal stress magnitude also increases the intensity of shear failure at roof centers above the entries (see Figures 4.36 – 4.43).

For all values of K , the maximum separation along the discontinuities takes place above the intersection. The increase of K value from 1 to 2, increased the magnitudes of maximum normal and shear displacements along the bedding planes by 7 mm and 9.5 mm respectively (see Figure 4.44 – 4.45).

As can be seen from Figure 4.46, with the increase of maximum horizontal stress magnitude, the intensity of the failures along the bedding planes increases and the type of

failure above the intersections changes from shear to tensile.

The modeling results described for this parametric study show that for the weak stratified roof, excessive maximum horizontal stress is not the only cause of the cutter roof failure and this type of failure can happen under the stress regimes with maximum horizontal stress close to the vertical stress in magnitude; and generally higher magnitudes of maximum horizontal stress will lead to adverse effect on roof.

Figure 4.35: The Z-displacement distribution at the roofline on cross section C-C for different values of K_0 .

Figure 4.36: Distribution of the accumulated plastic shear strain at the roofline on cross section C-C for different values of K_0 .

Figure 4.37: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different values of K_0 .

Figure 4.38: Distribution of the accumulated plastic shear strain at the immediate roof on cross section E-E for different values of K_0 .

Figure 4.39: Distribution of the post failure cohesion at the roofline on cross section C-C for different values of K_0 .

Figure 4.40: Distribution of the post failure cohesion at the immediate roof on cross section D-D for different values of K_0 .

Figure 4.41: Distribution of the post failure cohesion at the immediate roof on cross section E-E for different values of K_0 .

Figure 4.42: Distribution of the post failure cohesion at the immediate roof on cross section A2-A2 (entries) for different values of K_0 .

Figure 4.43: Distribution of the post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts) for different values of K_0 .

Figure 4.44: Joint normal displacement of the closest bedding planes to the roofline for different values of K_0 .

Figure 4.45: The joint shear displacement of the closest bedding planes to the roofline for different values of K_0 .

Figure 4.46: Failure state of the closest bedding planes to the roofline for different values of K_0 .

4.3.11 Effect of the Roof Rock Strength on Roof Stability

To account for the uncertainty of the rock mass strength, it is assumed that the coefficient of variation (COV) of the immediate roof rock mass strength, laminated silty shale, is about 15%. Using the assumed coefficient of variation, the standard deviations of the cohesion, angle of internal friction and tensile strength of the roof rock mass were calculated. For the numerical modeling, each strength parameter was set to the mean value plus and minus one standard deviation while the other two strength parameters were held at their mean values. Since for the reference case the modeling was performed based on the mean values of the strength parameters, only 6 calculations (two for each strength parameter) were needed.

Table 4.3 provides the summary of the roof rock strength values that were used as inputs in the numerical models.

Table 4.3: The roof rock strength values that were used as inputs in the numerical models.

Parameter	Mean value	STD*	M**+STD	M-STD
Cohesion (MPa)	5.1	0.765	5.865	4.335
Friction angle (°)	21	3.15	24.15	17.85
Tensile strength (MPa)	2.4	0.36	2.76	2.04

* STD = Standard deviation, ** M= Mean value

4.3.11.1 Effect of the Roof Rock Cohesion on Roof Stability

To study the influence of the immediate roof rock cohesion on stability of the roof, three different models were simulated using the mean and mean plus/minus one standard deviation of the immediate roof rock cohesion values. The obtained numerical modeling results are illustrated in Figures 4.47 – 4.58.

Figure 4.47 depicts that with the increase of roof rock cohesion value from 4.34 MPa to 5.87 MPa in steps of 0.765 MPa (standard deviation), the z-displacement decreases at

the roofline. For all the values of cohesion, the maximum z-displacement occurs above the intersections except the case with cohesion value of 4.34 MPa where z-displacement is maximum above the entries at roof centers.

From Figures 4.48a, 4.50a and 4.51a it can be seen that for the lowest value of the cohesion, 4.34 MPa, the shear failures take place at the upper edges of the entries and crosscuts (cutter roof) as well as at the roof centers above the entries. The intensity of the cutter roof, and shear failure at roof centers above the entries decreases with the increase of cohesion value (see Figures 4.48 – 4.55). As it can be seen from Figure 4.51, for the stronger roof compared with the weak type roof, the cutter roof failure at the roof line are more severe in the direction of the minimum horizontal stress above the entries.

The magnitudes of normal and shear displacements along the bedding planes decreases with the increase of cohesion (see Figures 4.56 and 4.57) and the maximum joint separation still takes place above the intersections as expected, except the case with cohesion of 4.34 MPa where the maximum joint separation happens above the entries (see Figure 4.56).

As seen in Figure 4.58, with the increase of cohesion value, the shear and tensile failures along the bedding planes are reduced.

From the modeling results it can be concluded that the cohesion of the roof strata has significant influence on the behavior of the roof.

