INVESTIGATION INTO MINE PILLAR DESIGN AND GLOBAL STABILITY USING THE GROUND REACTION CURVE CONCEPT

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INVESTIGATION INTO MINE PILLAR DESIGN AND GLOBAL STABILITY USING THE GROUND REACTION CURVE CONCEPT

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in the College of Engineering at the University of Kentucky

By

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ABSTRACT OF DISSERTATION

INVESTIGATION INTO MINE PILLAR DESIGN AND GLOBAL STABILITY
USING THE GROUND REACTION CURVE CONCEPT

Pillars form an important support structure in any underground mine. A bulk of the overburden load is borne by the mine pillars. Thus, the strength of pillars has been a subject of detailed research over more than 6 decades. This work has led to the development of largely empirical pillar design formulations that have reduced the risk of pillar failures and mine collapse. Current research, however, has drawn attention to the fact that some of the assumptions used in the development of conventional pillar design methodologies are not always valid. Conventional pillar design methodology assumes that the pillars carry the dead weight of the overburden. This conventional method treats the pillars as passive structures. The limitation of this approach is that the self-supporting capacity of the overburden is not incorporated in pillar design. This suspension theory of pillar design treats the strata-pillar interaction problem as a classic case of static equilibrium, without detailing the interactions of the two structures.

Globally, multiple pillar design methods have been developed, based on this suspension theory. Each of these methods approaches the calculation of pillar stability a little differently with respect to material properties, excavation geometries and stress conditions. Most of these design methods are derived empirically and lack a mechanics-based approach. Moreover, there is a lack of a unified pillar design methodology that can be used to design all types of mine pillars using a mechanics-based approach.

The Ground Reaction Curve has been used as a means of correlating strata displacements to stress conditions. In addition, the Support Reaction Curve has been used in modeling the response of a support system under load, as a function of support properties and installation time with respect to opening development. In comparing the Ground Reaction Curves and Support Reaction Curves for different support systems, one can evaluate the effectiveness of installed support systems in maintaining the integrity of the excavated area(s).
This approach has been widely used in designing secondary (artificial) support systems in both civil tunneling and the mining industry. Encouraged by the successful use of this single method in designing secondary support systems, this research revisits this concept for mine pillar design. This research investigates the utilization of the Ground Reaction Curve and Support Reaction Curve for the design of mine pillar support systems with respect to anticipated pillar loading and opening convergence. In addition, a conceptual three-tier solution to the pillar design problem, using a proper combination of numerical, analytical and data-driven methods is suggested, and a flowchart for the pillar design methodology is proposed. At the focus of this proposed method lies the Ground Reaction Curve (GRC) Concept. This research effort tries to verify the proposed pillar design flowchart using in-mine instrumentation and numerical modeling.

For the purpose of this research, a deep longwall coalmine is instrumented to measure changes in pillar stress and associated roof convergence, due to mining activity. Subsequently, numerical models were developed in FLAC3D to model the geomechanical effects of underground longwall mining. The numerical modeling results are validated and calibrated using instrumentation data and a surface subsidence profile. The calibrated numerical models are further used to generate the Ground Reaction Curve for the overburden and Support Reaction Curve for the coal pillar. The comparison of both curves gives a detailed view of the overburden stability with respect to the mine pillar loading, in a more mechanics-based sense. The developed numerical approach can be used in future research and further development of this methodology for various mine types and different pillar support systems.

Keywords: Mine Pillar design, Longwall chain pillars, Pressure arching, Ground Reaction Curve (GRC), Mine Instrumentation, Numerical Modeling in FLAC3D.
INVESTIGATION INTO MINE PILLAR DESIGN AND GLOBAL STABILITY USING THE GROUND REACTION CURVE CONCEPT

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DEDICATION

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1 INTRODUCTION

1.1 Overview

The design of mine structures is typically based on experience and empirical guidelines. This is true because of the following reasons: (1) the post-mining redistribution of stresses have not been fully quantified, (2) the concept of in-situ stresses has not been expressed clearly, and (3) the varying effects of complex geologic conditions have not been quantified. Therefore, to ensure the stability of a mine structure, designed using empirical methods, it is necessary to apply a safety factor of such a magnitude that the original method of solution becomes of questionable value (Caudle and Clark, 1955).

The above statements though stated over 60 years ago, still have a significant amount of validity. Current pillar design methods use a classical pillar strength formula divided by overburden loading to provide a safety factor against core pillar failure (Reed, Mctyer, and Frith, 2017). These methods are statistically derived and treat the pillar as a passive structure, designed to carry the fixed overburden dead weight. This dead weight is often calculated by simple rules based on the geometry of the excavation: like the “tributary area theory”, the “abutment angle theory”, and the “pressure arch theory”. However, the actual mechanics of the overburden and the effects of the pillar system on the overburden response are not included in these empirical methods. The geo-mechanical effects of varying geology and that of in-situ stress anisotropy are also not accounted for in the current pillar design methods. Moreover, there is a lack of a unified approach towards mine pillar design for different mine types.

In contrast, a support design methodology incorporating a broader mechanics-based approach to the strata-support interaction problem has been developed by the civil tunneling industry. There are certain similarities as well as differences between a tunnel opening and an underground mine excavation. It is important to focus on these similarities
(while keeping in mind the differences) and learn from this interdisciplinary understanding. The GRC concept is related to one of the tunnel support design approaches, developed in the ‘pre-numerical’ times, but is still in use today: The Convergence Confinement Method (CCM) or the New Austrian Tunneling Method (NATM). The CCM or NATM is based on a simple mechanics-based approach that the support load required to stabilize the excavation is considered to vary with inward convergence of the rockmass. Calculation of the required support pressure is a statically indeterminate problem and is assessed by examination of the strata-support interaction as described by the GRC concept (Fairhurst and Carranza-Torres, 2002). In summary, the principal behind NATM is formulated mathematically by a closed-form analytical solution using the CCM, while the GRC concept is a simple graphical representation of the CCM solution.

Since its first mention in 1961, the GRC concept has experienced many variations and developments. Most of these developments were in the field of (artificial) support systems in tunnels. This method also found application, though to a lesser extent, in designing natural support systems (Gill, Leite, and Labrie, 1994). In the mining industry, this method is used in designing artificial support systems like secondary supports in longwall gateroad and longwall shield supports (Mucho, Barczak, Dolinar, Bower, and Bryja, 1999; Barczak, Esterhuizen, and Dolinar, 2005; Medhurst and Reed, 2005). Esterhuizen, Mark, and Murphy, (2010b); and Damjanac, Pierce, and Board, (2014) used the GRC concept to investigate strata-pillar interactions using numerical analysis in FLAC3D.

The GRC concept, a tested and proven method in artificial system design, has still to gain relevance in mine pillar (natural support system) design. This research investigates the mechanics-based relation between pillar load and overburden strata convergence response by correlating it to the GRC concept. The overall objective is to propose an extension of this approach for mine pillar design based on this relationship and on the established methodology of using the GRC method in artificial support system design. A tested and
proven method of using a combination of numerical analysis, instrumentation data and analytical approach is recommended to realize the full potential of this proposed method.

1.2 Problem Statement

While the mining industry has immense knowledge and understanding of underground stress redistributions, ground behaviors, and strengths, our current methodologies utilized in the assessment of mine pillar stability do not adequately represent site-specific conditions or known mechanics-based behaviors of overburden and support materials. Since the early 1960’s, the mining industry has evaluated the structural stability of a given pillar by dividing an approximated pillar load by an approximated pillar strength. While this design methodology has served the industry well in the past, as modern mining operations continue to develop in more complex geological and geometric conditions coupled with market economics, industry professional are in dire need of a modified stability assessment approach which considers both the stress conditions and material displacements.

Depending on the mine type, available technology, geology, and geographic locations, many pillar design methods have evolved. Each of these methods have certain advantages and limitations. Nevertheless, the user is often puzzled about the proper method selection, and the proper sequence of using them. This exercise becomes more challenging when different methods are combined, where, for example, one has to estimate the pillar strength based on empirical equations (e.g., Mark-Bieniawski coal pillar strength equation), determine the safety (or stability) factor based on an analytical approach (e.g., tributary area load divided by pillar strength) and analyze pillar-strata interactions using numerical analysis. Challenges increase when current pillar design methods are designed as passive structures to balance the fixed dead weight of the overburden. This classic suspension design approach does not recognize the self-stabilizing capacity of the overburden (Frith and Reed, 2018). These pillar design methods have served the purpose for a safer design
philosophy. However, the question remains: can there be a unified and structured pillar design approach that can be used for all cases?

1.3 Scope of study

Within the civil tunneling industry, the GRC concept has been successfully utilized in designing underground support systems that enhance the stability of an underground excavation. Through the incorporation of current industry knowledge and understanding of ground and support behaviors, this research looks to investigate the development of a modernized pillar design methodology to further improve the analysis of global underground stability.

Building upon the mining industry’s current understanding of ground control, a modernized pillar design approach would provide the ability to evaluate the stability of the mining environment with respect to pillar, ground support, and rockmass behaviors. Thus, the proposed pillar design methodology will provide the mining industry with a more comprehensive means of evaluating the stability of the underground working environment with respect to industry understanding of ground behaviors and stress mechanisms. The result of this work will provide an all-encompassing pillar and ground stability analysis methodology which takes into consideration both the material stress condition and displacement as well as interactions between the seam and the surrounding overburden strata. Properly implementing the proposed analysis approach can provide insights into pillar and ground support optimizations, while further increasing the health and safety of the underground working environment.

The research performed includes the following tasks:

1. Review of current mine pillar design methodologies and determining their limitations.
2. Review of different support design approaches to find an improvement for pillar design methods.

3. Develop a new pillar design methodology, which includes the following:

   a. A quantitative evaluation of the GRC concept for mine pillar design. This concept is also compared with other available methods.

   b. Development of a flowchart for pillar design methodology using the GRC concept based on literature review. This flowchart uses a proper mix of numerical modeling, instrumentation data and the GRC Concept. This research follows this flowchart to prove this methodology for mine pillar design.

   c. Instrumentation of a deep longwall coalmine in Virginia in order to monitor the geomechanical effects of longwall mining on the abutment pillar and adjacent roof. Instrumentation included two borehole pressure cells (BPC) and six roof extensometers.

   d. Numerical simulations to validate and calibrate the numerical models in FLAC3D. Numerical simulations included pillar-scale models using the Hoek-Brown constitutive model and the Coal Mass Model (developed by NIOSH), to simulate the correct geomechanical behavior of coal material. The simulation technique is validated using data published in past research papers. The model is calibrated using the in-mine instrumentation data. The panel scale model is simulated to model the correct geomechanical behavior of the overburden material. The simulation technique is validated using data published in past research papers. The model is calibrated using a subsidence profile obtained from Surface Deformation Prediction System (SDPS).
e. Development of Support Reaction Curves for the coal pillar determined numerically from the calibrated pillar scale models.

f. Development of Ground Reaction Curves for the overburden determined numerically from the calibrated panel scale model.

4. The Support Reaction Curves and the Ground Reaction Curves are super-imposed to determine the strata-pillar interaction. This result is further compared with that of established ground control software, ACPS.

5. Recommendations for future research work are made based on the findings of this study.

1.4 Dissertation Outline

The first chapter in this dissertation gives an overview of the limitations of the current pillar design methodologies. It highlights the fact that current pillar design methods lack a mechanics-based approach to explain the strata-pillar interaction mechanism. It then gives a brief description of the proposed study. The chapter also presents the outline of this research.

Chapter 2 presents a literature review of the topics relevant to this field of research. It provides a detailed background about the current mine pillar design methodologies. It describes the mechanics-based support design methods developed in civil tunneling industry. It gives a brief description of the evolution of GRC concept and its constituents. It discusses the use of GRC concept in secondary support design methods in both mining and civil tunneling industry. Furthermore, it provides the technical details of previous attempts to using the GRC concept in mine pillar design.

Chapter 3 provides a quantitative evaluation of using the GRC concept in mine pillar design. It discusses the benefits of using the GRC concept for support design. It describes
the three different schools of thought, used for mine pillar design, and their relationships. A pillar design flowchart is proposed using a combination of numerical analysis, instrumentation data and analytical approach to realize the full potential of GRC concept.

Chapter 4 discusses the process of in-mine instrumentation of a deep longwall coalmine in Virginia. It depicts the entire process from devising an instrumentation plan, to the final process of installing the instruments. It presents the dimensions of the instrumentation layout in the mine. The chapter highlights the technical specifications of the instruments used. Moreover, the final data obtained from the coalmine measurements are also presented.

Chapter 5 describes the process of numerical modeling in FLAC3D, as used for this research. It presents the process of creating validated pillar scale models using Hoek-Brown constitutive model and the Coal Mass Model, to simulate coal material. Then it presents the method of calibrating the pillar scale model using in-mine instrumentation data. It also describes the process of creating validated panel scale model in detail. Then it presents the method of calibrating the panel scale model using the subsidence profile obtained from SDPS.

Chapter 6 presents the results and conclusions of this research. It describes the process to numerically determine the Support Reaction Curves from the calibrated pillar scale models. It then describes the process to numerically estimate the Ground Reaction Curves from the calibrated panel scale models. In addition, it discusses the process of comparing the Ground Reaction Curves and the Support Reaction Curve, to determine the mechanics-based interaction between the pillar and the overburden. Finally, the chapter discusses the comparison of the results obtained from GRC concept with that of widely used ground control software, ACPS.
Chapter 7 presents a summary of the research and the resulting conclusions. It also presents the author’s recommendations for future work in this field of research.
2 LITERATURE REVIEW

2.1 Overview of Current Pillar Design Methods

Current pillar design methods use a two-pronged approach to solve the problem. The first part of the problem is to determine the strength of the mine pillars. The second part is to determine the load applied on the mine pillars. The stability factor, for a given pillar design, is obtained by dividing the strength of the pillar to the load applied on that pillar.

2.1.1 Determining Pillar Strength

Globally, multiple pillar design methods have been utilized in the design of underground excavations. Each of these design methods approaches the calculation of pillar stability a little differently with respect to material properties, underground geometries and stress conditions. Analytical solutions can be used to determine the strength of homogeneous materials, but not in the case of non-homogeneous and non-isotropic mineral deposits. These equations are often derived using empirical methods. Pillar strength is defined as the maximum resistance of a pillar to axial loading (Brady and Brown, 1985). The strength of mine pillars is typically estimated through indirect methods. In the past, researchers have correlated the strength of a given size and width of a rock specimen to the strength of a mine pillar. Bauschiger, (1876) did the first study on the effect of geometry on rock strength on Swiss sandstone samples. Since then several empirical equations to determine pillar strength had been published by many researchers like Bunting (1911); Griffith and Conner (1912); Greenwald, Howarth, and Hartmann (1939); Holland and Gaddy (1957); Evana, Pomeroy, and Berenbaum (1961); and Holland (1964).

After the Coalbrook disaster in 1960, Salamon and Munro (1967) proposed the following equation to determine the strength of coal pillars in South Africa. Where, \( S_p \) is the pillar strength, \( H \) is the pillar height, and \( W \) is the pillar width. The equation is calibrated based
on data from 125 cases of room and pillar mining of stable and unstable cases and solved for the values of $k$, $\alpha$ and $\beta$. Salamon and Munro, (1967) suggested the values of $k = 7.176$ (MPa), $\alpha = -0.66$ and $\beta = 0.46$ and showed that 99% of failures occur for a factor of safety (stability factor) of 1.5.

$$S_p = kH^\alpha W^\beta$$

There had been 31 pillar collapses in Australia, recorded between 1966 to 1988, using the Salamon and Munro equation (Madden, 1991). Thereafter a combined database from South Africa and Australia was statistically analyzed using the maximum likelihood method. This combined database was best fit using the above equation to give new values of $k = 6.88$, $\alpha = -0.7$ and $\beta = 0.5$ (Galvin, Hebblewhite, and Salamon, 1999).

Bieniawski believed that removing the coal sample from site leads to degradation of the specimen, thus large scale in situ testing of coal samples was performed, and the following empirical pillar strength equation was derived (Bieniawski, 1968). Where, $S_p$ is the pillar strength in psi units.

$$S_p = 400 + 200\left(\frac{W}{H}\right)$$

Bieniawski performed a detailed study of 200 surveyed case studies of coalmines in the United States and led to the following empirical equation for room and pillar coalmines (Bieniawski, 1983).

$$S_p = S_c \left[0.64 + 0.36 \left(\frac{W}{H}\right)\right]$$

Where $S_p$ is the pillar strength; $S_c$ is the in-situ coal strength; $H$ is the pillar height; $W$ is the pillar width.
The above equation is valid for square pillars. This equation was later improved by including the length of the pillar and validated from a large database of retreat mining case histories, and renamed as the Mark-Bieniawski pillar strength equation with the launch of ARMPS software (Mark and Chase, 1997). This equation is still widely used in the U.S. and worldwide. The original ARMPS database consisted of approximately 150 case studies. This equation was also introduced in the AMSS (Mark, Chase, and Pappas, 2007), ALPS (Mark, 1990) and ACPS (Mark and Agioutantis, 2018) software packages.

\[ S_p = S_c \left[ 0.64 + \left( 0.54 \frac{W}{H} \right) - \left( 0.18 \frac{W^2}{HL} \right) \right] \]

Where \( S_p \) is the pillar strength; \( S_c \) is the in-situ coal strength; \( H \) is the pillar height; \( W \) is the pillar width and \( L \) is the pillar length.

2.1.2 Determining Load on Pillars

The first empirical method for coal pillar design, with a factor of safety (stability factor) of 2.5, was proposed by Bunting (1911). Bunting, (1911) assumed that the overburden stress is equally distributed among all the pillars and thus the vertical load is divided over pillar area. Salamon, (1970) also suggested a similar approach to determine pillar load in case of regular pillar pattern and a large mining area. This approach of determining the pillar stress based on overburden stress is still widely used, even after 100 years of its introduction, and is known as the “tributary area theory”.