Figure 4.47: The Z-displacement distribution at the roofline on cross section C-C for different values of roof rock cohesion.

Figure 4.48: Distribution of the accumulated plastic shear strain at the roofline on cross section C-C for different values of roof rock cohesion.

Figure 4.49: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different values of roof rock cohesion.

Figure 4.50: Distribution of the accumulated plastic shear strain at the immediate roof on cross section E-E for different values of roof rock cohesion.

Figure 4.51: Distribution of the post failure cohesion at the roofline on cross section C-C for different values of roof rock cohesion.

Figure 4.52: Distribution of the post failure cohesion at the immediate roof on cross section D-D for different values of roof rock cohesion.

Figure 4.53: Distribution of the post failure cohesion at the immediate roof on cross section E-E for different values of roof rock cohesion.

Figure 4.54: Distribution of the post failure cohesion at the immediate roof on cross section A2-A2 (entries) for different values of roof rock cohesion.

Figure 4.55: Distribution of the post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts) for different values of roof rock cohesion.

Figure 4.56: Joint normal displacement of the closest bedding planes to the roofline for different values of roof rock cohesion.

Figure 4.57: The joint shear displacement of the closest bedding planes to the roofline for different values of roof rock cohesion.

Figure 4.58: Failure state of the closest bedding planes to the roofline for different values of roof rock cohesion.

4.3.11.2 Effect of the Roof Rock Internal Friction Angle on Roof Stability

The increase of the friction angle of the material forming the roof strata causes a reduction in the z-displacement values at the roof line (see Figure 4.59).

From Figures 4.60 – 4.67 it can be seen that the intensity of shear failure at roof line decreases with the increase of roof rock friction angle and for the highest value of the friction angle, 24.15° , the shear failures are mainly concentrated at central part of the panel and mid span of the entries.

As shown in Figures 4.68 and 4.69, the magnitudes of normal and shear displacements along the bedding planes decrease with increase of the roof rock friction angle. The shear and tensile failures along the bedding planes decrease with increase of the roof rock friction angle (see Figure 4.70).

The numerical modeling results show that the friction angle of the roof rock has a significant impact on the stability of the roof.

Figure 4.59: The Z-displacement distribution at the roofline on cross section C-C for different values of roof rock friction angle.

Figure 4.60: Distribution of the accumulated plastic shear strain at the roofline on cross section C-C for different values of roof rock friction angle.

Figure 4.61: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different values of roof rock friction angle.

Figure 4.62: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different values of roof rock friction angle.

Figure 4.63: Distribution of the post failure cohesion at the roofline on cross section C-C for different values of roof rock friction angle.

Figure 4.64: Distribution of the post failure cohesion at the immediate roof on cross section D-D for different values of roof rock friction angle.

Figure 4.65: Distribution of the post failure cohesion at the immediate roof on cross section E-E for different values of roof rock friction angle.

Figure 4.66: Distribution of the post failure cohesion at the immediate roof on cross section A2-A2 (entries) for different values of roof rock friction angle.

Figure 4.67: Distribution of the post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts) for different values of roof rock friction angle.

Figure 4.68: Joint normal displacement of the closest bedding planes to the roofline for different values of roof rock friction angle.

Figure 4.69: The joint shear displacement of the closest bedding planes to the roofline for different values of roof rock friction angle.

Figure 4.70: Failure state of the closest bedding planes to the roofline for different values of roof rock friction angle.

4.3.11.3 Effect of the Roof Rock Tensile Strength on Roof Stability

As seen from the numerical modeling results, the increase of roof rock tensile strength from 2.04 MPa to 2.76 MPa in steps of 0.36 MPa, has slightly decreased the z-displacement at the roof line (see Figure [4.71](#)).

Figures [4.72](#) – [4.80](#) show that the intensity of the shear failure and joint normal displacement have also slightly increased and decreased, respectively, with the increase of roof rock tensile strength. The roof rock tensile strength has only slight or negligible effect on all performance measures selected to assess roof stability in this case study.

Figure 4.71: The Z-displacement distribution at the roofline on cross section C-C for different values of roof rock tensile strength.

Figure 4.72: Distribution of the accumulated plastic shear strain at the roofline on cross section C-C for different values of roof rock tensile strength.

Figure 4.73: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different values of roof rock tensile strength.

Figure 4.74: Distribution of the accumulated plastic shear strain at the immediate roof on cross section E-E for different values of roof rock tensile strength.

Figure 4.75: Distribution of the post failure cohesion at the roofline on cross section C-C for different values of roof rock tensile strength.

Figure 4.76: Distribution of the post failure cohesion at the immediate roof on cross section D-D for different values of roof rock tensile strength.

Figure 4.77: Distribution of the post failure cohesion at the immediate roof on cross section E-E for different values of roof rock tensile strength.

Figure 4.78: Distribution of the post failure cohesion at the immediate roof on cross section A2-A2 (entries) for different values of roof rock tensile strength.

Figure 4.79: Distribution of the post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts) for different values of roof rock tensile strength.

Figure 4.80: Joint normal displacement of the closest bedding planes to the roofline for different values of roof rock tensile strength.

Figure 4.81: The joint shear displacement of the closest bedding planes to the roofline for different values of roof rock tensile strength.

Figure 4.82: Failure state of the closest bedding planes to the roofline for different values of roof rock tensile strength.

4.3.12 Effect of Discontinuity Mechanical Properties on Roof Stability

The field measurements have confirmed the occurrence of slip and separation along the interfaces and bedding planes in the laminated roof strata part of the case study mine. The bedding planes and interfaces have very low or close to zero tensile strength perpendicular to the discontinuities and their shear strength is much lower than that of the rock layers. Therefore, due to the stress concentrations caused by excavations, slippage and separation can easily happen along the interfaces and bedding planes before initiation of failure within the rock layers. The intensity of slippage and bed separation in underground openings depends on the rock types as well as the bonding strength of the beds and magnitudes of the loads. The slippage and separation of bedding planes can magnify the adverse effects of stress concentrations around the excavation. Hence, the bedding planes play an important role on roof stability in underground excavations and the thinly bedded rock should be considered as a problematical roof in underground excavations.

To account for the uncertainty of the major discontinuity properties, the coefficient of variation (COV) of 15% was adopted. Using the assumed coefficient of variation, the standard deviations of the joint normal stiffness, joint shear stiffness and joint friction angle were calculated. For the numerical modeling, each parameter was set to the mean value plus and minus one standard deviation while the other two parameters were held at their mean values. Table 4.4 provides the summary of the discontinuity parameters that were used as inputs in the numerical models.

Table 4.4: The discontinuity parameter values that were used as inputs in the numerical models.

Parameter	Mean value	STD*	M**+ STD	M-STD
Joint Friction Angle (°)	17.5	2.625	20.125	14.875
Joint Normal Stiffness (Pa/m)	4.07E+08	6.11e7	4.68e8	3.46e8
Joint Shear Stiffness (Pa/m)	1.65E+07	2.48e6	1.90e7	1.40e7

* STD = Standard deviation, ** M= Mean value

4.3.12.1 Effect of Discontinuity Friction Angle on Roof Stability

To study the influence of the bedding plane friction angle on stability of the roof, three different models were simulated using the mean and mean plus/minus one standard deviation of the bedding plane friction angle values. The obtained numerical modeling results are illustrated in Figures 4.83 – 4.94.

Figure 4.83 shows that with increase of the friction angle of the bedding planes from 14.875° to 20.125° in steps of 2.625° (standard deviation), there is no significant change in z-displacement magnitudes at the roofline. For all the values of bedding plane friction angle, the maximum z-displacement occurs above the intersections.

From Figures 4.84 – 4.91 it can be seen that the intensity of the shear failure increases slightly with increase of the bedding plane friction angle.

The friction angle of the bedding planes has negligible influence on the magnitudes of normal and shear displacements along the bedding planes (see Figures 4.92 and 4.93).

As expected, the shear failure along the bedding planes has decreased with the increase of bedding plane friction angle (see Figure 4.94).

From the modeling results it can be concluded that a change in the friction angle of

the bedding planes does not impact z-displacement or normal and shear displacements along the bedding planes. It mainly influences the shear failure in the rock mass and along bedding planes in the immediate roof area.

Figure 4.83: The Z-displacement distribution at the roofline on cross section C-C for different values of joint friction angle.

Figure 4.84: Distribution of the accumulated plastic shear strain at the roofline on cross section C-C for different values of joint friction angle.

Figure 4.85: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different values of joint friction angle.

Figure 4.86: Distribution of the accumulated plastic shear strain at the immediate roof on cross section E-E for different values of joint friction angle.

Figure 4.87: Distribution of the post failure cohesion at the roofline on cross section C-C for different values of joint friction angle.

Figure 4.88: Distribution of the post failure cohesion at the immediate roof on cross section D-D for different values of joint friction angle.

Figure 4.89: Distribution of the post failure cohesion at the immediate roof on cross section E-E for different values of joint friction angle.

Figure 4.90: Distribution of the post failure cohesion at the immediate roof on cross section A2-A2 (entries) for different values of joint friction angle.

Figure 4.91: Distribution of the post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts) for different values of joint friction angle.

Figure 4.92: Joint normal displacement of the closest bedding planes to the roofline for different values of joint friction angle.

Figure 4.93: The joint shear displacement of the closest bedding planes to the roofline for different values of joint friction angle.

Figure 4.94: Failure state of the closest bedding planes to the roofline for different values of joint friction angle.

4.3.12.2 Effect of Discontinuity Normal Stiffness on Roof Stability

To investigate the effect of normal stiffness of bedding planes on stability of the roof, three different models were simulated using the mean and mean plus/minus one standard deviation of the normal stiffness values of the bedding planes. The obtained numerical modeling results are shown in Figures 4.95 – 4.106.