The tributary area method assumes that the weight of the overburden is equally distributed among the pillars, and that the overburden above a pillar is completely separated from the rest of the overburden strata (Figure 1). This method does not take into account the presence of barrier pillars, which can reduce the overburden load on the production pillars. It also does not account for the self-stabilizing capacity of the overburden. To estimate the
development loads, ARMPS software utilizes the “tributary area theory”. As shown in Figure 2, the loads transferred during the various post-development stages of mining is calculated using the “Abutment angle theory” (Mark, 2010).

Figure 1 Tributary area loading model for development mining (Mark, 2010).
The validity of the tributary area method was tested in shallow mines in South Africa by comparing it with the in-situ measurements of pillar stress at the middle of the panel. It was found that the tributary area method overestimates the stress by about 10% (Oravecz, 1977). Hill, Canbulat, Thomas, and Wijk, (2008) showed that the tributary area method overestimates the load on pillars (in Australian coalmines), by more than twice, since it does not account for the rock mass characteristics in the overburden strata.

The tributary area method also could not accurately determine the loads on pillars in deep mines. Therefore, in case of deep mines the pressure arch loading method is used. The concept of pressure arch loading was first observed in the coalmines of U.K. and popularized in the U.S.A. by Prof. C.T. Holland. As shown in Figure 3, the load above the pressure arch is transferred to abutment or barrier pillars and the load within the pressure arch is carried by the development pillars (Holland, 1973). In case of deep mines, the calculated tributary area is multiplied by an arch factor ($F_{pa}$) to get a more realistic stress
value. Pressure arch is used in mines with depth of cover \((H)\) more than panel width \((P_w)\). The arch factor determined by (Mark, 2010) is as follows.

\[
F_{pa} = 1 - 0.28 \ln \left( \frac{H}{P_w} \right)
\]

Figure 3 Implementation of pressure arch loadings in ARMPS. Transfer of pressure arch loads from the production pillars to the barrier pillars (Mark, 2010).

2.2 Overview of Support Design Methods in the Civil Tunneling Industry

The civil tunneling industry has successfully developed the following methods by incorporating a broader mechanics-based approach to the strata-support interaction problem, for underground support design.
2.2.1 The New Austrian Tunneling Method (NATM)

The New Austrian Tunneling Method (NATM) was developed by the work of Ladislaus von Rabcewicz, Leopold Muller and Franz Pacher and colleagues involved in the driving of transportation tunnels through the Austrian Alps between 1957 and 1965. This method takes advantage of the inherent geological strength available in the surrounding rock mass to stabilize the excavation. The support load required to stabilize the excavation is varied with the inward convergence of the excavation. The contribution of the method was that it developed a practical tunneling support technique based on the principles of convergence-confinement method, which allowed the support to be optimized on time (Fairhurst and Carranza-Torres, 2002).

2.2.2 The Convergence Confinement Method

The convergence confinement method, also known as the “characteristic lines method”, is known due to its simplicity and versatility. Since it was first mentioned (Lauffer and Sebber, 1961), the method has evolved into many variations and is mainly used in the civil tunneling industry. As shown in Figure 4, this method entails building an interaction diagram, comprising of the ground reaction curve (GRC) and the support reaction curve (SRC). The ground reaction curve, also known as the ground characteristic curve, describes the reaction of the excavation walls to the support pressure. The second curve describes the reaction of the support to the rock mass pressure. The interaction of the two curves gives the convergence confinement equilibrium conditions (Gill, Leite, and Labrie, 1994). This method assumes that the support load required to stabilize the excavation decreases as the tunnel wall converges. The fundamental concept of this method is that the intersection of the GRC and SRC gives the support pressure and tunnel deformation values at equilibrium. In summary, the principal behind NATM is formulated mathematically by a closed-form analytical solution using the CCM, while the GRC concept is a simple graphical representation of the CCM solution.
2.3 Ground Reaction Curve (GRC) and Support Reaction Curve (SRC)

Brown, Bray, Ladanyi, and Hoek were the first authors to explain the concept of the ground reaction curve. They proposed closed-form solutions for calculating the ground response curves, which helped improve the understanding of the rock support interaction. As shown in Figure 5, the analytical solutions were presented for a circular excavation, assuming plane strain conditions and under a hydrostatic in-situ stress field.
The GRC is represented by plotting the internal support pressure against the excavation convergence as shown in Figure 6. If the applied support pressure is equal to the stress in the surrounding rocks, no convergence takes place (point A). As the support pressure is reduced, the excavation converges inwards. Initially the rock mass converges in an elastic manner, thus a linear response. If the pressure is further reduced, the convergence is non-linear and self-supporting capacity of the rocks is lost (point B). A point is reached where the support resistance must increase to establish equilibrium capacity (point C). Then the
rocks lose all its strength and its dead weight must be supported (point D). The line PQR represents the response of the support system (Esterhuizen, Mark, and Murphy, 2010b).

Figure 6 Ground Reaction Curve (GRC) and Support Reaction Curve (SRC) (Esterhuizen, Mark, and Murphy, 2010b)
2.4 Past Research on the Application of the GRC Concept in Secondary Support Design

Roof support technology for longwall tailgates have changed dramatically over the past few decades. There are many secondary support systems, which have replaced the conventional wood and concrete cribs. With so many options to consider and the objective to attain adequate ground control at minimal cost, the trial and error approach to gate road support has to be changed (Mucho, Barczak, and Dolinar, 1999). A scientific and mechanics-based design methodology for the secondary tailgate supports was needed. The key to accomplish this task was to understand the strata-support interaction mechanism. The strata-support interaction was conceptualized by the use of GRC concept, which relates the secondary support resistance to the convergence of the longwall tailgate (Barczak, 2003). This technique requires in-mine measurements of tailgate support loading and roof convergence to establish a tailgate ground response behavior based on support and strata interaction.

Barczak, Chan, and Bower, (2003) used the GRC concept to optimize the usage of pumpable cribs in longwall tailgates. As shown in Figure 7, the support response curves for the pumpable cribs were obtained by laboratory testing and underground mine instrumentation. It was concluded that the key to optimizing the support utilization is to provide sufficient load density to prevent convergence from occurring beyond the peak loading capacity.
Figure 7 Loading profile for can supports (Barczak, 2003).

Mucho, Barczak, and Dolinar, (1999), and Barczak, (2003) have explained a simple in-mine measurements technique to determine the ground reaction curve for a longwall gateroad opening. The load and convergence data for different secondary supports or different arrangements of the same support systems, while installed in a gateroad, are to be measured. Each of these support type provides a single point on the ground reaction curve. The curve was then obtained using interpolation between the points (Figure 8).
The support reaction curve was obtained from testing the standing supports, to obtain the load-displacement curve, in the Mine Roof Simulator platform (Figure 9) at NIOSH. A database of the support reaction curves for various types of secondary support systems is available in the NIOSH STOP (2010) software program to guide in secondary support system design.
The Support Technology Optimization Program (STOP) tool was developed by NIOSH to facilitate in the optimization of tailgate support selection and utilization. This program can be used to obtain ground reaction data and develop design criteria for standing roof supports using the ground reaction curve developed from this data. STOP can then be used to evaluate different support products and optimize their use, based on the ground reaction criteria for a particular longwall tailgate (Barczak, 2003). This program also contains a database of the support reaction curves and loading profiles obtained by laboratory testing at the NIOSH Safety Structures Testing Laboratory (Figure 10). STOP was developed to provide a more comprehensive support design program, than that provided by the old Wood Crib Performance Model (Barczak, 2000). At the time this dissertation was defended the official STOP version on the NIOSH website was that published in 2010. However, NIOSH is planning to release an update of the STOP program with additional support models and GRCs in 2020.
Figure 10 Example of the loading profile available in STOP (Barczak, 2000).

Barczak, Esterhuizen, and Dolinar, (2005) used the ground reaction curve and numerical modeling to evaluate the impacts of standing support on the ground response (Figure 11). Numerical modeling in LaModel was used to evaluate the impact of standing support on the main roof and floor behavior and pillar yielding. Numerical modeling in FLAC was used to evaluate the near-seam roof and floor behavior, while taking into account the global in-situ stress conditions.
Figure 11 Loading characteristics of standing support systems based on full-scale testing in NIOSH Mine Roof Simulator (Barczak, Esterhuizen, and Dolinar, 2005).

Esterhuizen and Barczak, (2006) used numerical modeling in FLAC to supplement the mine instrumentation results by simulating longwall tailgate and the associated ground response. The ground response curve was developed by simulating different internal support pressures and recording the resulting convergence as shown in Figure 12 and Figure 13.
Figure 12 GRC derived from tailgate model under strong roof conditions: A is a stiff support; B is a soft support (Esterhuizen and Barczak, 2006).
Figure 13 GRC derived from tailgate model under weak roof conditions: A is a stiff support; B is a soft support (Esterhuizen and Barczak, 2006).

Medhurst and Reed (2005) used the GRC concept to provide a graphical representation of the longwall shield support and strata-support interaction mechanism (Figure 14). This approach was used to assess longwall support performance, and design support systems in many Australian mines. It was further concluded that one of the major advantages of the GRC concept is to graphically represent the typical longwall support response from real operating data.
Prusek, Plonka, and Walentek, (2016) mentioned that in recent years, numerical modeling and the ground reaction curve have been used to determine adequate longwall shield capacity.

2.5 Past Research on the Application of the GRC Concept in Mine Pillar Design

Starfield and Fairhurst, (1968) and Starfield and Wawersik, (1972), postulated the use of the convergence-confinement technique to design natural (pillar) support systems. They suggested that this method could be used to determine the mine stiffness coefficient and pillar stiffness coefficient, also known as the local mine-stiffness concept.
Salamon (1970), first applied the GRC concept to pillar design. Salamon (1970) discussed the analogy of the stability of a laboratory compression test of a brittle rock specimen in a loading machine to that of a mine pillar and the surrounding strata stressed under the overburden load (Figure 15). He derived an analytical relation between the stiffness of the mining layout to the load-convergence characteristics of the pillar, also known as the Salamon’s stability criterion, as shown in Figure 16.

Figure 15 Idealized loading system (a) with the illustration of the equilibrium between load and rock resistance (b) (Salamon, 1970).
Sarkka (1984) derived the ground reaction curves using numerical stress analysis codes and proposed an interactive method for the design of mine pillars. Gill, Leite, and Labrie, (1994) used the convergence-confinement method to predict the axial stresses in a pillar of...
uniform and non-uniform multiple pillar arrays. As the number of pillars increase, the graphical solution is no longer feasible for higher number of pillars, thus an algorithm was suggested to make use of the pillar reaction curve for predicting the axial stress acting on each pillar. The mathematical model describing the pillar reaction curve considers the fact that mine pillars are pre-stressed rock masses and the ground reaction curve was obtained using a boundary element numerical code.

Esterhuizen, Mark, and Murphy (2010b) used numerical models to examine the interaction between coal pillar systems and the host rock for weak and strong geologic conditions at various spans and depths of cover. The results showed that the span-to-depth ratio was an important factor in determining the ultimate stress and deformation in the pillars. Based on this study, the span-to-depth ratio was added as an input parameter in the updated ARMPS-2010 software.

Damjanac, Pierce, and Board (2014) used the ground reaction curve approach for examining the potential for collapse of room-and-pillar trona mine. Support Reaction Curves for the trona pillars were generated for various extraction ratios. Ground Reaction Curves was generated for various panel and barrier pillar widths. Damjanac, Pierce, and Board (2014) superimposed the curves and analyzed that the collapse was the effect of high extraction ratio combined with a shear failure of the massive overburden bed, which could have applied the full overburden loading on the panel geometry.
3 DEVELOPING A NEW PILLAR DESIGN METHODOLOGY

Previous research has shown that current pillar design methodologies are mostly empirical in nature and lack a mechanics-based approach. Therefore, the focus of this research is to develop a new pillar design methodology that uses the latest available technology and incorporates the strata-pillar interaction mechanism. Based on literature review and preliminary studies, the research is narrowed down to the use of GRC Concept in mine pillar design. In addition, a pillar design flowchart is developed based on this preliminary study.

3.1 GRC Concept – A Mechanics-Based Approach

Application of the GRC concept to mine pillar design is analogous to designing permanent tunnel lining (Frith and Reed, 2018). In order to present the technical justification for the use of the GRC approach in mine pillar design, it is necessary to review the abilities of this, single, method in bridging the gap in present understanding of the strata-support interaction phenomena, as noted by Caudle and Clark (1955).

3.1.1 Quantifying the Redistribution of Stresses: Using the GRC Approach

In the case of any underground excavation, the in-situ stresses are disrupted and redistributed. The response of host rock to the creation of an underground mine opening, and the redistribution of stresses, determines the load applied to the support systems. The driver for any type of support failure is the direct response of this reaction from the surrounding rock mass (Esterhuizen, Mark, and Murphy, 2010b). The redistribution of stresses, after an excavation is created is often referred to as a “pressure arch” (Barrientos and Parker, 1975). The pressure arch concept is related to a fundamental rock mechanics hypothesis, i.e., the GRC concept (Mark, 2010).
The Ground Reaction Curve can be used to represent the rock mass response to mining and its effect on support systems (Esterhuizen and Barczak, 2006). It is a useful analysis tool, which helps in estimating the equilibrium after an excavation is created. One of the advantages of this method is that the result is not limited to a single equilibrium, but it allows verifying how the equilibrium of the excavation changes for any change in the stress/support conditions (Amberg, 2011). Esterhuizen, Mark, and Murphy (2010b) used the GRC concept to present and quantify the pressure arching effects as a function of the span of excavation. As shown in Figure 17, Esterhuizen, Mark, and Murphy (2010b) use numerical modeling in FLAC3D to obtain the Ground Reaction Curves, and prove that as the panel span decreases, the Ground Reaction Curves becomes stiffer and the arching effect is more pronounced, in case of a stronger overburden strata.

Figure 17 Ground Reaction Curves at the mid-span of panels under strong overburden at 450 meters depth of cover, with varying panel size (Esterhuizen, Mark, and Murphy 2010b).
From the above statements, it is understood that the failure or success of any support system depends on a proper understanding of the redistribution of stresses. It can, further, be suggested that the GRC concept is well appointed in explaining and quantifying the pressure arching mechanism, a well-established concept in current pillar design techniques (Mark, 2010).

3.1.2 Quantifying the In-situ Stresses: Using the GRC Approach

Underground coal mines in Eastern US experience maximum horizontal stresses three times that of overburden stresses and 40% more than the minimum horizontal stress (Mark, Mucho, and Dolinar, 1998). Field observations and stress measurements have indicated that the horizontal stresses in the limestone formations of Eastern and Midwestern US are often higher than the overburden stress (Iannacchione, Marshall, and Prosser, 2002). In order to prevent roof falls in a mine opening under high horizontal stresses, it is necessary to understand the ground response to high horizontal stresses and the corresponding roof support requirements (Zhang, Gearhart, Dyke, Su, Esterhuizen, and Tulu, 2018).

The Ground Reaction Curve can be very effective in analyzing the effect of stress anisotropy. Amberg, (2011) conceptually shows (Figure 18) the Ground Reaction Curves for a circular excavation in anisotropic in-situ stress conditions. This example shows the ability of the GRC approach in quantifying anisotropic in-situ stresses.

The failure or success of any support system depends on a proper understanding of pre-existing stress conditions and their interactions with the excavation. The GRC concept can quantify and explain the strata excavation interactions (both stabilizing and destabilizing) in case of high horizontal stresses.
3.1.3 Quantifying the geo-mechanical effects of geology: Using the GRC Approach

Given the variability and unpredictability of geological materials, it is necessary for a support design methodology to accurately quantify the change in geo-mechanical response of the rockmass. For all practical purposes, the geological conditions and the resultant mechanical behavior can be represented mathematically by the stiffness modulus of the rock mass. In case of strata-pillar interaction problem, the stresses and associated displacements in the host rock or in the pillar support system depends on the stiffness of the strata and that of the pillar support system (Brown, Bray, Ladanyi, and Hoek, 1983). The concept of pillar stiffness and strata stiffness is well established in the field of pillar
design and has been used to evaluate the potential for violent pillar collapse (Esterhuizen, Mark, and Murphy, 2010b).

Esterhuizen and Barczak, (2006) used FLAC modeling results to show the effects of mechanical properties of strata and their interfaces, on the Ground Reaction Curves. Esterhuizen, Mark, and Murphy, (2010b) used the Ground Reaction Curves to quantify the distribution of overburden loads between the strata and natural support system, as a function of their stiffness (Figure 19). Esterhuizen, Mark, and Murphy, (2010b) used numerical model analysis in FLAC3D to prove that in case of relatively soft strata, the full overburden weight is transferred to the pillar system, while in case of stiffer strata; a greater portion of the overburden load is transferred to the abutments by an arching mechanism.
Therefore, it can be summarized that a proper quantification of the geo-mechanical conditions is crucial to the design of any support system. It can also be argued that the GRC concept is well equipped in explaining the geo-mechanical effects of pillar and strata relative stiffness, a prominent concept in current pillar design methods.

3.2 GRC Concept – A Unified Design Methodology

There are three main design approaches with respect to the analysis of the stability of underground openings: (a) the numerical approach, (b) the analytical approach, and (c) the
data-driven approach. In simple terms, the numerical approach is defined as the method of discretizing the domain and obtaining a solution using numerical methods to solve governing equations based on energy or mechanical equilibria. The analytical approach is based on mathematical formulations derived from the understanding of mechanics; these formulations are typically based in simple geometries and can, therefore, be represented by closed form solutions. The data-driven approach involves obtaining data from instrumentation or observed results. These approaches are briefly described below:

(a) Numerical approach: Given the complexity and variability in geometry, material properties and in-situ conditions in an underground mine, a numerical analysis can be a feasible solution. In addition, numerical methods are based on Newtonian mechanics and hence a mechanics-based solution for the pillar design problem can be expected. Although there are a plenty of numerical modeling tools available, it is important to select the optimum tool for each modeling exercise. Moreover, the same numerical tool can produce different results, depending on the problem definition (e.g., degree of discretization). As the results obtained depend on the numerical tools and also on the modeler, it is important to use a yard stick for comparing the various results. Finally, the numerical modeling approach needs to be validated and the results calibrated.