With the increase of normal stiffness of bedding planes from $3.46\text{e}8$ Pa/m to $4.68\text{e}8$ in steps of $6.11\text{e}7$ Pa/m, the magnitude of maximum z-displacement at the roof line decreases from 6.4 cm to 5 cm (see Figure 4.95).

From Figures 4.96 – 4.103 it can be seen that the intensity of the cutter roof failures and shear failure at roof centers above the entries, decreases and increases, respectively with the increase of joint normal stiffness.

The magnitudes of normal and shear displacements along the bedding planes decrease with the increase of joint normal stiffness (see Figures 4.104 and 4.105).

As seen in Figure 4.106, the shear and tensile failures along the bedding planes decrease with the increase of joint normal stiffness.

From the modeling results it can be concluded that the joint normal stiffness value has a significant influence on roof stability.

Figure 4.95: The Z-displacement distribution at the roofline on cross section C-C for different values of joint normal stiffness.

Figure 4.96: Distribution of the accumulated plastic shear strain at the roofline on cross section C-C for different values of joint normal stiffness.

Figure 4.97: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different values of joint normal stiffness.

Figure 4.98: Distribution of the accumulated plastic shear strain at the immediate roof on cross section E-E for different values of joint normal stiffness.

Figure 4.99: Distribution of the post failure cohesion at the roofline on cross section C-C for different values of joint normal stiffness.

Figure 4.100: Distribution of the post failure cohesion at the immediate roof on cross section D-D for different values of joint normal stiffness.

Figure 4.101: Distribution of the post failure cohesion at the immediate roof on cross section E-E for different values of joint normal stiffness.

Figure 4.102: Distribution of the post failure cohesion at the immediate roof on cross section A2-A2 (entries) for different values of joint normal stiffness.

Figure 4.103: Distribution of the post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts) for different values of joint normal stiffness.

Figure 4.104: Joint normal displacement of the closest bedding planes to the roofline for different values of joint normal stiffness.

Figure 4.105: The joint shear displacement of the closest bedding planes to the roofline for different values of joint normal stiffness.

Figure 4.106: Failure state of the closest bedding planes to the roofline for different values of joint normal stiffness.

4.3.12.3 Effect of Discontinuity Shear Stiffness on Roof Stability

To investigate the effect of shear stiffness of the bedding planes on stability of the roof, three different models were simulated using the mean and mean plus/minus one standard deviation of the shear stiffness values of the bedding planes. The obtained numerical modeling results are illustrated in Figures [4.107](#) – [4.118](#).

As seen from the numerical modeling results, the increase of the shear stiffness of bedding planes from $1.40\text{e}7$ Pa/m to $1.90\text{e}7$ Pa/m in steps of $2.5\text{e}6$ Pa/m, has negligible effect on all performance measures selected to assess roof stability in this case study; it only slightly decreased the shear displacement along the bedding planes (see Figure [4.117](#)).

Figure 4.107: The Z-displacement distribution at the roofline on cross section C-C for different values of joint shear stiffness.

Figure 4.108: Distribution of the accumulated plastic shear strain at the roofline on cross section C-C for different values of joint shear stiffness.

Figure 4.109: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different values of joint shear stiffness.

Figure 4.110: Distribution of the accumulated plastic shear strain at the immediate roof on cross section E-E for different values of joint shear stiffness.

Figure 4.111: Distribution of the post failure cohesion at the roofline on cross section C-C for different values of joint shear stiffness.

Figure 4.112: Distribution of the post failure cohesion at the immediate roof on cross section D-D for different values of joint shear stiffness.

Figure 4.113: Distribution of the post failure cohesion at the immediate roof on cross section E-E for different values of joint shear stiffness.

Figure 4.114: Distribution of the post failure cohesion at the immediate roof on cross section A2-A2 (entries) for different values of joint shear stiffness.

Figure 4.115: Distribution of the post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts) for different values of joint shear stiffness.

Figure 4.116: Joint normal displacement of the closest bedding planes to the roofline for different values of joint shear stiffness.

Figure 4.117: The joint shear displacement of the closest bedding planes to the roofline for different values of joint shear stiffness.

Figure 4.118: Failure state of the closest bedding planes to the roofline for different values of joint shear stiffness.

4.3.13 Effect of Variation in Room and Pillar Size on Roof Stability

In room and pillar mines, the relation between the room and pillar sizes is known as the extraction ratio. The increase (decrease) in the pillar size results as a decrease (increase) in the room size and therefore, the extraction ratio decreases and increases respectively with increase and decrease of the pillar size.

To study the influence of the pillar and room sizes on stability of the roof, three different models were simulated. In the first case, pillar width of the reference case was increased by 10%, and in the second case, it was decreased by the same amount, see Table 4.5. The obtained numerical modeling results are illustrated in Figures 4.119 – 4.131.