(b) Analytical approach: Conservative methods like the tributary area method uses a fixed dead-weight loading approach. While these methods worked well in the past, they need to be replaced in the favor of safety and economics (Frith and Reed, 2018). On the other hand, the Convergence Confinement Method uses a more mechanics-based approach of approximating the overburden load as a function of support/strata deformation (Fairhurst and Carranza-Torres, 2002). The CCM can prove to be a better alternative, however, the closed form analytical solutions are typically available for simple geometries and stress conditions.
(c) Data-driven approach: When physical modeling is not feasible or possible and mathematical solutions cannot be derived, in situ measurements can be used to understand the underlying phenomena and the geo-mechanical behavior of the excavation. When a clear understanding cannot be derived, a set of experimental results can be used to obtain an empirical solution using statistical analysis (e.g., pillar strength). Availability of economical instruments and the improvement in measurement techniques have increased the use of in-mine instrumentation. However, the experimental approach has certain limitations. The complete phenomena are difficult to measure, and the data is often influenced by several external factors. Moreover, the results obtained for a given condition cannot be extrapolated to a different set of conditions. The same is true for empirical solutions (Mark, 1999).

As a general philosophy for mine pillar design, the broader techniques are illustrated using a Venn diagram as shown in Figure 20. The space is divided into the three most widely used techniques in mine pillar design: the numerical approach, the analytical approach and the experimental (or empirical) approach.

A large number of technical papers on mine pillar design are available in the literature. It is not possible to give an overview of all these papers; therefore, a Venn diagram is used to classify the broader techniques suggested in these papers. Pillar design has often been a two-way solution: determining the load acting on the pillars and estimating the strength of the pillars. Over the years, several schools of thought have developed to solve the pillar design problem. While there has been attempts to solve the problem using only one of the three approaches mentioned above, more often the complexity inherent in this problem warranted the use of a combination of these techniques (Ray, Agioutantis, and Kaklis 2019).

In one of the most widely used school of thoughts, the overburden load (e.g., tributary area loads adjusted by abutment and pressure arch loading) and the pillar strength (e.g., the
Mark-Bieniawski pillar strength equation) is determined using a case history database, but
the stability factor is calculated using a simple analytical approach of dividing the strength
with the load (Mark, 2010). Another school of thought being, the analytical approach of
CCM or GRC is used in conjunction with numerical analysis results (Gill, Leite, and
Labrie, 1994). The more recent school of thought uses experimental data to calibrate
numerical models (Gale, 1999; Esterhuizen, Mark, and Murphy, 2010a).

These three schools of thought, when represented on this Venn diagram often fall at the
intersection of two of the three broader techniques: i.e., the analytical and experimental
approach, the numerical and experimental approach, and the numerical and analytical
approach. Depending on the mine type, available technology, geology and geographic
locations, these schools of thought were developed and used. These methods have served
the purpose for a safer design philosophy; however, the question is: can there be a unified
approach that can be used for all cases (Ray, Agioutantis, and Kaklis, 2019).

Contrary to the pillar design methods, another school of thought for artificial support
design (e.g., tunnel lining, secondary support) has been using a combination of all the three
broader design techniques. This method has its origin in the development of mechanics-
based methods like CCM (GRC is the graphical representation of the closed form
mathematical solutions). It initially started with closed form analytical methods and later
incorporated the benefits of numerical modeling (to determine the GRC concept), often
calibrated to instrumentation data (Barczak, Esterhuizen, Ellenberger, and Zhang, 2008).
It can thus be argued that this method lies at the intersection of the three broader techniques
on the Venn Diagram and uses the benefits of all the three methods while eliminating their
individual limitations (Ray, Agioutantis, and Kaklis, 2019).

Based on the principles used for artificial support design, more recently a similar approach
for mine pillar design has been developed (Esterhuizen, Mark, and Murphy, 2010b;
Damjanac, Pierce, and Board, 2014). Numerical analysis in $FLAC3D$ was used to obtain
the Ground Reaction Curves and Support Reaction Curves. The numerical model was calibrated using experimental results (observation or instrumentation data). Proponents of this method have suggested that mine pillar design can be possible by using the GRC concept (Barczak, 2011).

Figure 20 Venn diagram for classifying three broader techniques used in pillar design methods (Ray, Agioutantis, and Kaklis, 2019).
Similar to current pillar design methods that deal with the pillar strength and the overburden load separately, the GRC concept also treats the pillar mechanics and overburden mechanics individually. The GRC concept is simple to understand and to use if its components are decoupled and investigated separately (Ray, Agioutantis, and Kaklis, 2019).

The stress displacement behavior of a support (in this case the mine pillar) is called the Support Characteristics or the Support Reaction Curve (SRC). The SRC is a function of pillar stiffness and post-failure behavior of the pillar, which in turn depends on several factors. Mine pillars are made up of different materials depending on the ore being mined (e.g., coal or limestone). Often in the same mine, pillars may vary in shape and size depending on the method of mining, such as longwall or bord and pillar. Mine pillar geometry also changes according to the sequence of excavation (e.g., retreat mining in coal or benching in limestone) and structural needs (e.g., yield pillars or abutment pillars). Pillar properties are further influenced by geologic disturbances like joints and intrusions. Therefore, pillar stiffness and its post-failure response is a function of different combination of variations in geometry, geology, and in-situ conditions. The SRC can be used to reflect the effect of these variations on the mechanical response of the mine pillar support system (Ray, Agioutantis, and Kaklis, 2019).

The Ground Reaction Curve (GRC) is the stress displacement response of the overburden for a given panel span and barrier pillar layout. The GRC expresses the unloading stiffness of the overburden and reflects the ability of the overburden strata to form a stable pressure arch over the panel (Damjanac, Pierce, and Board, 2014). The GRC is a function of in-situ stresses, overburden geology, and mining geometry. Therefore, the GRC can be used to express the mechanical response of the overburden as a function of in-situ stresses, geology, and geometry (Esterhuizen, Mark, and Murphy, 2010b).
The GRC and SRC illustrate the geo-mechanical behavior of the overburden and that of the pillar respectively (Figure 21). The point of intersection of the GRC and SRC gives the condition for static equilibrium between the ground movement and pillar resistance. This intersection of both the curves manifests the GRC concept. The equilibrium condition changes for any change in the GRC or the SRC, therefore reacting to any change in the overburden conditions or the pillar conditions. Thus, the GRC concept can be used to determine the conditions for stability in a mine, where both the overburden response and pillar conditions are subjected to change, over time, due to various mining activities (Ray, Agioutantis, and Kaklis, 2019).

Figure 21 GRC and SRC interaction diagram (Ray, Agioutantis, and Kaklis, 2019).
While the numerical models (saved state) obtained from different numerical analysis tools or that from different modelers is often difficult to correlate, the results in the form of simple stress-displacement graphs can be compared. The SRC for a given pillar condition (e.g., same material and geometry) and the GRC for a given strata condition (e.g., same coalfield and panel geometry) should be similar and are expected to match (Barczak, Esterhuizen, Ellenberger, and Zhang, 2008). Any deviation, however, from the ideal graph should be investigated in detail. Thus, the GRC method can be used as a yard stick to determine the validity of numerical modeling results (Ray, Agioutantis, and Kaklis, 2019).

Likewise, numerical modeling can be used to model the entire mining process for variable geometries, thus determining the GRC and SRC for the strata and support systems. While the complete curve for GRC and SRC is difficult to obtain using in-mine measurements, instrumentation data at a few points can be used to calibrate the numerical model and improve its validity (Barczak, Esterhuizen, Ellenberger, and Zhang, 2008). Therefore, it can be proposed that the three broader techniques, however diverse, can complement each other when used in sync (Ray, Agioutantis, and Kaklis, 2019).

3.3 Proposed Pillar Design Methodology

The proposed method is an extrapolation of the support design approach developed for secondary support systems (Barczak, 2011). A three-tier solution for mine pillar design using a combination of numerical, analytical and experimental approach is derived from the works of Esterhuizen, Mark, and Murphy (2010b) and Damjanac, Pierce, and Board (2014). This approach is based on the mechanics-based relationship between pillar load and strata convergence. A flowsheet for the proposed design methodology is shown in Figure 22.
Figure 22 Flowchart for proposed pillar design methodology (Ray, Agioutantis, and Kaklis, 2019).
The post-mining redistribution of stresses will manifest itself as the loading of mine pillars and corresponding strata convergence. Thus, the strata-pillar interaction can be explained by measuring pillar loading and mine convergence. This stress displacement measurement for the overburden and for the pillar supplies the input for the GRC and SRC, which in turn can be used in the GRC concept. Therefore, the GRC concept can be used to incorporate a mechanics-based approach to the strata-pillar interaction problem in a pillar design methodology (Ray, Newman, and Agioutantis, 2019).

Pillar load and strata convergence can be measured by simple instruments like the borehole pressure cells (BPC) and roof extensometers. Esterhuizen, Mark, and Murphy (2010b) used the ground response at the center of the extraction line in the case of a bord and pillar coal mine panel with uniform pillar dimensions. Damjanac, Pierce, and Board (2014) determined the GRC in the middle of a trona mine panel with varying pillar geometry. The underlying concept, however, is to measure the stress displacement relation for the panel and for the pillars at a location where the geomechanical effects are most pronounced.

The instrumentation data can be used to determine only a small section of the GRC and SRC curves (Barczak, 2006). However, this data can be used to calibrate the numerical model and hence obtain the full GRC and SRC curves (Barczak, Esterhuizen, and Dolinar, 2005).

The finite difference software \textit{FLAC3D} can be used to realistically model the mining process and the corresponding geo-mechanical response. It can be used to analyze the overburden and pillar behavior, thus providing the stress displacement curves: the GRC and SRC respectively (Esterhuizen, Mark, and Murphy, 2010b).

For the purpose of determining the SRC and GRC, the pillars and the panel are modeled separately since there is a difference in scale. While the details involved in pillar failure can be modeled using a pillar scale model, the details in the response of the overburden
can be obtained using a panel scale model (Damjanac, Pierce, and Board, 2014). In case of the panel scale model, the presence of pillars affects the stiffness of the overburden, therefore it is necessary to model the individual pillars in the panel scale model.

### 3.3.1 Preliminary Numerical Modeling

The first modeling exercise comprises of making a preliminary FLAC3D model of the mine panel, pillars and adjacent mining activity. At this stage of numerical modeling, the input parameters and material properties can be obtained from laboratory testing and/or from published data in the literature. The purpose of this model is to get a first-hand understanding of the stresses and displacements, at the location in the panel and of the pillar, the GRC and SRC for which have to be obtained. This preliminary understanding of the geomechanical effects will become a guiding factor in determining the type and capacity of instruments to be installed at the given location.

### 3.3.2 Modeling Pillars to determine the SRC

During the second modeling exercise, the pillar support response is analyzed using the pillar scale models (smaller discretization). Pillar scale models can be made for pillars with different shapes and sizes (Figure 23), and for pillars with different extraction ratios (Figure 24). Depending on pillar geometry, the models can be two-dimensional (elongated pillars) or three-dimensional (equidimensional pillars). Since, the immediate overburden and floor have an important effect in the strata-pillar interaction problem, a portion of the floor and roof should be included in the pillar-scale models (Damjanac, Pierce, and Board, 2014). Laboratory data and empirical classifications published in literature (for example see Mohamed, Tulu, and Murphy, 2016; and Esterhuizen, Bajpayee, Ellenberger, and Murphy, 2013) can be used in determining the model input parameters.
The models can then be subjected to vertical loads until the pillar fails completely. Different stress-strain response can be obtained as a function of pillar geometry, geologic material and extraction ratio (Figure 25). The numerical models can be calibrated to observed pillar behavior as well as instrumentation data for change in pillar stress and opening convergence (Damjanac, Pierce, and Board, 2014).

Figure 23 Representation of different pillar patterns for FLAC3D analysis to determine the SRC (Damjanac, Pierce, and Board, 2014).
Figure 24 Representation of plan view of pillars with different extraction ratios for FLAC3D analysis to determine the SRC (Damjanac, Pierce, and Board, 2014).
Figure 25 Resulting SRC for pillars with extraction ratios of 66% to 78% (Damjanac, Pierce, and Board, 2014).
3.3.3 Modeling Panels to determine the GRC

During the final modeling exercise, the panel-scale model can be used to determine the stress-displacement response of the overburden for a given panel geometry and geology. The GRC expresses the unloading stiffness of the overburden while reflecting the ability
of the overburden to form a pressure arch around the mined out opening. The discretization (element/zone size) for the panel can be larger than the pillar-scale model, but sufficient enough to represent the panel scale geomechanical phenomena. Depending on panel geometry, the models can be two-dimensional or three-dimensional. Material properties for the overburden (also of the gob) can be determined from published literature (for example see Esterhuizen, Bajpayee, Ellenberger, and Murphy, 2013; and Esterhuizen, Mark, and Murphy, 2010a).

The pillars may not be explicitly modeled. Each pillar can be represented in the panel scale model using the corresponding SRC generated from the pillar scale model (Damjanac, Pierce, and Board, 2014). The equivalent element method can be used to model the stress strain behavior of the pillars in the large-scale three-dimensional panel layout (Esterhuizen, Mark, and Murphy, 2010b).

Esterhuizen, Mark, and Murphy, (2010b) obtained the GRC by reducing the pillar stiffness simultaneously in all the remaining pillars in the panel in a stepwise manner and determining the pillar stress and associated convergence at the mid-span of the extraction line (Figure 27). However, Damjanac, Pierce, and Board, (2014) determined the GRC for the overburden by gradually reducing the average pressure applied to the roof and determining the corresponding roof displacement at the center of the panel. Moreover, it is possible to obtain the GRC for any pillar location in the panel using either of the methods.
Figure 27 Schematic diagram showing a partially extracted panel and the location of the pillar that was used to determine the GRC (Esterhuizen, Mark, and Murphy, 2010b).

Figure 28 Schematic illustration of model to determine overburden GRC, with pillars replaced by a uniform pressure (Damjanac, Pierce, and Board, 2014).
3.3.4 Plotting the GRC and SRC to assess the strata-pillar interaction

The strata-pillar interaction can be assessed by adding the individual stress-displacement (strain) curves for different pillars (based on geometry or extraction ratio) on the GRC chart (for different panel geometry and overburden conditions). The point of intersection for a given pair of GRC and SRC will denote the average equilibrium pillar stress and opening closure. It can also indicate if equilibrium is possible for a given pair of GRC and SRC. Esterhuizen, Mark, and Murphy, (2010b) used the GRC approach to demonstrate the effects of ground response on ultimate pillar loading and pillar deformation. The effects of span-to-depth ratio on the ground response was determined and therefore added as a parameter in the updated ARMPS-2010 (Mark, 2010). Esterhuizen, Mark, and Murphy, (2010b) proved that a review of pillar strains from the numerical models and stability factors calculated from ARMPS showed that a relationship exists between ultimate pillar strain and the likely success of retreat mining.
Figure 29 SRC for pillars with extraction ratios of 50 to 90% superimposed with the GRC for the given panel (Damjanac, Pierce, and Board, 2014).
3.3.5 Comparing results to established empirical methods

Current mine pillar designs methodologies have evolved since the last few decades. A number of empirical methods have been developed based on a large data set. Some of the most common tools used to determine pillar stability are the ground control tools like ALPS (Mark, 1990), ARMPS (Mark and Chase, 1997), AMSS (Mark, Chase, and Pappas, 2007),
ACPS (Mark and Agioutantis, 2018) and S-Pillar (Esterhuizen, Dolinar, Ellenberger, and Prosser, 2011).

While the GRC approach for pillar design is a relatively new method, the confidence in this method can be improved by comparing it to the results obtained from already established empirical methods.
4 MINE INSTRUMENTATION

The strata-pillar interaction can be quantified using the change in pillar stress and associated roof convergence. In order to get the physical data for this phenomenon, this research project also involved the instrumentation of an abutment pillar in a deep longwall mine.

4.1 Background Research about Coalmine Instrumentation

A thorough background research about coalmine instrumentation is performed, to help in designing the instrumentation pan. Technical papers published in the field of underground instrumentation (US and Global) were critically reviewed and the various instrumentation methods and technologies compared and evaluated to provide an optimum solution for the proposed underground coalmine instrumentation plan. Table 1 presents the comparison table for recent underground coalmine instrumentation from literature survey.
Table 1 Comparison table from literature survey on underground coalmine instrumentation.