Figure 4.119 depicts that with the increase of pillar size from 11.40 m to 14.0 m in steps of 1.30 m, the magnitude of maximum z-displacement at the roofline decreases from 7.5 cm to 3 cm and for all the values of pillar sizes, the maximum z-displacement occurs above the intersections.

From Figure 4.120 it can be seen that the intensity of the tensile failures at roof centers in roofline decreases with the increase of pillar size.

The intensity of the cutter roof, and shear failures at roof centers above the entries decreases with the increase (decrease) of pillar (room) size (see Figures 4.121 – 4.128).

The magnitudes of normal and shear displacements along the bedding planes decrease with the increase of pillar size (see Figures 4.129 and 4.130) and the maximum joint separation still takes place above the intersections as expected.

As seen in Figure 4.131, with the increase of pillar size, the shear and tensile failures along the bedding planes are reduced.

From the modeling results it can be concluded that the pillar size has significant influence on the behavior of the roof.

Table 4.5: Room and pillar sizes and the resultant extraction ratios.

	Room Width (m)	Pillar Width (m)	Extraction Ratio
Case 1	7.2	11.4	62.4%
Case 2	6.1	12.7	54.3%
Case 3	5.0	14.0	45.7%

Figure 4.119: The Z-displacement distribution at the roofline on cross section C-C for different pillar sizes.

Figure 4.120: Distribution of the accumulated plastic tensile strain at the roofline on cross section C-C for different pillar sizes.

Figure 4.121: Distribution of the accumulated plastic shear strain at the roofline on cross section C-C for different pillar sizes.

Figure 4.122: Distribution of the accumulated plastic shear strain at the immediate roof on cross section D-D for different pillar sizes.

Figure 4.123: Distribution of the accumulated plastic shear strain at the immediate roof on cross section E-E for different pillar sizes.

Figure 4.124: Distribution of the post failure cohesion at the roofline on cross section C-C for different pillar sizes.

Figure 4.125: Distribution of the post failure cohesion at the immediate roof on cross section D-D for different pillar sizes.

Figure 4.126: Distribution of the post failure cohesion at the immediate roof on cross section E-E for different pillar sizes.

Figure 4.127: Distribution of the post failure cohesion at the immediate roof on cross section A2-A2 (entries) for different pillar sizes.

Figure 4.128: Distribution of the post failure cohesion at the immediate roof on cross section B2-B2 (crosscuts) for different pillar sizes.

Figure 4.129: Joint normal displacement of the closest bedding planes to the roofline for different pillar sizes.

Figure 4.130: The joint shear displacement of the closest bedding planes to the roofline for different pillar sizes.

Figure 4.131: Failure state of the closest bedding planes to the roofline for different pillar sizes.

Chapter 5

Summary, Conclusions and Future Recommendations

5.1 Summary and Conclusions

The primary goal of this research was to develop an efficient and robust three-dimensional distinct element based model to analyze the roof stability in underground coal mines. As discussed in Chapter 2, the complexity of the three-dimensional distinct element methods has pushed researchers to use empirical techniques and continuum based numerical methods beyond the limits of their validity for addressing the roof stability in underground coal mines.

An underground coal mine at western Pennsylvania is selected as a case study mine to investigate the underlying causes of roof falls at this mine. The immediate roof at the case study mine consists of laminated silty shale, shale, or sandstone that changes from area to area, and the floor is shale or soft fireclay. This study was mainly focused in the stability analysis of the roofs with the laminated silty shale rock type, where the majority of roof falls had taken place in the roof with this type of roof material. Extensive

laboratory tests were performed on the core samples obtained from the case study mine to estimate the intact rock and discontinuity properties of the materials that occur in large extent at the selected interest area of the case study mine. Laminated or thinly bedded shale, is one of the most frequently seen overlaying strata above the coal deposits. The bedding planes and interfaces have very low or close to zero tensile strength in the direction perpendicular to the bedding planes, and their shear strength is much lower than of the rock layers. Therefore, due to the stress concentrations caused by excavations, slippage and separation can easily happen along the bedding planes before initiation of failure within the rock layers. In the presence of discontinuities, the distribution of stresses and deformations induced in a rock mass can differ significantly from those predicted by continuum based techniques. The deformation modulus, strength and stress-strain responses of discontinuous rock masses can all be affected by discontinuities in a nonlinear and anisotropic manner. The homogenization and combined continuum–interface approaches are two commonly employed procedures in continuum based methods to account for the presence of discontinuities in rock masses. However, the methods based on these approaches are limited by the fact that the deformation mechanisms involving movements such as separation, slip and rotation of blocks cannot be modeled properly by these methods. Therefore, realistic modeling of the mechanical behavior of discontinuities is essential for successful numerical modeling of discontinuous rock masses. The employed numerical technique should be able to capture the broad range of rock mass behaviors caused by interactions between the different moduli and strengths of intact rock, discontinuity mechanical properties and support system. In contrast to the continuous based methods, distinct element methods provide more accurate and realistic description of the physical behavior of discontinuous rock masses. In this research, the three-dimensional distinct element method was used to investigate the stability

of the roof in an underground room-and-pillar coal mine. The implemented technique can accurately capture the possible failure of the discontinuities and intact material part of the rock masses. In order to accurately replicate the post failure behavior of the rock layers in the immediate roof area, the strain-softening material constitutive law was applied to this region. To accurately capture the bending and buckling behavior of the thinly laminated materials at the roof strata, each individual layer is meshed such that at least five overlaying elements are created across the section of each thin layer.

Extensive numerical parametric studies were conducted to investigate the effect of different parameters such as the variation of immediate roof rock mass strength properties, variation of discontinuity mechanical properties, orientations and magnitudes of the horizontal in-situ stresses, and the size of pillars and excavations on stability of the excavations. The distribution of post failure cohesion along with other measures such as accumulated plastic shear strain, distribution of Z-displacements at the roofline, failure state (joint slip and tensile failure) and displacement (normal and shear displacements) of discontinuities were used to accurately assess the roof stability in this case study.

The following are the major conclusions drawn from the three dimensional distinct element modeling performed in this dissertation:

The effect of variation of the immediate roof rock mass strength properties on roof stability was investigated and found to be significant. The conclusions derived from the comprehensive parametric studies on the significance of roof rock strength parameters on roof stability are summarized as follows:

- The magnitude of z-displacement at roof line, the intensity of shear and tensile failures along the bedding planes, the magnitude of shear and normal displacements along the

bedding planes, and the intensity of the cutter roof, and shear failure at roof centers above the entries decrease with the increase of roof rock cohesion and friction angle values.

- The magnitude of the roof strata friction angle impacts the pattern and location of the shear failures at the immediate roof area.
- The roof rock tensile strength has only slight or negligible effect on all performance measures selected to assess the roof stability in this case study.

The magnitude and orientation of the maximum in-situ horizontal stress are known to significantly impact the stability of the underground excavations. In this research, different case studies were performed to study the influence of the magnitude and orientation of the in-situ horizontal stress on roof stability. The summary of the conclusions made from these case studies are as follows:

- The orientation of the maximum in-situ horizontal stress changes the pattern and intensity of the failures in the roof strata.
- The intensity of cutter roof failures above the crosscuts and entries, decreases and increases, respectively, with increasing maximum horizontal in situ stress angle.
- The magnitude of normal and shear displacements along the bedding planes decreases with the increase of maximum horizontal in situ stress angle, β , and for $\beta=45^\circ$ they reach to their minimum values.
- The results for $\beta=30^\circ$ and 45° show that the shear failures are likely to snake in-between the pillars and the orientation of shear failures at top of the four-way intersections

changes with the orientation of the maximum horizontal in-situ stress and are nearly in the direction of minimum horizontal in-situ stress. Offsetting the pillar might help to avoid this type of failure.

- As was expected, the direction of slippage along the bedding planes changes with the direction of the maximum horizontal in-situ stress.
- The intensity of failure along the bedding planes at the immediate roof decreases with the increase of β , and above the intersections the type of failure in bedding planes changes from tensile to shear.
- The intensity of the failures along the bedding planes increases with the increase of maximum horizontal in situ stress magnitude.
- The increase in the magnitude of the maximum horizontal in situ stress increases the intensity of the failures along the bedding planes.
- The magnitude of z-displacement at the roofline increases with increase of maximum horizontal in situ stress magnitude.

The bedding planes and interfaces are significantly impacting the stability of the roof in underground excavations. Based on the parametric studies conducted in this research, the following conclusions are drawn:

- The magnitude of the bedding plane friction angle has negligible effect on deformation of the immediate roof strata and the magnitude of normal and shear displacements along the bedding planes.

- As expected, the intensity of shear failure along the bedding planes decreases with the increase of the bedding plane friction angle.
- The intensity of the cutter roof, and shear failure at roof centers above the entries slightly increases with the increase of the bedding plane friction angle.
- The slippage and separation of bedding planes are amplifying the adverse effects of stress concentrations around the excavation.
- The normal stiffness of bedding planes has a significant impact on roof stability.
- The magnitude of the maximum z-displacement at the roof line decreases remarkably with the increase of the normal stiffness of bedding planes.
- The intensity of cutter roof failures and shear failures at roof centers above the entries, decreases and increases, respectively as the joint normal stiffness value increases.
- The magnitudes of normal and shear displacements as well as the intensity of shear and tensile failures along the bedding planes decrease with the increase of bedding plane normal stiffness value.
- The increase of the shear stiffness of bedding planes has negligible effect on all performance measures selected to assess roof stability in this case study and it only slightly decreases the shear displacement along the bedding planes.