<table>
<thead>
<tr>
<th>Paper</th>
<th>Stress indicator</th>
<th>Strain indicator</th>
<th>Data logger</th>
<th>Mine type</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Esterhuizen, Gearhart, Klemetti, Dougherty, and Dyke, 2018)</td>
<td>cable bolt/ roof bolt pressure cells</td>
<td>roof-extensometers</td>
<td>-----</td>
<td>Coal</td>
</tr>
<tr>
<td>(Gearhart, Esterhuizen, and Zhang, 2018)</td>
<td>4-cable bolt load cells</td>
<td>3-six-channel roof extensometers / 4-convergence measurements on crib / 1-multipoint borehole extensometer to measure the pillar expansion</td>
<td>Instrumentation connected to two multiplexers to a single Campbell Scientific permissible data logger</td>
<td>Coal</td>
</tr>
<tr>
<td>(Mirabile and Westman, 2018)</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>Coal</td>
</tr>
<tr>
<td>(Zhang, Gearhart, Dyke, Su, Esterhuizen, and Tulu, 2018)</td>
<td>4-cable bolt load cells to measure the load in the cable bolts / 6-borehole pressure cells to measure the stress change in the pillars</td>
<td>3-six-point roof extensometers to measure roof deformation / 4-convergence meters to measure crib convergence / 1-multipoint borehole extensometer to measure the pillar expansion</td>
<td>-----</td>
<td>Coal</td>
</tr>
<tr>
<td>(Pariseau, McCarter, and Wempen, 2018)</td>
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<td>-----</td>
<td>Coal</td>
</tr>
<tr>
<td>(Lu and Hasenfus, 2018)</td>
<td>Borehole Pressure Cell</td>
<td>tell-tale</td>
<td>-----</td>
<td>Coal</td>
</tr>
<tr>
<td>(Scovazzo, 2018)</td>
<td>borehole pressure cells</td>
<td>Extensometers / convergence rod.</td>
<td>-----</td>
<td>Coal</td>
</tr>
<tr>
<td>(Tulu, Esterhuizen, Gearhart, Klemetti, Mohammed, and Su, 2017)</td>
<td>3-hollow inclusion (HI) cells installed over the panel 3-installed over the pillar</td>
<td>-----</td>
<td>-----</td>
<td>Coal</td>
</tr>
<tr>
<td>(Ted, Mark, and Tulu, 2017)</td>
<td>instrumented pumpable cribs with load cells (Jack packs)</td>
<td>instrumented pumpable cribs with convergence monitors (string pots)</td>
<td>-----</td>
<td>Coal</td>
</tr>
<tr>
<td>(Gearhart, Esterhuizen, and Tulu, 2017)</td>
<td>load cells attached to cable bolts / hydraulic load cell on cribs / borehole pressure cells / hollow inclusion cells</td>
<td>Eight-foot-long extensometers / Twenty-foot-long extensometers / wire pull transducer on cribs / multipoint borehole extensometer</td>
<td>MIDAS permissible data logger</td>
<td>Coal</td>
</tr>
</tbody>
</table>
4.2 Instrumentation Plan

4.2.1 Investigating the relation between Pillar Load and Opening Convergence

The relation between secondary support loading and the corresponding roof convergence is utilized in obtaining the Ground Reaction Curve for a longwall gateroad. Before generalizing this philosophy to all support systems, it is necessary to understand the underlying principles of strata mechanics.

A fundamental concept in strata mechanics is that the ground tends to move from a region of high stress to low stress, just like air or water moves from high pressure to low pressure area (Figure 31). A zone of low stress is created, due to the loss of confinement in an excavation. The ground tends to close the opening, and this closure process continues until the stresses have redistributed to the surrounding strata and support systems, and an equilibrium is attained. Because of different material properties and stress conditions, this equilibrium process is often more complex than is explained here. Regardless of what the exact mechanism is and how the equilibrium is attained, this phenomenon will manifest itself as a change in support loading and opening convergence. Thus, it is can be possible to explain this phenomenon of strata-support interaction by taking measurements of support loading and the associated mine convergence. Fundamentally, this process embodies the measurement of the Ground Reaction Curve (Barczak, 2003).
Figure 31 Conceptual model of strata mechanics (Barczak, 2003).

A simple conceptual model of the redistribution of overburden load and overburden strata convergence is used to explain the relation between pillar load and opening convergence. For the sake of simplicity elastic conditions are assumed and it is also assumed that the mine floor is very strong and hence undisturbed. During the pre-mining conditions the stress path is vertical (assuming no horizontal stresses) and is equally distributed over the mining horizon (Figure 32). As a result of mining, there is a loss of confinement in the excavation and the overburden strata sags downwards, therefore compressing the mine pillar. The compression induced in the mine pillar, causes it to resist the overburden strata movement.
This cyclic process of strata convergence and mine pillar strength mobilization continues until a stress equilibrium is attained. Thus, the in-situ stresses redistribute to the surrounding strata and mine pillar (Figure 33). This phenomenon of strata-pillar interaction and attainment of a static equilibrium can be represented graphically using the GRC concept (Figure 6). Conversely, physical measurements of pillar load and overburden strata convergence can be used, at least conceptually, to obtain the GRC and SRC curves.
4.2.2 Instrumenting a Longwall Coalmine

Installation at the underground coal mine in Virginia was completed on 01/03/19 and 01/04/19. The instruments were installed in the gateroad between longwall panels, at a depth of around 2000 feet. The longwall panels are 700 feet wide and 10,000 feet long. This mine used a yield-abutment-yield gateroad pillar system. The abutment pillars are 430 feet long, 150 feet wide, and 10 feet high. The yield pillars are 150 feet long, 40 feet wide, and 10 feet high. The gateroads were around 10 feet high and 25 feet wide. Two rows (for the purpose of redundancy) of instruments (1 BPC and 3 extensometers in each row) were installed near crosscut # 9, between longwall Panel 5East and Panel 6East of Area H. The instruments were installed in the stable (abutment) pillar of the head gate area of Panel 5East near crosscut # 9. Longwall mining of Panel 5East was finished on 25th of February
2019, and the first wave of abutment pressure recorded. After Panel 5East was mined, the instrumentation location became a part of the gob and hence inaccessible. The data logger was located inwards from the recovery room at crosscut #3 such that it was accessible to retrieve data. However, the Datalogger that was installed cannot be used when the instruments were inside the gob, since it was not a permissible Datalogger. Therefore, mine officials disconnected the Datalogger on 25th of February 2019. The data collected, until date, was retrieved by the research team on 31st of January 2019. A single data point was obtained on 31st of January 2019, when the instruments were brought online to collect data.

Meanwhile a permissible data logger was borrowed from NIOSH for the remaining period of data acquisition. The ground control team from NIOSH, connected a permissible data logger on 12th of March 2019. It was expected that the Borehole Pressure Cells, roof extensometers and connected wirings should remain intact to register the second abutment load from longwall mining of Panel 6East. The ground control team from NIOSH, disconnected the permissible data logger on 29th of April 2019 and also collected the data.

The extensometers are installed in the roof of the entry (one in the middle of entry, one near to the pillar and one in between the other two) to monitor the full displacement profile of the entry.

Figure 34 and Figure 35 show the instrumentation on a mine map. The Instrumentation is located near crosscut #9 and the data logger and the multiplexer are located at crosscut #3. Figure 36 shows the approximate location of the longwall face with respect to the instrumentation location.
Figure 34 Location of instrumentation, data logger and multiplexer in the mine.

Figure 35 Instrumentation location in stable pillar and roof (Plan view).
Figure 36 Approximate distance between the face location and the instrumentation location at time of installation was in the order of 1000 ft.

Figure 37, Figure 38, and Figure 39 show the cross-section view of instrumentation at Location 1 and Location 2. Location 1 includes BPC-1 and Potentiometers 1, 2 and 3. Location-2 includes BPC-2 and Potentiometers 4, 5 and 6. The two locations are separated by a horizontal distance of 28 feet. Location 2 is closer towards the recovery room while Location-1 is towards the bleeder entries.
Figure 37 Instrumentation layout at location-1 (cross-section view).
Figure 38 Instrumentation layout at location-2 (cross-section view)
Figure 39 Instrumentation layout in stable pillar (cross-section view)

4.3 Instruments Used

Instrumentation equipment were purchased from two separate companies: Geokon and Simplified Mine Instruments (SMI). It was ensured that the data logger is compatible with the two different types of instruments, which were purchased, from two different providers. The data logger, one multiplexer, two Borehole Pressure Cells (along with the hydraulic connections, installation tool and data cables) and associated software was procured from Geokon. Simplified Mine Instruments provided the six borehole extensometers and connecting rods along with the data cables and installation tools. See the detail specifications of instruments in Appendix 1.

Figure 40, Figure 41, Figure 42, and Figure 43 show parts and tools used for the BPC installation. Figure 40 shows the pre-encapsulated BPC flat-jacks in quick setting cement.
The pre-encapsulation was performed by Geokon and the BPCs were received as shown in Figure 40. Each BPC was inserted to the farthest end of the drill hole (into the pillar rib) using the installation kit shown in Figure 42 and was pressurized to 2000 psi using the hydraulic pump shown Figure 41. The diameter of the BPC is 2.25 inches. Initially the hole was drilled with a 2.25” diameter drill bit, but the encapsulated BPC could not make it to the end of the drill hole. To overcome this issue, a 2.5” hole was drilled up to 9.5 feet from the rib and then a 2.25” bit was used for the remainder of the drill hole. The encapsulated BPC was then installed very easily.

Figure 40 Borehole Pressure Cell shown pre-encapsulated in a cylinder of quick setting cement (photo retrieved from Geokon website).
Figure 41 Hydraulic pump used to install the BPCs.

Figure 42 Borehole Pressure Cell installation tool.
Figure 43 Data logger to multiplexer connections (photo from the Geokon website).

Figure 44 and Figure 45 show the parts of the borehole extensometers that were utilized in this instrumentation project. Figure 44 shows the potentiometer for the borehole extensometer. The potentiometer protrudes about 4 inches from the mouth of the borehole. It was then connected to the extension rods, which are inserted into the borehole. The extensometers have a full travel capacity of 1.5 inches. The data cable from the potentiometer was then connected to the data logger.
Figure 44 Potentiometer for borehole extensometer.

Figure 45 shows the extension rods. One end was attached to the rock at the top of the drill hole (in the roof) via the small flexible steel plate and the other is attached to the potentiometer at the collar of the borehole. Figure 46 shows the specifications of the Spring Return Linear Position Sensor used inside the potentiometer.
Figure 45 Borehole extensometer connecting rod and attachments.
### Specifications

<table>
<thead>
<tr>
<th>Model</th>
<th>9605</th>
<th>9610</th>
<th>9615</th>
</tr>
</thead>
<tbody>
<tr>
<td>Part Number</td>
<td>B0901.7XL2.0</td>
<td>B0H03.4XL2.0</td>
<td>B0H05.1XL2.0</td>
</tr>
<tr>
<td>Total Electrical Travel (A) inches (mm)</td>
<td>0.90 (22.7)</td>
<td>1.00 (25.4)</td>
<td>1.50 (38.1)</td>
</tr>
<tr>
<td>Total DC Resistance ± 25%</td>
<td>1.7K</td>
<td>3.4K</td>
<td>5.1K</td>
</tr>
<tr>
<td>Linearity Over Active Electrical Travel</td>
<td>± 1.0%</td>
<td>± 1.0%</td>
<td>± 1.0%</td>
</tr>
<tr>
<td>Best Practical Linearity (Option)</td>
<td>± 0.5%</td>
<td>± 0.5%</td>
<td>± 0.5%</td>
</tr>
<tr>
<td>Power Rating at 70°C, Watts</td>
<td>0.25</td>
<td>0.50</td>
<td>0.75</td>
</tr>
<tr>
<td>Mechanical Travel ± 0.05 (± 0.4)</td>
<td>0.89 (22.6)</td>
<td>1.92 (48.8)</td>
<td>1.89 (48.0)</td>
</tr>
<tr>
<td>(B) inches (mm)</td>
<td>0.90 (22.9)</td>
<td>1.96 (49.8)</td>
<td>1.90 (48.0)</td>
</tr>
<tr>
<td>(C) inches (mm)</td>
<td>0.10 (2.5)</td>
<td>0.20 (5.1)</td>
<td>0.20 (5.1)</td>
</tr>
<tr>
<td>Terminal Spacing (D) inches (mm)</td>
<td>0.30 (7.6)</td>
<td>0.60 (12.7)</td>
<td>0.90 (23.0)</td>
</tr>
<tr>
<td>(E) inches (mm)</td>
<td>0.20 (5.1)</td>
<td>0.50 (12.7)</td>
<td>0.70 (17.8)</td>
</tr>
<tr>
<td>Fully Extended Length ± 0.015 (± 0.4)</td>
<td>6.810 (173.0)</td>
<td>1.310 (33.3)</td>
<td>1.810 (45.9)</td>
</tr>
<tr>
<td>(F) inches (mm)</td>
<td>0.810 (20.6)</td>
<td>1.310 (33.3)</td>
<td>1.810 (45.9)</td>
</tr>
<tr>
<td>Mechanical Life</td>
<td>1,000,000 Full Cycles, 10,000,000 Dither Cycles</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seal Strength (Newtons)</td>
<td>386 (86.0)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actuation Force (Newtons)</td>
<td>14.4 (4.3) Maximum, supplied with metal spring to return actuator to extended position</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Humidity</td>
<td>85% @ 38°C</td>
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<tr>
<td>Vibration</td>
<td>15g’s 50 to 5,000 Hz, 3 axes</td>
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<td></td>
</tr>
<tr>
<td>Shock</td>
<td>Up to 50G’s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temperature Limits</td>
<td>-40°C to +155°C</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 46 Specifications for the sensor used in the potentiometer.
Figure 47 shows the instrumentation layout diagram. The 6 potentiometers of the borehole extensometers are connected to the data logger while the 2 BPCs are connected to the Multiplexer and the Multiplexer is connected to the data logger. Data was recorded every one hour.

Figure 47 Instrumentation layout diagram and connections to data logger and multiplexer units.
4.4 Instrumentation Results

4.4.1 Data from Borehole Pressure Cells

The two BPCs instrumented in the abutment pillar are named as “BPC-1” and “BPC-2” for this research. BPC-1 was connected to channel 1 and BPC-2 was connected to channel 2 of the multiplexer. The vibrating wire pressure transducer calibration report as provided by Geokon, includes the calibration equations to determine the measured pressure by the Borehole Pressure Cells (calibration sheets in Appendix 2). Two different equations are provided for calculating pressure: a linear equation and a polynomial equation. For the purpose of this research, the linear equation, as shown below, is used.

\[ P = G(R_1 - R_0) + K(T_1 - T_0) - (S_1 - S_0) \]

In the above equation, \( G \) is the Linear Gauge Factor (MPa), \( K \) is the Thermal Factor (MPa/°C), \( T_0 \) is the temperature reading (provided by the manufacturer), \( T_1 \) is the temperature readings obtained from the BPC, \( R_0 \) is the factory zero reading (provided by the manufacturer), and \( R_1 \) is the temperature readings obtained from the BPC. The part of the equation \((S_1 - S_0)\) is not required in the case of vented transducers.

The serial number for BPC-1 is “1619923”. Table 2 shows the list of values used to convert the Datalogger readings for BPC-1 to pressure (MPa).

<table>
<thead>
<tr>
<th>(MPa) Linear Gauge factor (G)</th>
<th>-0.008589 (MPa/digit)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thermal Factor (K)</td>
<td>0.002974 (MPa/°C)</td>
</tr>
<tr>
<td>Factory Zero Reading</td>
<td>8905</td>
</tr>
<tr>
<td>Temperature</td>
<td>21.5 °C</td>
</tr>
</tbody>
</table>

Table 2 Parameters used to calculate pressure for BPC-1.
The serial number for BPC-2 is “1619922. Table 3 shows the list of values used to convert the Datalogger readings for BPC-2 to pressure (MPa).

Table 3 Parameters used to calculate pressure for BPC-2.

| (MPa) Linear Gauge factor (G) | -0.008552 (MPa/digit) |
| Thermal Factor (K)            | 0.002803 (MPa/°C)    |
| Factory Zero Reading          | 8901                 |
| Temperature                   | 21.5 °C              |

The readings from the Datalogger was retrieved using the Loggernet software. The data was then converted into pressure readings (MPa). The raw data collected between January 4, 2019 and April 29, 2019 is shown in Figure 48.
Figure 48 Raw Pressure readings obtained from BPC-1 and BPC-2.

From Figure 48 it is clear that between March 12, 2019 and April 29, 2019, when the Datalogger from NIOSH was connected, the pressure readings are a flat line maxed out at around 76 MPa. That makes it easier to argue that the Borehole Pressure Cells have failed or the cables were cut.

This data is refined for the period between January 4, 2019 until January 25, 2019, when the first abutment load was registered. Figure 49 shows the pressure readings obtained from BPC-1. Figure 50 shows the pressure readings obtained from BPC-2. From Figure 49 and Figure 50, it is clear that the peak vertical stress values registered at a depth of 10 feet inside the pillar ribs are between 55 MPa to 60 MPa. The loading profile obtained from both the Borehole Pressure Cells is identical.
Figure 49 Pressure readings obtained from BPC-1.
4.4.2 Data from Roof Extensometers

The six Roof Extensometers are named as “POT-1”, “POT-2”, “POT-3”, “POT-4”, “POT-5”, and “POT-6” for this research. POT-1 was connected to channel 1; POT-2 was connected to channel 2; POT-3 was connected to channel 3; POT-4 was connected to channel 4; POT-5 was connected to channel 5; and POT-6 was connected to channel 6 of the Datalogger. The chronology of events was the same for the extensometers, as that for the Borehole Pressure Cells, explained before.

The roof extensometers were calibrated such that the Loggernet software will directly show the roof sagging data in inches (Figure 51 and Figure 52). The software registers and stores any movement in the connecting rod.
Figure 51 Loggernet software reading when extensometer is in lock position.
Figure 52 Loggernet software reading when extensometer is in released position.

Figure 53 to Figure 58 shows the readings for the six extensometers.
Figure 53 Roof extension data obtained from POT-1.
Figure 54 Roof extension data obtained from POT-2.
Figure 55 Roof extension data obtained from POT-3.
Figure 56 Roof extension data obtained from POT-4.
Figure 57 Roof extension data obtained from POT-5.
Figure 58 Roof extension data obtained from POT-6.