The relative size of the room and pillars represents a compromise between maintaining the stability of the excavations and extraction ratio of the coal. The following conclusions can be made about the impact of the room and pillar sizes on the stability of the roof in underground coal mines:

- With the increase (decrease) of the pillar (room) size, the stability of the roof improves significantly.
- The magnitude of maximum z-displacement at the roofline decreases considerably with the increase (decrease) of the pillar (room) size.
- The increase of the pillar size decreases the magnitudes of normal and shear displacements along the bedding planes meaningfully.
- The intensity of the cutter roof, and shear failures at roof centers above the entries decreases considerably with the increase (decrease) of the pillar (room) size.
- The intensity of the tensile failures at roof centers above the entries and crosscuts in roofline decreases with the increase of the pillar size.
- A noticeable decrease in the intensity of the shear and tensile failures along the bedding planes occurs with the increase of the pillar size.

In summary, the research conducted in this dissertation showed that the bedding planes play an important role on the behavior of roof in underground excavations. Therefore, an appropriate numerical modeling technique which incorporates the effect of discontinuities should be employed to simulate the realistic behavior of the discontinuous rock masses such as the layered materials in roof strata of the underground coal mines. The three-dimensional distinct element method used in this research showed the clear superiority of this technique over the continuum based methods. The laboratory tests are essential for determination of intact rock and discontinuity properties to make accurate predictions of real world underground stability problems. The roof failure in underground excavations is most likely a progressive failure that starts at the weakest part of the rock mass where

the stress concentrations are higher than the roof rock strength and then propagates in the least resistance path within the rock mass. Because of the limitations of the standard Mohr-Coulomb constitutive law, it is impossible to simulate this process using this model and, as shown in this research, the strain-softening constitutive law is well-suited for this purpose.

5.2 Future Research

Based on the investigations carried out in this present study, the following factors still need to be considered in order to develop a better understanding of the underlying causes of roof instability in underground coal mines:

- In this research, due to the limitation of 3DEC 4.1 in handling large scale models, the whole excavation was created in one step. It is recommended to incorporate the sequence of excavations into the numerical modeling in future research.
- For the considered mine the pore water pressure was not an issue. However, the pore water pressures can significantly impact the stability of certain underground excavations. It is recommended to study the pore pressure effect on performance of the bedding planes and on roof stability of underground coal mines.
- Different aspects of support design such as the properties, spacing and length of the cables and bolts can be considered in future research.
- In the conducted research, the main focus was on the impact of the bedding planes on the behavior of the immediate roof; in future research, it is recommended to consider also the effect of presence of inclined or vertical joints on performance of the

immediate roof.

Appendix	A
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Lab Test Results

Figure A.1: The plot of shear stress versus shear displacement for sample # 1 of Shale under four different normal stresses.

Figure A.2: The plot of shear stress versus shear displacement for sample # 2 of Shale under four different normal stresses.

Figure A.3: The plot of shear stress versus shear displacement for sample # 3 of Shale under four different normal stresses.

Figure A.4: The plot of shear stress versus shear displacement for sample # 1 of Sandstone under four different normal stresses.

Figure A.5: The plot of shear stress versus shear displacement for sample # 2 of Sandstone under four different normal stresses.

Figure A.6: The plot of shear stress versus shear displacement for sample # 3 of Sandstone under four different normal stresses.

Figure A.7: The plot of shear stress versus shear displacement for sample # 4 of Sandstone under four different normal stresses.

Figure A.8: The plot of shear stress versus shear displacement for sample # 1 of Sandy Shale under four different normal stresses.

Figure A.9: The plot of shear stress versus shear displacement for sample # 2 of Sandy Shale under four different normal stresses.

Figure A.10: The plot of shear stress versus shear displacement for sample # 3 of Sandy Shale under four different normal stresses.

Figure A.11: The plot of shear stress versus shear displacement for sample # 4 of Sandy Shale under four different normal stresses.

Figure A.12: The plot of shear stress versus shear displacement for sample # 1 of Laminated Silty Shale under four different normal stresses.

Figure A.13: The plot of shear stress versus shear displacement for sample # 2 of Laminated Silty Shale under four different normal stresses.

Figure A.14: The plot of shear stress versus shear displacement for sample # 3 of Laminated Silty Shale under four different normal stresses.

Figure A.15: The plot of shear stress versus shear displacement for sample # 4 of Laminated Silty Shale under four different normal stresses.

Figure A.16: Direct shear test data and linear fit for peak shear strength of sample # 1 of Shale.

Figure A.17: Direct shear test data and linear fit for residual shear strength of sample # 1 of Shale.

Figure A.18: Direct shear test data and linear fit for shear strength of sample # 2 of Shale.

Figure A.19: Direct shear test data and linear fit for shear strength of sample # 3 of Shale.

Figure A.20: Direct shear test data and linear fit for shear strength of sample # 1 of Sandstone.

Figure A.21: Direct shear test data and linear fit for shear strength of sample # 2 of Sandstone.

Figure A.22: Direct shear test data and linear fit for shear strength of sample # 3 of Sandstone.

Figure A.23: Direct shear test data and linear fit for residual strength of sample # 4 of Sandstone.