The instrumentation layout was designed such that a pair of extensometers are located at equal distance from the pillar ribs (Figure 37 and Figure 38). The pair of extensometers that are at equal distance from the pillar ribs are POT-1 and POT-4; POT-2 and POT-5; and POT-3 and POT-6. Since the extensometers are at equal distance from the pillar ribs, it is expected to get similar roof displacement readings. Comparing the readings from POT-1 and POT-4; POT-2 and POT-5; and POT-3 and POT-6, it can be seen that the displacements readings did not match (Figure 59 to Figure 61). Therefore, it can be concluded that the results from the extensometers are mostly inconclusive.
Figure 59 Comparing extensometer readings for POT-1 and POT-4
Figure 60 Comparing extensometer readings for POT-2 and POT-5
Figure 61 Comparing extensometer readings for POT-3 and POT-6
5 NUMERICAL MODELING

5.1 Background Research about Numerical Modeling of Coalmines

The physical study of the strata-pillar interaction is difficult to achieve. Measurements of this interaction process may be possible at a few points, but the complete picture is not obtained. Instrumentation data in a mine is often influenced by several uncontrollable external factors. These issues thus, justify the use of numerical analysis (Barczak, 2011). Today, numerical modeling is used ubiquitously in engineering problems. However, it should be noted that the numerical modeling in Rock Mechanics follows a different approach than other branches of engineering.

Figure 62 illustrates the classification of modeling problems, as explained by Holling et al. (1978), and quoted by Starfield and Cundall (1988). Holling et al. (1978) introduced two axes: the vertical axes indicating quantity/quality of available data and the horizontal axes indicates understanding of the problem. This space is divided into four quadrants. In quadrant 1, there is little understanding but enough data (statistical analysis is feasible for this case). In quadrant 3 both understanding, and quality data is available. In quadrant 2 and quadrant 4, there is always a scarcity of data. Modeling problems in rock mechanics falls into quadrants 2 and 4, which are data-limited problems. Holling’s classification explained here is the general methodology for rock mechanics problems (Starfield and Cundall, 1988). This classification can also be regarded as being suitable for numerical modeling of the strata-pillar interaction problem.
Figure 62 Classification of modeling problems cited by Starfield and Cundall (1988) from Holling et al. (1978).

Esterhuizen, Mark, and Murphy (2010b) used numerical analysis in FLAC3D to model the Ground Reaction Curves for various panel spans, as well as the Support Reaction Curves for varying coal pillar sizes. This numerical model was based on input parameters, material properties and modeling approach as established in Esterhuizen, Mark, and Murphy, (2010a).

Table 4 provides an inclusive but not exhaustive list of recent publications describing numerical model validation and calibration techniques, for accurately simulating the mechanical response of coal mines in FLAC3D. Procedures to estimate the rock mass properties based on point load test and the CMRR values, was introduced by Esterhuizen, Bajpayee, Ellenberger, and Murphy (2013). Tulu, Esterhuizen, Mohamed, and Klemetti
(2017) describes the modeling approach and input parameters for overburden material, coal material and gob material. Entry scale numerical models have been generated to determine the local ground response and secondary support response (Esterhuizen, Gearhart, Klemetti, Dougherty, and Dyke, 2018). Numerical modeling procedures have been validated and calibrated to model coal measure response; varying from panel scale phenomena like caving (Larson and Lavoie, 2016), to pillar scale phenomena like coal pillar rib failures (Mohamed, Tulu, and Murphy, 2016). Therefore, modeling the large scale to small scale response of underground coal mines can be satisfactorily achieved by using the validation and calibration techniques suggested in such publications. However, the input parameters and modeling approaches suggested in these publications can be useful for an initial estimation. Therefore, each numerical model needs to be calibrated independently.
Table 4 Recent list of publications about numerical model validation and calibration of coalmines in *FLAC3D* (Ray, Newman, and Agioutantis, 2019).

<table>
<thead>
<tr>
<th>Publication Details</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Esterhuizen, Gearhart, and Tulu, 2018)</td>
<td>Instrumentation data was used to calibrate numerical model in <em>FLAC3D</em> and validate the numerical modeling procedures developed to evaluate entry support systems and cable bolts defined in the model.</td>
</tr>
<tr>
<td>(Esterhuizen, bajpayee, Ellenberger, and Murphy, 2013)</td>
<td>Procedures were developed to estimate rock strength properties suitable for numerical modeling in <em>FLAC3D</em> based on the point load test and the CMRR outputs. Rocks were modeled using the ubiquitous joint approach and contacts modeled as explicit interfaces. Procedures were explained to estimate bedding cohesion, tensile strength and friction angle from CMRR data.</td>
</tr>
<tr>
<td>(Tulu, Esterhuizen, Mohamed, and Klemetti, 2017)</td>
<td>This paper provides a basic set of input data and a modeling approach for modeling overburden material, coal material and gob material in <em>FLAC3D</em>. Shearing along the bedding planes was modeled with ubiquitous joint elements and interface elements. Coal was modeled with the Coal Mass Model. Gob response was calibrated with back analysis of subsidence data.</td>
</tr>
<tr>
<td>(Esterhuizen, Gearhart, Klemetti, Dougherty, and Dyke, 2018)</td>
<td>Instrumentation data was used to validate numerical model in <em>FLAC3D</em>, to assist in the design of entry support systems. The developed models were used to compare the load paths and expected support requirements.</td>
</tr>
<tr>
<td>(Esterhuizen, Mark, and Murphy, 2010a)</td>
<td>Numerical model in <em>FLAC3D</em> was calibrated and validated against observed and measured performance of coal pillars and the overburden. Input parameters for the overburden were determined from a large database of laboratory tests and model was calibrated against maximum subsidence and subsidence curvature.</td>
</tr>
<tr>
<td>(Larson and Lavoie, 2016)</td>
<td>A study was conducted to determine the procedure for calibrating a caving model in <em>FLAC3D</em>. The model is calibrated to the measured surface subsidence data.</td>
</tr>
<tr>
<td>(Mohamed, Tulu, and Murphy, 2016)</td>
<td>This paper presented a coal rib model, using Coal Mass Model, in <em>FLAC3D</em> to simulate the mechanism of coal deformation and fracturing that leads to rib falls. This coal rib model was calibrated against published data.</td>
</tr>
</tbody>
</table>

5.2 Numerical Modeling of the Longwall Coalmine in *FLAC3D*

Numerical modeling is a powerful tool to analyze complex Geomechanics problems. This tool is used extensively to model, verify and calibrate the geomechanical response of the coal pillar and longwall panels. The numerical modeling technique is used as an integral...
part of this research, right from the initial step of preliminary modeling to guide the instrumentation process to the intermediate step of calibrating the pillar scale models and panel scale models to the observed and measured data. Finally, the Support Reaction Curves and Ground Reaction Curves is obtained using numerical methods.

This research is devoted to applying numerical modeling techniques to study the geomechanical effects of strata-pillar interaction using the GRC Concept. The studies are conducted using the FLAC3D numerical modeling code, developed by Itasca Consulting Group, Inc. This code uses the finite-difference approach to solve an explicit Lagrangian solution scheme using mixed discretization procedures. This approach makes it a powerful tool to address non-linear problems, thus making it suitable to modeling geo-materials, mining excavations and support systems.

The software also has a built-in programming language called FISH, which facilitates addition of functionalities not included in the standard code. Thus, it allows the user to control loads and displacements in the model. This feature is explicitly used to determine the Support Reaction Curves and the Ground Reaction Curves from the calibrated models. The numerical modeling of the longwall coalmine required the modeling of coal material response, overburden response, gob loading, and pillar loading during various stages of the longwall mining process.

5.3 Pillar Scale Modeling

The strength of coal material has been extensively tested at various scales. The results indicate that as the sample size of coal specimen increases, the strength decreases. For the purpose of numerical modeling, it is critical to know the strength of coal material, its residual strength and the rate of strength decay.
In the recent literature, two methods for modeling coal material response have been proposed. Esterhuizen, Mark, and Murphy (2010a) suggested the first method. This method uses the Hoek-Brown constitutive model to simulate the coal material. Later, Mohamed, Tulu, and Murphy (2016) developed a Coal Mass Model based on the Mohr-Coulomb constitutive model. This research tries to use both methods to model the coal material response.

5.3.1 Modeling coal material using the Hoek-Brown constitutive model

The Hoek-Brown criterion is a non-linear relationship between maximum and minimum principal stresses. The resulting equation is written as follows:

$$
\sigma_1 = \sigma_3 + \sigma_c \left( m_b \frac{\sigma_3}{\sigma_c} + s \right)^a
$$

In this equation, $\sigma_1$ and $\sigma_3$ are the major and minor principal stresses, $\sigma_c$ is the unconfined compressive strength of the intact rock, and $m_b$, $s$ and $a$ are material constants derived empirically. While the parameters can be determined for smaller coal samples, it is not practical to determine the parameters for large-scale samples by direct testing. Therefore, these material constants were determined using a combination of methods that involved statistical analysis of triaxial test data and numerical modeling of coal pillars calibrated against empirically derived pillar strength equations (Esterhuizen, Mark and Murphy, 2010a).

5.3.1.1 Validating coal pillar model using Hoek-Brown constitutive model

For the purpose of this research, similar numerical models of coal pillars are created with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal, and 2 meters of floor strata. Due to symmetry, quarter pillar models are created. The coal material is
contained between elastic roof and elastic floor materials. Interface elements are used between the coal layer, and both the roof and floor materials. The same boundary conditions and Hoek-Brown input parameters (Table 5), as used by Esterhuizen, Mark, and Murphy (2010a) are used in the models. The coal material is modeled using the strain-softening Hoek-Brown constitutive model in FLAC3D. The strength of the coal material is decreased from the peak to the residual value, over a plastic strain value of 0.04 for the given zone sizes. The zone sizes used are of 0.33 m (0.33 x 0.33 x 0.33). The increasing stress state of the pillar models were simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining the sides of the model (the excavated sides of the coal material has no boundary conditions). The stress-strain response of the pillars is obtained by recording the average stress in the coal material elements and the strain between two points located vertically at the top and bottom of the coal seam. The pillar models are tested such that the stress-strain curve passes through the elastic part up to their peak strength and are then allowed to yield to their residual strength values. It should be noted that changing the strength of the roof and floor materials or changing the interface properties, could have a significant effect on the obtained stress-strain curves (Esterhuizen, Mark, and Murphy, 2010a).
Table 5 Hoek-Brown parameters for modeling coal material in *FLAC3D* as used by Esterhuizen, Mark, and Murphy. (2010a).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (lab scale)</td>
<td>20 MPa</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>3 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
</tr>
<tr>
<td>m-value</td>
<td>1.47</td>
</tr>
<tr>
<td>s-value</td>
<td>0.07</td>
</tr>
<tr>
<td>m-residual</td>
<td>1.0</td>
</tr>
<tr>
<td>s-residual</td>
<td>0.001</td>
</tr>
<tr>
<td>Interface friction angle</td>
<td>25</td>
</tr>
<tr>
<td>Interface Cohesion</td>
<td>0.1 MPa</td>
</tr>
<tr>
<td>Interface tensile strength</td>
<td>0.0</td>
</tr>
<tr>
<td>Interface normal stiffness</td>
<td>100 GPa/m</td>
</tr>
<tr>
<td>Interface shear stiffness</td>
<td>50 GPa/m</td>
</tr>
<tr>
<td>a-parameter</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Figure 63 through Figure 67 show the resulting stress-strain curves obtained from the pillar models, using the Hoek-Brown constitutive model for coal material, with varying pillar width-to-height ratios. Figure 68 shows the stress-strain behavior of the pillars and is similar to the results published by Esterhuizen, Mark, and Murphy. (2010a). The pillars exhibit a strain-softening behavior for width-to-height ratios below 8, and a strain-hardening behavior above 8 (Figure 68). Figure 69 shows the comparison between the peak strengths of the modeled coal pillars to the empirical Bieniawski pillar strength equation.
Figure 63 Stress-strain curve of a quarter pillar with width-to-height ratio of 3 using a Hoek-Brown constitutive model for coal material.

Figure 64 Stress-strain curve of a quarter pillar with width-to-height ratio of 4 using the Hoek-Brown constitutive model for coal material.
Figure 65 Stress-strain curve of a quarter pillar with width-to-height ratio of 6 using the Hoek-Brown constitutive model for coal material.

Figure 66 Stress-strain curve of a quarter pillar with width-to-height ratio of 8 using the Hoek-Brown constitutive model for coal material.
Figure 67 Stress-strain curve of a quarter pillar with width-to-height ratio of 10 using the Hoek-Brown constitutive model for coal material.

Figure 68 Stress-strain curves obtained from numerical models of coal pillars with width-to-height ratios from 3 to 10 using the Hoek-Brown constitutive model.
Figure 69 Pillar peak strength results obtained by numerical models after validating the models (using the Hoek-Brown constitutive model for coal material) to the empirical pillar strength equation of Bieniawski (1989).

Therefore, it can be concluded that the Hoek-Brown parameters for modeling coal material in *FLAC3D*, as suggested by Esterhuizen, Mark, and Murphy (2010a) can be used to realistically model the coal material and the load-deformation response of coal pillars for coalmines in the United States.

### 5.3.1.2 Calibrating Hoek-Brown model parameters for coal material in the Longwall Mine

The method described above helps to validate the use of the Hoek-Brown constitutive model to simulate coal material, as used by Esterhuizen, Mark, and Murphy (2010a). However, the Hoek-Brown parameters suggested by Esterhuizen, Mark, and Murphy (2010a) is a general representation of coal material in the United States. These Hoek-
Brown parameters has to be further calibrated to accurately simulate the coal material of the longwall coalmine under study.

For the purpose of calibration, a fully discretized model of an abutment pillar is created, with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal seam, and 2 meters of floor strata. The model is 134 meters long and 48 meters wide. The extraction ratio for this pillar (when viewed as a stand-alone element and when surrounded by half width entries) is 5.59% (the entry width is taken to be 1 meter). The material properties for the roof, floor and interface elements is kept the same as used to model the quarter pillars, as suggested by Esterhuizen, Mark, and Murphy (2010a). The zone sizes used for this model is 0.33m (0.33 x 0.33 x 0.33). This model consists of 1,022,544 zones. The increasing stress state on the model is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining all the four sides of the model (only the roof and floor rock is constrained, while the excavated coal layer had a free face).

The vertical stress (zz stress) at a depth of 10 feet inside the pillar ribs (instrumented locations for BPC-1 and BPC-2) of the abutment pillar is recorded. The calibrated model results are shown in Figure 70. The pressure data obtained from BPC-1 and BPC-2 is used to calibrate the coal material for this longwall coalmine. The calibrated Hoek-Brown parameters for modeling the coal material for the longwall coalmine are shown in Table 6. In Figure 70, it can be seen that the vertical stress obtained from the calibrated numerical simulation, is similar to the vertical loading profile measured by BPC-1 and BPC-2 (as shown in Figure 49 and Figure 50). It can be noted that, to achieve calibration, only the UCS value for the coal material has been increased to 30 MPa. Since, this coal mine is at a greater depth than other coal mines in the USA, it is assumed that the coal material at this mine is of higher strength value.
Figure 70 Calibrated Abutment Pillar (using Hoek-Brown constitutive model to simulate coal material) showing the vertical stress at a depth of 10 feet (Instrumented location for BPC-1 and BPC-2) from pillar ribs.
Table 6 Calibrated Hoek-Brown parameters for modeling coal material of the longwall coalmine in FLAC3D.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCS (lab scale)</td>
<td>30 MPa</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>3 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.25</td>
</tr>
<tr>
<td>m-value</td>
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</tr>
<tr>
<td>s-value</td>
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</tr>
<tr>
<td>m-residual</td>
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</tr>
<tr>
<td>s-residual</td>
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</tr>
<tr>
<td>Interface friction angle</td>
<td>25</td>
</tr>
<tr>
<td>Interface Cohesion</td>
<td>0.1 MPa</td>
</tr>
<tr>
<td>Interface tensile strength</td>
<td>0.0</td>
</tr>
<tr>
<td>Interface normal stiffness</td>
<td>100 GPa/m</td>
</tr>
<tr>
<td>Interface shear stiffness</td>
<td>50 GPa/m</td>
</tr>
<tr>
<td>a-parameter</td>
<td>0.65</td>
</tr>
</tbody>
</table>

5.3.1.3 Modeling coal pillars and coal panels using the Equivalent Element Method

After calibrating the material properties for modeling coal material, it is time to model the coal pillars and coal panels in the panel scale model. The material property is calibrated for a zone size of 0.33m (0.33 x 0.33 x 0.33). For modeling a longwall coal panel (of dimensions 210 meters wide, 3050 meters long and 2 meters high) using the same zone size, around 35 million zones are required. Therefore, modeling the geomechanical details of the coal pillars and the coal seam in the panel scale model, in a single three-dimensional FLAC3D model is a challenge in terms of memory and computing time.
To overcome this challenge, a modeling technique known as the equivalent element method is used to model the coal pillars and coal panels. Board, Damjanac, and Pierce (2007) developed this method to model trona pillars. Esterhuizen and Mark (2009) used this technique to model coal pillars.

The equivalent element method is based on replacing the coal seam, the surrounding rooms, part of the immediate roof and floor rocks, by an equivalent element that has the same load-deformation response (Figure 71). This method allows simulating the details of the coal seam, roof and floor rock and boundary conditions, to be incorporated in the equivalent elements with lesser discretization. By using the equivalent elements, the panel scale model can be used to include the geomechanical details, without costing computing efficiency (Esterhuizen and Mark, 2009).

Figure 71 Concept of an equivalent element model showing (a) the detailed geology and mining geometry being investigated, (b) the resulting load-displacement relationship, and (c) the uniform equivalent element that follows the same load-displacement relationship (Esterhuizen and Mark, 2009).
During various stages of mining activities, the coal material and the pillars undergo different loading conditions. This loading can cause pillar punching, roof collapse, floor heave or failure of pillars. To capture such detailed phenomena, a smaller discretization of zones is necessary. However, such a detailed analysis is not required for the panel scale models. Therefore, an equivalent element that has the same load-deformation curve can be used in the panel scale model with lesser discretization making the model less computationally intensive (Esterhuizen and Mark, 2009).

The panel scale model consists of mine structures like abutment pillars, yield pillars, two longwall panels and unmined coal seam. The Hoek-Brown parameters to model the coal material have already been determined through calibration. In case of large, mine structures like abutment pillars and longwall panels, the coal elements on the edges and corners, have a different loading profile than that in the middle of the coal structure. The elements on the edges and corners undergo plastic deformation, while the elements in the middle behave elastically. Therefore, in order to accurately model the coal material, three different types of equivalent elements has to be designed that corresponds to the load-deformation behavior of edge elements on the edges of the coal structure, corner elements on the corners of the coal structure, and solid elements in the middle of the coal structure.

In order to determine the stress-strain curve for the edge elements, a densified numerical model of zone size 0.33 m (0.33 x 0.33 x 0.33) is created with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal, and 2 meters of floor strata. The material properties for the roof, floor and interface elements is kept the same as used to model the quarter pillars. The coal material is modeled using the calibrated Hoek-Brown parameters (Table 6). An extraction ratio of 16.66% is used in the case of edge elements (as seen in the plan view the coal seam is excavated from one side to represent an edge element of a pillar).
The increasing stress state on the model for edge element is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining the sides of the model (with one excavated side of the coal material having a free surface). The stress-strain response of the model for edge element is obtained by recording the average stress at the base of the model and the strain between two points located vertically at the top and bottom of the model.

There is a significant difference between how the stress-strain curve is determined for the quarter pillar models (Figure 63 to Figure 67), to the stress-strain curve for equivalent elements. In case of coal pillar models, the focus is to determine the stress-strain in the coal material embedded between the roof and floor rocks, thus the average stress in the coal material and vertical strain in the coal material is measured. In case of the equivalent elements, the focus is to find the total stress and total strain response of the full model; that includes the response of coal material, roof and floor rocks. This is a slight difference in the method as used by Esterhuizen and Mark (2009) and Damjanac, Pierce, and Board (2014), where a formula is suggested to convert the stresses based on extraction ratio. The edge element model is loaded such that the stress-strain curve passes through the elastic part up to its peak strength and is then allowed to yield to its residual strength value (Figure 72).
Figure 72 Stress-strain curve of a densified edge element model (using Hoek-Brown calibrated parameters for coal material) with extraction ratio of 16.66 %.

The equivalent edge element (represented by a single element) shall respond to the vertical closure between the roof and floor of the panel scale model in the same way as that of the actual edge elements with higher discretization. The Coulomb strain-softening logic in FLAC3D is used to control the response of the equivalent edge elements. It is important to modify the behavior of the equivalent elements such that the horizontal confinement will not be generated when it undergoes deformation in the panel scale model, since the effects of confinements is already accounted for in the discretized edge element model. Therefore, the Poisson’s ratio of the equivalent element should be zero so that lateral dilation does not occur, that can cause additional lateral confinement. In addition, the horizontal stress components are reset to zero (after every solution step) during the model solution. The friction angle of the equivalent element is set to zero, so that the strength and load-deformation response of the equivalent element can be controlled by varying the cohesion values only (Esterhuizen and Mark, 2009).
In order to model the equivalent edge element, it is assumed that the model behaves elastically up to 12.8 MPa, and then reaches a peak stress of 30.4 MPa, followed by yielding to a residual strength value of 22.9 MPa. At every point on the stress-strain curve, the elastic component of strain is subtracted from the total strain, to generate the cohesion table (pairs of plastic strain corresponding to the cohesion value). The elastic modulus for the edge model (that represents the combined modulus of the coal material, roof and floor rock) is determined to be 6.86 GPa from the stress-strain curve (Figure 72). The calculations to determine the cohesion and plastic strain values are given in Table 7. The equivalent edge element is created with dimensions of 6m (6 x 6 x 6). The density is calculated to be 2355 kg/m³. The stress-strain curve for the edge equivalent element is shown in Figure 73. It can be noticed in Figure 73 that there are no horizontal principal stress tensors components since the horizontal stresses were zeroed after each solution step.
Table 7 Calculation of the pairs of plastic strain and cohesion values for the cohesion table to create equivalent edge element.

<table>
<thead>
<tr>
<th>Total Stress (Pa)</th>
<th>Total Strain</th>
<th>Elastic Strain</th>
<th>Plastic Strain</th>
<th>Cohesion (Pa)</th>
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</table>
In order to determine the stress-strain curve for the corner elements, a densified numerical model of zone size 0.33 m (0.33 x 0.33 x 0.33) is created with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal, and 2 meters of floor strata. The material properties for the roof, floor and interface elements is kept the same as used to model the quarter pillars. The coal material is modeled using the calibrated Hoek-Brown parameters (Table 6). An extraction ratio of 31.42% is used in the case of corner elements (as seen in the plan view the coal seam is excavated from two sides to represent a corner element of a pillar).

The increasing stress state on the model for corner element is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining only two sides of the model (with the other two excavated sides of the coal material having a free surface). The stress-strain response of the model for corner element is obtained by recording the average stress at the base of the model and the strain between two points located vertically at the top and bottom of the model. The corner element model
is tested such that the stress-strain curve passes through the elastic part up to its peak strength and is then allowed to yield to its residual strength value (Figure 74).

**Figure 74 Stress-strain curve of the densified corner element model (using Hoek-Brown calibrated parameters for coal material) with extraction ratio of 31.42 %.

Then the equivalent corner element (represented by a single element) is created to respond to the vertical closure between the roof and floor of the panel scale model, in the same way as that of the actual edge elements with higher discretization. The Coulomb strain-softening logic in FLAC3D is used to control the response of the equivalent edge elements. The modeling method is similar as that used to model the equivalent edge element.

In order to model the equivalent edge element, it is assumed that the model behaves elastically up to 9.77 MPa, and then reaches a peak stress of 16.3 MPa, followed by yielding to a residual strength value of 9.4 MPa. At every point on the stress-strain curve, the elastic component of strain is subtracted from the total strain, to generate the cohesion table. The elastic modulus for the corner element model is determined to be 5.23 GPa from the stress-strain curve (Figure 74). The calculations to determine the cohesion and plastic strain...
values are given in Table 8. The equivalent corner element is created with dimensions of 6m (6 x 6 x 6). The density is calculated to be 2262 kg/m³. The stress-strain curve for the corner equivalent element is shown in Figure 75. It can be noticed in Figure 75 that there are no horizontal principal stress tensor components since the horizontal stresses were zeroed after each solution step.

Table 8 Calculation of the pairs of plastic strain and cohesion values for the cohesion table to create equivalent corner element.

<table>
<thead>
<tr>
<th>Total Stress (Pa)</th>
<th>Total Strain</th>
<th>Elastic Strain</th>
<th>Plastic Strain</th>
<th>Cohesion (Pa)</th>
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After creating the equivalent edge element and equivalent corner element, the equivalent solid element has to be modeled. The solid coal element is in the middle of the coal structure (abutment pillar or coal panels), and thus undergoes higher confinement stress since it is compressed from all four sides (while the edge element is confined on three sides and corner element is confined on only two sides).

In order to determine the stress-strain curve for the solid elements, a densified numerical model of zone size 0.33 m (0.33 x 0.33 x 0.33) is created with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal, and 2 meters of floor strata. The material properties for the roof, floor and interface elements is kept the same as used to model the quarter pillars. The coal material is modeled using the calibrated Hoek-Brown parameters (Table 6).

The increasing stress state on the model for solid elements is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining all the four sides of the model. The stress-strain response of the densified solid...
elements model is obtained by recording the average stress at the base of the model and the strain between two points located vertically at the top and bottom of the model. The model results are shown in Figure 76. It can be noticed that the stress developed in the solid elements model is more than 320 MPa, without any sign of failure (due to confinement on all 4 sides). Therefore, for all practical purposes the solid elements can be treated as non-yielding elements.

Figure 76 Stress-strain curve of the densified solid elements model (using Hoek-Brown calibrated parameters for coal material).

The equivalent solid element is modeled similar to the equivalent corner element or the equivalent edge element. The density is calculated to be 2466 kg/m³. The Young’s modulus is calculated to be 7.44 GPa. The stress-strain curve for the equivalent solid element is shown in Figure 77.
Figure 77 Stress-strain curve of the equivalent solid element model.

It is a good modeling practice to test and verify the new elements, as it will be used in the panel scale models, after creating the equivalent elements. The combined response of the equivalent edge element, equivalent corner element, and equivalent solid element are tested by comparing the stress-strain response of fully discretized models of a yield pillar and an abutment pillar, to that of the models of yield pillar and abutment pillars made from a combination of equivalent elements.

A fully discretized model of a yield pillar is created with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal, and 2 meters of floor strata. The model is 42 meters long and 12 meters wide (looking from the top view). The extraction ratio for this pillar is 20.63% (the coal seam is excavated till a depth of 1 meter in the model). The width to height ratio for the yield pillar is 5. The material properties for the roof, floor and interface elements is kept the same as used to model the quarter pillars. The coal material is modeled using the calibrated Hoek-Brown parameters (Table 6). This model consists of 76,032 zones. The increasing stress state on the model is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and
constraining all the four sides of the model (only the roof and floor rock is constrained, while the coal material had a free surface on all 4 sides). The stress-strain response of the discretized yield pillar model is obtained by recording the average stress at the base of the model and the strain between two points located vertically at the top and bottom of the model. The model result is shown in Figure 78. The stress-strain curve shows that the yield pillar reaches a peak strength of over 24 MPa.

![Figure 78 Stress-strain curve of the discretized yield pillar model (using Hoek-Brown calibrated parameters for coal material).](image)

The yield pillar is again modeled using a combination of 4 equivalent corner elements and 10 equivalent edge elements. Therefore, only 14 elements are required when using equivalent elements, in comparison to the 76,032 elements used in the case of a fully discretized yield pillar model. The model is 42 meters long, 12 meters wide and 6 meters high. The material properties are the same as used for the equivalent corner and equivalent edge elements. The increasing stress state on the model is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary (the four sides need not be constrained in this case, as the equivalent elements are designed to
incorporate the respective boundary conditions). The stress-strain response of the equivalent elements yield pillar model is obtained by recording the average stress at the base of the model and the strain between two points located vertically at the top and bottom of the model. The model result is shown in Figure 79. It is evident from the similarities of the two curves in Figure 79 and Figure 78, that the equivalent elements technique is useful in modeling the response of the yield pillar.

Figure 79 Stress-strain curve of the yield pillar model created using equivalent corner elements and equivalent edge elements.

Again, a fully discretized model of an abutment pillar is created with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal, and 2 meters of floor strata. The model is 134 meters long and 48 meters wide (looking from the top view). The extraction ratio for this pillar is 5.59% (the coal seam is extracted to a depth of 1 meter in the model). The width to height ratio for the abutment pillar is 23. The material properties for the roof, floor and interface elements is kept the same as used to model the quarter pillars. The coal material is modeled using the calibrated Hoek-Brown parameters (Table 6). This model consists of 1,022,544 zones. The increasing stress state on the model is simulated by
imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining all the four sides of the model (only the roof and floor rock is constrained, while the coal material had a free surface on all 4 sides). The stress-strain response of the discretized yield pillar model is obtained by recording the average stress at the base of the model and the strain between two points located vertically at the top and bottom of the model. The model results are shown in Figure 80. From the distribution of vertical stresses, it can be seen that the elements on the edges and corner undergo failure, while the elements in the middle behave as non-yielding elements (Figure 81).

Figure 80 Stress-strain curve of the discretized abutment pillar model (using Hoek-Brown calibrated parameters for coal material).
Figure 81 Vertical stress distribution in the abutment pillar model (using Hoek-Brown calibrated parameters for coal material).

The abutment pillar is then modeled using a combination of 4 equivalent corner elements, 52 equivalent edge elements, and 120 equivalent solid elements. Thus, a total of 176 elements is required when using equivalent elements, in comparison to the 1,022,544 elements used in the case of a fully discretized abutment pillar model. The model is 134 meters long, 48 meters wide and 6 meters high. The material properties are the same as used for the equivalent corner element, equivalent edge element and equivalent solid element. The increasing stress state on the model is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary (the four sides need not be constrained in this case, as the equivalent elements are designed to incorporate the respective boundary conditions). The stress-strain response for the equivalent elements abutment pillar model is obtained by recording the average stress at the bottom of the model and the strain between two points located vertically at the top and bottom of the model. The model result is shown in Figure 82. It is evident from the similarities of the two curves in Figure 82 and Figure 80, that the equivalent elements technique is successful in modeling the response of the abutment pillar.
Figure 82 Stress-strain curve of the abutment pillar model created using equivalent corner elements, equivalent edge elements, and equivalent solid elements.

After verifying the ability of the equivalent elements, when used together, to successfully reproduce the stress-strain curves, as obtained by the densified models of the yield pillar and abutment pillar, it is time to model the coal seam using these equivalent elements. Thus, the coal seam in the panel scale model is divided into equivalent edge elements, equivalent corner elements and equivalent solid elements, according to its location in the panel scale model (Figure 83).
Figure 83 Coal seam divided into equivalent edge, equivalent corner, and equivalent solid elements as per its location in the panel scale model (zoomed-in plan view of the coal seam, while the overburden is hidden).

5.3.2 Modeling coal material using the Coal Mass Model

Another method to model coal material is by using the Coal Mass Model, developed at NIOSH (Mohamed, Tulu, and Klemetti, 2015). The Coal Mass Model simulates the pre-peak and post-peak behaviors of the coal material by using the strain softening, ubiquitous joint model available in FLAC3D. This model can simulate the fractures in the coal material using scale dependent constitutive parameters (Mohamed, Tulu, and Murphy, 2016).

The framework for describing the coal material model is established on three constitutive models: (1) the generalized Hoek-Brown failure criterion to calculate the peak-strength of rock-mass; (2) the Fang and Harrison local degradation model to calculate the residual stiffness and strength of rock; and (3) the Alejano and Alonso peak-dilation model to calculate the deformation of rock (Mohamed, Tulu, and Murphy, 2016). Figure 84 shows the four stages experienced by the coal material.
The first stage is the elastic stage. During this stage, Hooke’s law describes the coal material. The elastic range is defined by its Young’s modulus and Poisson’s ratio.
The second stage is the yielding stage. During this stage, the coal material reaches the peak strength, and is defined by generalized Hoek-Brown criteria. Mohamed, Tulu, and Murphy (2016) mentioned that the Mohr-Coulomb constitutive model is available in most numerical codes. Thus, the equivalent Mohr-Coulomb model parameters derived from the Hoek-Brown criterion is used.

The third stage is the degradation stage. During this stage, the stiffness and strength of the coal material degrades gradually until it reaches its residual Young’s modulus and residual strength. The coal material also dilates during this stage. The residual strength of the coal material is defined by matching it with the Fang and Harrison local degradation model to the generalized Hoek-Brown criteria. Again, the equivalent Mohr-Coulomb parameters is derived from the Hoek-Brown criterion.

The fourth stage is the tensile stage. During this stage, the tensile strength of the non-yielded coal material is calculated from the Hoek-Brown failure criterion (Mohamed, Tulu, and Klemetti, 2015). Tulu, Esterhuizen, Mohamed, and Klemetti (2017); and Rashed, Mohamed, Gearhart, and Esterhuizen (2019) used the Coal Mass Model to simulate the coal material for longwall mines.

In order to use the Coal Mass Model to simulate coal material, this method has to be validated. The stress-strain behavior of the coal pillars generated by using the Hoek-Brown constitutive model (Figure 63 to Figure 67) is assumed to be the reference method to validate coal material (Esterhuizen, Mark, and Murphy, 2010a). In order to compare the stress-strain behavior of the coal pillars, generated by using the Coal Mass Model, similar numerical models are created with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal, and 2 meters of floor strata. Due to symmetry, quarter pillars models are created. The coal material is contained between elastic roof and elastic floor materials. Interface elements is used between the coal material and both the roof and floor.
materials. The same boundary conditions and input parameters for roof rock, floor rock and interface elements were used as that by Esterhuizen, Mark, and Murphy (2010a).

The increasing stress state on the pillar models are simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining the sides of the model (with the excavated sides of the coal material with no boundary conditions). The stress-strain response of the pillars is obtained by recording the average stress in the coal material elements and the strain between two points located vertically at the top and bottom of the coal pillar. The pillar models are tested such that the stress-strain curve passes through the elastic part up to their peak strength and are then allowed to yield to their residual strength values. For the purpose of this research, the coal material of the instrumented abutment pillar (pillar scale model) is modeled using a zone size of 0.33 meter. Therefore, the Coal Mass Model parameters have to be validated for a zone size of 0.33 meter.

### 5.3.2.1 Validating coal pillar model using Coal Mass Model for 0.33-Meter zone size

For this validation case, the zone size for coal material is kept as 0.33 m (0.33 x 0.33 x 0.33). Figure 85 through Figure 89 shows the resulting stress-strain curves obtained from the pillar models, using Coal Mass Model for coal material, with varying pillar width-to-height ratios. Figure 91 shows the stress-strain behavior of all the pillars and is similar to the results published by Esterhuizen, Mark, and Murphy (2010a); and Tulu, Esterhuizen, Mohamed, and Klemetti (2017). Figure 92 shows the comparison between the peak strengths of the modeled pillars to the empirical Bieniawski pillar strength equation. The Coal Mass Model material properties used to validate 0.33-Meter zone size is shown in Table 9.
Table 9 Coal Mass Model material properties used to validate 0.33-Meter zone size.

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<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
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</tr>
<tr>
<td>Strength parameter, Ca</td>
<td>0.15</td>
</tr>
<tr>
<td>Fracture plastic shear strain</td>
<td>0.3</td>
</tr>
<tr>
<td>Fracture plastic tensile strain</td>
<td>0.3</td>
</tr>
<tr>
<td>Joint friction angle</td>
<td>25 degrees</td>
</tr>
</tbody>
</table>

Figure 85 Stress-strain curve of a quarter pillar with width-to-height ratio of 3 using the Coal Mass Model.
Figure 86 Stress-strain curve of a quarter pillar with width-to-height ratio of 4 using the Coal Mass Model.