Figure A.24: Direct shear test data and linear fit for peak strength of sample # 4 of Sandstone.

Figure A.25: Direct shear test data and linear fit for shear strength of sample # 1 of Sandy Shale.

Figure A.26: Direct shear test data and linear fit for shear strength of sample # 2 of Sandy Shale.

Figure A.27: Direct shear test data and linear fit for shear strength of sample # 3 of Sandy Shale.

Figure A.28: Direct shear test data and linear fit for shear strength of sample # 4 of Sandy Shale.

Figure A.29: Direct shear test data and linear fit for residual strength of sample # 1 of Laminated Silty Shale.

Figure A.30: Direct shear test data and linear fit for residual strength of sample # 2 of Laminated Silty Shale.

Figure A.31: Direct shear test data and linear fit for residual strength of sample # 3 of Laminated Silty Shale.

Figure A.32: Direct shear test data and linear fit for peak strength of sample # 3 of Laminated Silty Shale.

Figure A.33: Direct shear test data and linear fit for residual strength of sample # 4 of Laminated Silty Shale.

Figure A.34: Direct shear test data and linear fit for peak strength of sample # 4 of Laminated Silty Shale.

Figure A.35: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Laminated Silty Shale.

Figure A.36: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Sandstone.

Figure A.37: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Sandy Shale.

Figure A.38: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Shale.

Figure A.39: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Sandstone- Shale

Figure A.40: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Shale-Sandy Shale.

Figure A.41: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Laminated Silty Shale - Shale.

Figure A.42: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Sandstone - Sandy Shale.

Figure A.43: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Laminated Silty Shale - Sandy Shale.

Figure A.44: Plots of total deformation versus axial stress and intact rock deformation versus axial stress for Laminated Silty Shale – Sandstone.

Figure A.45: Plot of joint normal deformation versus axial stress for Laminated Silty Shale.

Figure A.46: Plot of joint normal deformation versus axial stress for Sandstone.

Figure A.47: Plot of joint normal deformation versus axial stress for Sandy Shale.

Figure A.48: Plot of joint normal deformation versus axial stress for Shale.

Figure A.49: Plot of joint normal deformation versus axial stress for Laminated Silty Shale – Sandstone joint.

Figure A.50: Plot of joint normal deformation versus axial stress for Laminated Silty Shale – Sandy Shale joint.

Figure A.51: Plot of joint normal deformation versus axial stress for Laminated Silty Shale – Shale joint.

Figure A.52: Plot of joint normal deformation versus axial stress for Sandstone – Sandy Shale joint.

Figure A.53: Plot of joint normal deformation versus axial stress for Sandstone – Shale joint.

Figure A.54: Plot of joint normal deformation versus axial stress for Shale – Sandy Shale joint.

Figure A.55: The plot of axial stress versus the joint normal deformation and exponential curve fit for Laminated Silty Shale.

Figure A.56: The plot of axial stress versus the joint normal deformation and exponential curve fit for Sandstone.

Figure A.57: The plot of axial stress versus the joint normal deformation and exponential curve fit for Sandy Shale.

Figure A.58: The plot of axial stress versus the joint normal deformation and exponential curve fit for Shale.

Figure A.59: The plot of axial stress versus the joint normal deformation and exponential curve fit for Laminated Silty Shale – Sandstone joint.

Figure A.60: The plot of axial stress versus the joint normal deformation and exponential curve fit for Laminated Silty Shale – Sandy Shale joint.

Figure A.61: The plot of axial stress versus the joint normal deformation and exponential curve fit for Laminated Silty Shale – Shale joint.

Figure A.62: The plot of axial stress versus the joint normal deformation and exponential curve fit for Sandstone – Sandy Shale joint.

Figure A.63: The plot of axial stress versus the joint normal deformation and exponential curve fit for Sandstone – Shale joint.

Figure A.64: The plot of axial stress versus the joint normal deformation and exponential curve fit for Shale – Sandy Shale joint.

Figure A.65: The plot of joint normal stiffness versus normal stress for Laminated Silty Shale.

Figure A.66: The plot of joint normal stiffness versus normal stress for Sandstone.

Figure A.67: The plot of joint normal stiffness versus normal stress for Sandy Shale.

Figure A.68: The plot of joint normal stiffness versus normal stress for, Sandy Shale.

Figure A.69: The plot of joint normal stiffness versus normal stress for Laminated Shale – Sandstone joint.

Figure A.70: The plot of joint normal stiffness versus normal stress for Laminated Shale – Sandy Shale joint.

Figure A.71: The plot of joint normal stiffness versus normal stress for Laminated Shale – Shale joint.

Figure A.72: The plot of joint normal stiffness versus normal stress for Sandstone – Sandy Shale joint.

Figure A.73: The plot of joint normal stiffness versus normal stress for Sandstone – Shale joint.

Figure A.74: The plot of joint normal stiffness versus normal stress for Shale – Sandy Shale joint.

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