Figure 87 Stress-strain curve of a quarter pillar with width-to-height ratio of 6 using the Coal Mass Model.
Figure 88 Stress-strain curve of a quarter pillar with width-to-height ratio of 8 using the Coal Mass Model.

Figure 89 Stress-strain curve of a quarter pillar with width-to-height ratio of 10 using the Coal Mass Model.
Figure 90 Stress-strain curves obtained from numerical models of coal pillars with width-to-height ratios from 3 to 10 using the Coal Mass Model.
5.3.2.2 Calibrating Coal Mass Model parameters for coal material in the Longwall Mine for 0.33-Meter zone size

The method described above is used to validate the use of Coal Mass Model to simulate coal material. However, the Coal Mass Model parameters are a general representation for coal material in the United States. The Coal Mass Model parameters have to be further calibrated for the coal material of the longwall coalmine under study.

For the purpose of calibration, a model of the abutment pillar is created, with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal, and 2 meters of floor strata. The model is 134 meters long and 48 meters wide (looking from the top view). The extraction ratio for the pillar is 5.59% (the coal seam is excavated till a depth of 1 meter in the model). The material properties for the roof, floor and interface elements is kept the
same as used to model the quarter pillars above. The increasing stress state on the model is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining all the four sides of the model (only the roof and floor rock is constrained, while the coal material had a free surface).

For the abutment pillar model, the zone sizes are 0.33m (0.33 x 0.33 x 0.33). The model consists of 1,022,544 zones. The vertical stress (zz stress) at a depth of 10 feet inside the pillar ribs (instrumented locations for BPC-1 and BPC-2) of the abutment pillar is recorded. The pressure data obtained from BPC-1 and BPC-2 is used to calibrate the coal material for this longwall coalmine. The calibrated model results for abutment pillar with 0.33 m zone size is shown in Figure 92. In Figure 92, it can be seen that the vertical stress from the calibrated numerical simulation for abutment pillar using the Coal Mass Model, is similar to the vertical loading profile measured by BPC-1 and BPC-2 (as shown in Figure 49 and Figure 50). The Coal-mass material properties used to calibrate 0.33-meter zone size is shown in Table 10. It can be noted that, to achieve calibration, only the intact compressive strength value for the coal material has been increased to 13 MPa. Since, this coal mine is at a greater depth than other coal mines in the USA, it is assumed that the coal material at this mine is of higher strength value.
Figure 92 Calibrated Abutment Pillar (using Coal Mass Model for 0.33m zone size) showing the vertical stress at a depth of 10 feet (Instrumented location for BPC-1 and BPC-2) inside pillar ribs.

Table 10 Coal Mass Model parameters used to calibrate abutment pillar with 0.33 m zone size.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk modulus</td>
<td>1333.3 MPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>800 MPa</td>
</tr>
<tr>
<td>Coal-mass scale</td>
<td>80</td>
</tr>
<tr>
<td>Intact compressive strength</td>
<td>13 MPa</td>
</tr>
<tr>
<td>Strength parameter, Ca</td>
<td>0.15</td>
</tr>
<tr>
<td>Fracture plastic shear strain</td>
<td>0.3</td>
</tr>
<tr>
<td>Fracture plastic tensile strain</td>
<td>0.3</td>
</tr>
<tr>
<td>Joint friction angle</td>
<td>25 degrees</td>
</tr>
</tbody>
</table>
5.4 Gob Response Modeling

Extraction of coal from longwall panel leads to the caving of overburden strata over the extracted panel. This caved zone is also known as gob. During the process of longwall extraction, the overburden stresses is redistributed over the unmined coal and gate road pillars. This redistribution of stresses is also known as abutment loading. After a critical span of coal is extracted from the panel, the overburden caves in and subsides over the extracted panel and is further compressed. This causes some of the abutment stresses to redistribute over the gob material. Therefore, it is necessary to realistically model the gob material to study the load distribution over gateroad pillars.

Esterhuizen, Mark, and Murphy (2010a) had explained that the gob can be modeled using two approaches. The first approach is to explicitly model the formation of gob such that the loading conditions and variations in geology can be studied. In this approach, the focus is to study the process of roof fracturing, caving and gob development. The second approach is to implicitly model the gob, such that the gob compaction and load redistribution between surrounding rocks and pillars is correctly modeled. As used in the case of Esterhuizen, Mark, and Murphy (2010a); and Tulu, Esterhuizen, Mohamed, and Klemetti (2017), this research uses the explicit method for gob modeling.

It is difficult to determine the gob characteristics from field data because of the large-scale displacements involved and the nature of caved rocks. However, laboratory tests of fragmented rocks have provided valuable insights into the compaction behavior of gob material. Laboratory tests on shale and sandstone fragments have showed that the stress-strain response of the caved material follows a strain-hardening curve (Pappas and Mark, 1993). Laboratory tests was performed on rock fragments similar to gob composition. Pappas and Mark (1993) used photographs of gob sites to estimate the size distribution of
rocks in the gob. These size distributions were scaled down to laboratory scale sample size. It was noticed that the stronger sandstone gob had a stiffer curve than the weaker shale material. Pappas and Mark (1993) have used the following hyperbolic function derived by Salamon (1990) to best fit the strain-hardening gob response curve.

\[ \sigma = \frac{a \times \varepsilon}{b - \varepsilon} \]

In this equation, \( \sigma \) is the vertical gob stress (MPa), \( \varepsilon \) is the vertical gob strain, \( b \) is the maximum strain parameter related to void ratio, and \( a \) is the gob stress when \( \varepsilon = b / 2 \) (MPa). Esterhuizen, Mark, and Murphy (2010a) had calibrated the hyperbolic equation by comparing the subsidence profile of super-critical panels obtained from FLAC3D model results with the subsidence profiles that were obtained from the Surface Deformation Prediction System (SDPS).

The gob parameters were selected in the same way as used by SDPS, in which the gob material is characterized by the ratio of the average thickness of strong and weak rocks in the immediate overburden. Rocks like shale and clay stone, that have a field scale Uniaxial Compressive Strength (UCS) of less than 40 MPa, were classified as weak rocks. Rocks like sandstone, siltstone and limestone, that have a field scale Uniaxial Compressive Strength (UCS) of more than 40 MPa, were classified as strong rocks. Esterhuizen, Mark, and Murphy (2010a) determined the values of the \( a \) parameter, for four gob types (weak, moderate, strong and very strong) along with the maximum vertical strain parameter to be 0.44, that corresponds to an initial bulking factor of 1.79 (Table 11). The modeling results obtained by Esterhuizen, Mark, and Murphy (2010a) are shown in Figure 93. The strong and moderate curves are identical to the laboratory results for sandstone and shale materials, as published by Pappas and Mark (1993).
Table 11 Parameters for modeling various gob types (modified from Esterhuizen, Mark, and Murphy, 2010a)

<table>
<thead>
<tr>
<th>Overburden Type</th>
<th>Ratio of strong: weak rocks</th>
<th>Parameter (a) (MPa)</th>
<th>Parameter (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak</td>
<td>25%</td>
<td>5.9</td>
<td>0.44</td>
</tr>
<tr>
<td>Moderate</td>
<td>35%</td>
<td>8.6</td>
<td>0.44</td>
</tr>
<tr>
<td>Strong</td>
<td>50%</td>
<td>12.8</td>
<td>0.44</td>
</tr>
<tr>
<td>Very Strong</td>
<td>65%</td>
<td>25.2</td>
<td>0.44</td>
</tr>
</tbody>
</table>
The average initial bulking factors used by Pappas and Mark (1993) were 1.80 for shale, 1.74 for sandstone, and 1.87 for strong sandstone. In a mine gob, the void ratio decreases with distance above the floor (Esterhuizen and Karacan, 2007). Therefore, Tulu, Esterhuizen, Mohamed, and Klemetti (2017) suggests that a value of 1.5 appears to be a good representation of average bulking factor.

Tulu, Esterhuizen, Mohamed, and Klemetti (2017) excluded the fractured rock above the caved zone and modeled the caved material in the gob. The parameters proposed by Esterhuizen, Mark, and Murphy (2010a) was modified by assuming that the gob was...
formed with an initial bulking factor of 1.5, corresponding to a maximum strain of 33%. Two types of gob parameters were suggested for strong overburden and weak overburden (Table 12). The FLAC3D modeling results obtained by Tulu, Esterhuizen, Mohamed, and Klemetti (2017) is shown in Figure 94.

Table 12 Parameters for modeling various gob types (modified from Tulu, Esterhuizen, Mohamed, and Klemetti, 2017).

<table>
<thead>
<tr>
<th>Overburden Type</th>
<th>a (MPa)</th>
<th>b</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak</td>
<td>3</td>
<td>0.33</td>
</tr>
<tr>
<td>Strong</td>
<td>7.24</td>
<td>0.33</td>
</tr>
</tbody>
</table>
Figure 94 The stress-strain curve for gob materials used in the FLAC3D model and results of laboratory tests on gob materials, after Pappas and Mark (1993) (Tulu, Esterhuizen, Mohamed, and Klemetti, 2017).

For the purpose of this research, the parameters for a strong overburden type, as suggested by Tulu, Esterhuizen, Mohamed, and Klemetti, (2017) is used (Table 12). Figure 95 shows the FLAC3D model results for the gob elements used for a strong overburden type.
5.5 Overburden Response Modeling

In case of full extraction mining like that of a longwall panel, the overburden will settle on to the gob, leading to the redistribution of stresses. The subsidence or collapse of the overburden has a significant impact on this redistribution of stresses. The amount of stress redistribution is a function of gob stiffness, overburden stiffness and mining geometry. A stiffer overburden stratum will lead to the formation of a stable pressure arch over the mined-out area, causing higher stresses in the unmined coal and lower stresses in the gob material. Therefore, it is imperative that while modeling overburden materials, the overburden properties are depicted as accurately as possible (Esterhuizen, Mark, and Murphy, 2010a).

Esterhuizen, Mark, and Murphy (2010a) had published a list of suggested properties for modeling large-scale coal measure rocks in the United States (Table 13). These properties
can be used for modeling panel scale models. Tulu, Esterhuizen, Mohamed, and Klemetti (2017) made some modifications to that data (Table 14).
Table 13 Representative rock properties (Esterhuizen, Mark, and Murphy, 2010a)

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Laboratory sample</th>
<th>In situ rock material</th>
<th>In situ bedding planes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UCS (MPa)</td>
<td>Elastic modulus (GPa)</td>
<td>Friction angle (deg)</td>
</tr>
<tr>
<td>Limestone 1</td>
<td>140</td>
<td>40</td>
<td>42</td>
</tr>
<tr>
<td>Limestone 2</td>
<td>100</td>
<td>35</td>
<td>42</td>
</tr>
<tr>
<td>Limestone 3</td>
<td>80</td>
<td>30</td>
<td>40</td>
</tr>
<tr>
<td>Sandstone 1</td>
<td>120</td>
<td>40</td>
<td>42</td>
</tr>
<tr>
<td>Sandstone 2</td>
<td>100</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Sandstone 3</td>
<td>80</td>
<td>35</td>
<td>37</td>
</tr>
<tr>
<td>Sandstone 4</td>
<td>60</td>
<td>35</td>
<td>35</td>
</tr>
<tr>
<td>Sandstone 5</td>
<td>40</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>Shale 1</td>
<td>80</td>
<td>25</td>
<td>32</td>
</tr>
<tr>
<td>Shale 2</td>
<td>60</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Shale 3</td>
<td>40</td>
<td>15</td>
<td>25</td>
</tr>
<tr>
<td>Shale 4</td>
<td>30</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>Shale 5</td>
<td>20</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>Shale 6</td>
<td>10</td>
<td>5</td>
<td>20</td>
</tr>
<tr>
<td>Shale 7</td>
<td>5</td>
<td>4</td>
<td>20</td>
</tr>
</tbody>
</table>
Table 14 Suggested intact rock properties (Tulu, Esterhuizen, Mohamed, and Klemetti, 2017)

<table>
<thead>
<tr>
<th>Type</th>
<th>UCS (MPa)</th>
<th>E (GPa)</th>
<th>Friction Angle (deg.)</th>
<th>Cohesion (MPa)</th>
<th>Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
<td>140</td>
<td>31.51</td>
<td>42</td>
<td>18.08</td>
<td>8.12</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>26.86</td>
<td>42</td>
<td>12.91</td>
<td>5.8</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>24.54</td>
<td>40</td>
<td>10.82</td>
<td>4.64</td>
</tr>
<tr>
<td>Sandstone</td>
<td>120</td>
<td>23.32</td>
<td>42</td>
<td>15.49</td>
<td>6.96</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>20.46</td>
<td>40</td>
<td>13.52</td>
<td>5.8</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>17.6</td>
<td>37</td>
<td>11.57</td>
<td>4.64</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>14.74</td>
<td>35</td>
<td>9.06</td>
<td>3.48</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>11.88</td>
<td>30</td>
<td>6.7</td>
<td>2.32</td>
</tr>
<tr>
<td>Shale</td>
<td>80</td>
<td>17.6</td>
<td>32</td>
<td>12.86</td>
<td>4.64</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>14.74</td>
<td>30</td>
<td>10.05</td>
<td>3.48</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>11.88</td>
<td>25</td>
<td>7.39</td>
<td>2.32</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>10.45</td>
<td>20</td>
<td>6.09</td>
<td>1.74</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>9.02</td>
<td>20</td>
<td>4.06</td>
<td>1.16</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>7.59</td>
<td>20</td>
<td>2.03</td>
<td>0.58</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>6.88</td>
<td>20</td>
<td>1.02</td>
<td>1.29</td>
</tr>
</tbody>
</table>

The UCS values as shown in Table 13 and Table 14 are the laboratory scale values. This value can be multiplied with 0.58 to get the field scale values for UCS as shown in the equation below (Esterhuizen, Mark, and Murphy, 2010a; Hoek and Brown, 1980).

\[ UCS_{field} = UCS \times 0.58 \]
Tulu, Esterhuizen, Mohamed, and Klemetti (2017) suggested the following equation to determine the elastic modulus (E) for sandstone and shale. This equation was obtained from the regression analysis of a large number of UCS tests. The $UCS$ is the laboratory scale value in MPa and the resulting elastic modulus will be in GPa.

$$E = 0.143 \times UCS + 6.16$$

The overburden and floor strata are modeled using elastic material model (Rashed, Mohamed, Gearhart, and Esterhuizen, 2019). The Young’s modulus is determined using Table 14. The Poisson’s ratio is kept as 0.25 for each stratigraphic layer in the overburden and floor.

The interfaces between each geological layer is modeled using the interface elements logic available in $FLAC3D$. Table 15 gives the list of interface elements properties used in the panel scale models.

**Table 15 Interface elements properties used in panel scale models.**

<table>
<thead>
<tr>
<th>Stiffness-normal</th>
<th>5000 MPa/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stiffness-shear</td>
<td>500 MPa/m</td>
</tr>
<tr>
<td>Friction</td>
<td>25 degrees</td>
</tr>
<tr>
<td>Cohesion</td>
<td>0.57 MPa/m</td>
</tr>
<tr>
<td>Cohesion-residual</td>
<td>0.2 MPa/m</td>
</tr>
<tr>
<td>Friction-residual</td>
<td>14 degrees</td>
</tr>
<tr>
<td>Tension</td>
<td>0.11 MPa/m</td>
</tr>
<tr>
<td>Tension-residual</td>
<td>0.07 MPa/m</td>
</tr>
</tbody>
</table>
5.6 Insitu Stresses

The pre-mining vertical stresses in the model is generated due to gravity. The data in Table 16 is used to generate the pre-mining vertical stresses.

Table 16 Data used to generate gravity stresses in panel scale models.

<table>
<thead>
<tr>
<th>Acceleration due to Gravity</th>
<th>9.8 m/s²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone density</td>
<td>2400 kg/m³</td>
</tr>
<tr>
<td>Shale density</td>
<td>2600 kg/m³</td>
</tr>
<tr>
<td>Coal density</td>
<td>2000 kg/m³</td>
</tr>
</tbody>
</table>

Horizontal stresses are normally a function of overburden depth. High horizontal stresses are developed due to the tectonic activities caused by the movements of the North American Plates (Zoback and Zoback, 1989). High horizontal stresses are higher in stiffer strata than in softer strata (Dolinar, 2003). Therefore, the pre-mining horizontal stresses is a function of both the modulus of elasticity and overburden depth. Mark and Gadde (2008) suggested the following equations to calculate the maximum and minimum horizontal stresses in MPa units.

\[
\sigma_{h1} = 1.2\sigma_v + 2.6 + 0.003E
\]

\[
\sigma_{h2} = 1.2\sigma_v + 0.0015E
\]

In the above equations, \(E\) is the elastic modulus of the given geologic layer and \(\sigma_v\) is the vertical overburden stress. Esterhuizen, Mark, and Murphy (2010a) used this equation to model the high horizontal stresses.

The orientation and magnitude of the high horizontal stresses for the longwall mine, as measured in the strong sandstone roof, is shown in Figure 96. The maximum horizontal
stress is at an angle of 58 degrees from the North. For the purpose of modeling, it is assumed that the direction of horizontal stresses remains the same throughout the overburden depth. The values of maximum and minimum horizontal stresses are calculated for each geologic layer, using the above equations.

![Diagram of high horizontal stresses direction and magnitude in the strong sandstone overburden for the longwall mine (data provided by mine personnel).](image)

Figure 96 High horizontal stresses direction and magnitude in the strong sandstone overburden for the longwall mine (data provided by mine personnel).

The orientation of the high horizontal stresses (principal stresses) were different from the co-ordinate system used for modeling in FLAC3D (Figure 97). Therefore, stress transformation is done using Mohr’s circle, such that the co-ordinate system of the horizontal stresses are aligned to the co-ordinate system used to develop the FLAC3D models (Figure 98). It can be noticed that a shear stress component is introduced due to this transformation.
Figure 97 High horizontal stresses direction with respect to the co-ordinate system used in *FLAC3D*. 
Figure 98 High horizontal stresses direction and magnitude after stress transformation, to align with the co-ordinate system used in FLAC3D models.

5.7 Panel Scale Modeling

The longwall coalmine is located in the Pocahontas No.3 coal seam at a depth of 1400-2400 feet. The longwall panels are 700 feet wide and 10,000 feet long. The gate road consists of yield-abutment-yield system. The abutment pillars are 430 feet long, and the yield pillars are 150 feet long. The average mining height is 6.6 feet.

The Pocahontas No. 3 seam is associated with thickly to massively bedded sandstones, with small lenses of shale in the overburden. The geology above the seam consists of silty dark shale. Above this shale, there is sandstone bed that is named as “Sandstone 1” by the mine. Above this sandstone bed is a shale parting. Above this shale layer is a sandstone bed named as “Sandstone 2”. Table 17 shows the measured strength parameters of the rocks near the coal seam, as measured using an extensive testing program conducted by the mine company.
Table 17 Material properties of the rocks near the coal seam (data provided by mine personnel).

<table>
<thead>
<tr>
<th>Height / Depth Above / Below Below of Coal</th>
<th>ANISOTROPY of Coal</th>
<th>Strength</th>
<th>DIA METRAL PLI-BASED PLI-BASED Strength</th>
<th>Strength</th>
<th>Strength</th>
<th>Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top / Bottom of sample</td>
<td>THICKNESS</td>
<td>LITHOLOGY</td>
<td>UNIAXIAL</td>
<td>UNIAXIAL</td>
<td>UNIAXIAL</td>
<td>Axial/Diametral</td>
</tr>
<tr>
<td>(ft.)</td>
<td>(ft.)</td>
<td></td>
<td>STRENGTH</td>
<td>STRENGTH</td>
<td>STRENGTH</td>
<td></td>
</tr>
<tr>
<td>73.15</td>
<td>16.40</td>
<td>Sandstone, Gray, W/S</td>
<td>20,400</td>
<td>15,100</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td>63.35</td>
<td>9.80</td>
<td>Sandstone, Gray</td>
<td>20,200</td>
<td>16,400</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>63.35</td>
<td>9.80</td>
<td>Hard Sandstone</td>
<td>15,900</td>
<td>15,300</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>57.15</td>
<td>4.15</td>
<td>Hard Sandstone</td>
<td>15,400</td>
<td>15,400</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>52.45</td>
<td>0.70</td>
<td>Sandstone</td>
<td>15,500</td>
<td>9,800</td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>46.40</td>
<td>6.05</td>
<td>Fireclay, Gray</td>
<td>11,000</td>
<td>1,000</td>
<td>11.0</td>
<td></td>
</tr>
<tr>
<td>42.05</td>
<td>4.35</td>
<td>Sandy Fireclay, Dark</td>
<td>1,900</td>
<td>1,300</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>34.15</td>
<td>7.90</td>
<td>Shale W/S, Stk, Drk</td>
<td>22,800</td>
<td>2,500</td>
<td>9.1</td>
<td></td>
</tr>
<tr>
<td>33.00</td>
<td>1.15</td>
<td>Sandy Fireclay, Dark</td>
<td>11,600</td>
<td>600</td>
<td>19.3</td>
<td></td>
</tr>
<tr>
<td>29.35</td>
<td>3.65</td>
<td>Shale W/S, Stk, Drk</td>
<td>21,600</td>
<td>6,900</td>
<td>3.1</td>
<td></td>
</tr>
<tr>
<td>28.70</td>
<td>0.65</td>
<td>Shale, Dark Gray</td>
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<td>2,800</td>
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The panel scale model uses the equivalent elements technique (using Hoek-Brown material properties to model coal material).
5.7.1 Panel scale model using the equivalent elements technique (using Hoek-Brown constitutive model)

The panel scale model is created such that there are 93 geologic layers in the roof and 8 geologic layers in the floor (Figure 99). The height of the overburden from the coal seam is 2350 feet. The depth of the floor from the coal seam is 500 feet. The total length of the model is 12400 feet and the total width of the model is 3200 feet. The total extent of the model is such as to include the dimensions of two longwall panels and the associated development entries. The zones are more densified closer to the coal seam and less densified farther away. The zones were densified to maintain proper aspect ratio.

![Figure 99 First panel scale model showing all geologic layers.](image)

Then the development entries and longwall panels were created to suit the height of the equivalent elements (Figure 100). The abutment pillars, yield pillars, coal panels, and non-mined coal material were created using the solid equivalent elements, corner equivalent elements, and edge equivalent elements (Figure 101).
Figure 100 First panel scale model showing the development entries and longwall panel in the coal seam (overburden is hidden).

Figure 101 First panel scale model showing the use of corner, edge and solid equivalent elements to model coal seam (overburden is hidden).
The surface of the longwall mine is not near horizontal. There are surface features like hills and valleys over the excavated longwall panel, such that the elevation difference exceeds over 800 feet. Thus, it is necessary to create the surface topography in the FLAC3D model. Therefore, the surface map of the mine is imported into the model (Figure 102) and the surface topography created (Figure 103). This is a major difference from the FLAC3D modeling approach used by Esterhuizen, Mark, and Murphy, (2010a) and Tulu, Esterhuizen, Mohamed, and Klemetti (2017).

Figure 102 First panel scale model showing the imported surface map in FLAC3D.
Figure 103 First panel scale model showing the created surface topography.

Next step is to create the interfaces between geologic layers (Figure 104). Then the material properties for each geologic layer is provided (Figure 105 and Figure 106).
Figure 104 First panel scale model showing the interface elements created between each geologic layer.

Figure 105 First panel scale model showing the Young’s modulus for each geologic layer.
Figure 106 First panel scale model showing the densities for each geologic layer.

The bottom of the model is fixed in the z-direction. The four sides of the model were given roller boundary conditions such that they can move only in the vertical direction. The model is then solved for insitu stresses (Figure 107 and Figure 108).
Figure 107 First panel scale model showing the contours for in situ stresses in z-direction.

Figure 108 First panel scale model showing the in situ principal stresses directions.

After modeling panel scale model for the pre-mining stress state, the development entries were excavated. Figure 109 and Figure 110 shows the vertical stresses contours due to
development excavation. Figure 111 and Figure 112 shows the vertical displacements contours due to development excavations.

Figure 109 First panel scale model showing the contours for zz-stress due to development excavation.
Figure 110 First panel scale model showing the zone displacement vectors due to development excavation (zones are hidden).

Figure 111 First panel scale model showing the contours for z-displacement due to development excavation.
Figure 112 First panel scale cross-section model showing the contours for z-displacement due to development excavation.

The next step is to excavate longwall panel 5 and replace the panel with gob elements. Figure 113 and Figure 114 shows the vertical stresses contours due to subsidence over panel 5. Figure 115 and Figure 116 shows the vertical displacements contours due to subsidence over panel 5.
Figure 113 First panel scale model showing the \(zz\)-stress contours due to subsidence over panel 5.

Figure 114 First panel scale cross-section model showing the \(zz\)-stress contours due to subsidence over panel 5.
Figure 115 First panel scale model showing the z-displacement contours due to subsidence over panel 5.

Figure 116 First panel scale cross-section model showing the z-displacement contours due to subsidence over panel 5.
Then the longwall panel 6 is excavated and replaced with gob elements. Figure 113 and Figure 114 shows the vertical stresses contours due to subsidence over panel 5 and panel 6. Figure 115 and Figure 116 shows the vertical displacements contours due to subsidence over panel 5 and panel 6.

Figure 117 First panel scale model showing the zz-stress contours due to subsidence over panel 5 and panel 6.
Figure 118 First panel scale cross-section model showing the zz-stress contours due to subsidence over panel 5 and panel 6.

Figure 119 First panel scale model showing the z-displacement contours due to subsidence over panel 5 and panel 6.
Figure 120 First panel scale cross-section model showing the z-displacement contours due to subsidence over panel 5 and panel 6.

5.8 Calibrating Panel Scale Models

5.8.1 Subsidence prediction using Surface Deformation Prediction System (SDPS)

The Surface Deformation Prediction System (SDPS) is a software package widely used in the US mining industry for subsidence prediction. This software uses the well-established and accepted influence function method for subsidence prediction. The influence function calculates the final surface deformation based on the subsidence engineering parameters like, the angle of influence, the supercritical subsidence factor, and the edge effect offset distance (Agioutantis and Karmis, 2013). Moreover, the SDPS software has been tested extensively in numerous case studies (VPI&SU, 1987; Karmis, Jarosz, and Agioutantis, 1989; Agioutantis and Karmis, 2002; Karmis and Agioutantis, 2004).
The SDPS software package is used to predict the subsidence over both the longwall panels. Figure 121 to Figure 123 shows the steps to using the SDPS software for this research.

Figure 121 Displaying the geometry of the longwall panels in SDPS.
Figure 122 Calculating subsidence in SDPS.
5.8.2 Calibrating the panel scale model with SDPS results

In Figure 124, the distance from 400 to 1250 feet corresponds to longwall Panel 5E, and the distance from 1250 to 2100 feet corresponds to longwall Panel 6E. When comparing the subsidence profile obtained from FLAC3D to that of SDPS, it can be seen that the subsidence over Panel 6E is very similar. This indicates that the large scale stiffness and deformation of the overburden is modeled with good accuracy. However, the subsidence profile over Panel 5E does not match exactly. This may be due to mining over previous panels that have not been modelled in either software packages.
Figure 124 Comparison of subsidence profiles (longitudinally across both longwall panels) obtained from SDPS and panel scale modeling in FLAC3D.
6 RESULTS AND DISCUSSION

6.1 Determining the Support Reaction Curve for the Abutment Pillar

For the purpose of determining the Support Reaction Curve, two fully discretized models of an abutment pillar are created, with a total model height of 6 meters; with 2 meters of roof strata, 2 meters of coal seam, and 2 meters of floor strata. The model is 134 meters long and 48 meters wide. The extraction ratio for this pillar is 5.59%. The material properties for the roof, floor and interface elements is kept the same as used to model the quarter pillars, as suggested by Esterhuizen, Mark, and Murphy (2010a). The zone sizes used for this model is 0.33m (0.33 x 0.33 x 0.33). This model consists of 1,022,544 zones. The increasing stress state on the model is simulated by imparting a downward velocity to the top surface of the model, while fixing the lower boundary and constraining all the four sides of the model (only the roof and floor rock is constrained, while the excavated coal layer had a free face).

The Support Reaction Curve is same as the average stress-deformation curve for the abutment pillar model. The average stress-deformation response of the abutment pillar model is obtained by recording the average stress at the base of the model and the relative deformation between two points located vertically at the top and bottom of the model. Since the bottom of the model is fixed, the deformation will be the total vertical displacement of the top surface.

The first abutment pillar is modeled with the calibrated Hoek-Brown parameters for simulating coal material (Table 6). The Support Reaction Curve for the first abutment pillar is shown in Figure 125.
Figure 125 Support Reaction Curve for the first Abutment Pillar (coal material modeled using Hoek-Brown parameters).

The second abutment pillar is modeled with the calibrated Coal Mass Model parameters for simulating coal material (Table 10). The Support Reaction Curve for the second abutment pillar is shown in Figure 126.
6.1.1 Results and Discussion

The Support Reaction Curve for the first abutment pillar model shows that the average stress developed in the pillar is a little above 80 MPa for a roof displacement of 0.08 m (Figure 125). The Support Reaction Curve for the second abutment pillar model shows that the average stress developed in the pillar is a little below 80 MPa for a roof displacement of 0.08 m (Figure 126). In case of both the models, the abutment pillar does not yield until 80 MPa, however the stress-deformation graph for the second abutment pillar model shows a slight downward bending at higher stress values.

It is expected that the abutment pillar, for a width to height ratio of 16, will behave as strain hardening. For the purpose of this research the Ground Reaction Curve obtained from the second abutment pillar model (using Coal Mass Model to simulate coal material) is assumed to be more accurate.
6.2 Determining the Ground Reaction Curves for the Overburden

The Ground Reaction Curve is generated in the panel scale $FLAC3D$ models, by gradually reducing the average stress (over 10 cycles) applied by the instrumented abutment pillar, and by determining the corresponding roof convergence at the same location (Figure 127). However, obtaining the Ground Reaction Curve for the overburden is more complex than determining the Support Reaction Curve for the supports.

![Figure 127 Conceptual representation of replacing the abutment pillar (a) with internal pressure (b) to determine the Ground Reaction Curve for the overburden.](image)

The first issue is deciding the location in the panel for which the curve has to be obtained. The Ground Reaction Curve represents the internal pressure-displacement curve at every point on the excavation surface. The gate road in a longwall mine runs along the full length of the panel. Therefore, the Ground Reaction Curve can be determined for many locations. Typically, it will be prudent to get the Ground Reaction Curve for a location that should experience the maximum geomechanical effects (like the top geometric center of the
For the purpose of this research, the Ground Reaction Curve is determined for the location of the instrumented abutment pillar (Figure 128). Due to the size of the abutment pillar, the grid point co-ordinate of the top geometric center of the instrumented abutment pillar is selected.

Figure 128 Ground Reaction Curve is determined for the location of the instrumented abutment pillar (overburden is hidden).

The next issue is deciding the various loading cycles for which the Ground Reaction Curves has to be determined. An independent Ground Reaction Curve should be obtained for each loading cycle (Ray, Agioutantis, and Kaklis, 2019). In case of a longwall mine the loading cycles experienced by the gate road pillars is very complex. For the purpose of this research, a total of 5 loading cycles is considered. Past research has suggested that the Ground Reaction Curve for the overburden can be determined numerically using either a two or three-dimensional model of the overburden (Damjanac, Pierce, and Board, 2014; and Esterhuizen, Mark, and Murphy, 2010b).
6.2.1 Determining the Ground Reaction Curve for two-dimensional overburden model

To estimate the Ground Reaction Curve for a two-dimensional overburden, the stress-displacement response of a slice of overburden (Figure 129), over the instrumented pillar, is numerically generated for different stages of longwall mining.

Figure 129 Slice of overburden used to determine the Ground Reaction Curve for a cross-section of the overburden over the instrumented pillar (coal seam and floor is hidden).

The First loading cycle occurs due to development mining. After the gate road entries is developed, the overburden load above the abutment pillar is distributed to the adjacent longwall coal panels (Panel 5E coal and Panel 6E coal), as shown conceptually by the pressure arch in Figure 130. For the First loading cycle, the pressure arch would be the smallest in size.
Figure 130 Conceptual diagram for First loading cycle due to development mining (pressure arch represents the distribution of load in two-dimensional model).

The Ground Reaction Curve, determined from two-dimensional overburden model, due to First loading cycle is shown in Figure 131 as GRC1. The GRC1 starts from the peak stress value of 15.5 MPa for no convergence, to the lowest stress value of 1.55 MPa over a convergence of 0.066 m.
Figure 131 Ground Reaction Curve for two-dimensional model due to First loading cycle.

The second loading cycle occurs when panel 5E is extracted (close to the instrumented abutment pillar). After panel 5E is mined, the immediate overburden collapses into the gob region (close to the instrumented abutment pillar) but has not started taking load. Thus, the overburden is hanging over the newly formed gob. The overburden load above the abutment pillar is distributed to panel 6E coal and chain pillars between panel 5E and panel 4E, as shown conceptually by the pressure arch in Figure 132. It should be noted that the pressure arch would increase in size, during the second loading cycle.
Figure 132 Conceptual diagram for Second loading cycle due to development and extraction of Panel 5E (pressure arch represents the distribution of load in two-dimensional model).

The Ground Reaction Curve, determined from two-dimensional overburden model, due to Second loading cycle is shown in Figure 133 as GRC2. The GRC2 starts from the peak stress value of 23.4 MPa for no convergence, to the lowest stress value of 2.34 MPa over a convergence of 0.11 m. It can be noticed in Figure 133 that GRC2 is above GRC1, representing an increase in size of the pressure arch.
Figure 133 Ground Reaction Curve for two-dimensional model due to Second loading cycle (solid line compared with the dashed curves from earlier loading cycle).

The third loading cycle occurs when gob 5E is compacted (close to the instrumented abutment pillar). After panel 5E is mined, the immediate overburden collapses into the gob region, and after a critical width of the coal panel is extracted, the gob starts to compress and take load (close to the instrumented abutment pillar). Therefore, the overburden load above the abutment pillar is distributed over Panel 6E and Gob 5E, as shown conceptually by the pressure arch in Figure 134. It should be noted that the pressure arch would decrease in size, during the Third loading cycle.
Figure 134 Conceptual diagram for Third loading cycle due to development and compaction of Gob 5E (pressure arch represents the distribution of load in two-dimensional model).

The Ground Reaction Curve, determined from two-dimensional overburden model, due to the third loading cycle is shown in Figure 135 as GRC3. The GRC3 starts from the peak stress value of 22.8 MPa for no convergence, to the lowest stress value of 2.28 MPa over a convergence of 0.10 m. It can be noticed in Figure 135 that GRC3 is below GRC2, representing a decrease in size of the pressure arch.
Figure 135 Ground Reaction Curve for two-dimensional model due to Third loading cycle (solid line compared with the dashed curves from earlier loading cycles).

The fourth loading cycle occurs when Panel 6E is mined (close to the instrumented abutment pillar). After Panel 6E is mined, the immediate overburden collapses into the gob region (close to the instrumented abutment pillar) but has not started taking load. Thus, the overburden is hanging over the newly formed gob. The overburden load above the abutment pillar is distributed to Gob 6E and chain pillars beyond Panel 6E, as shown conceptually by the pressure arch in Figure 136. It should be noted that the pressure arch would increase in size, during the fourth loading cycle.
Figure 136 Conceptual diagram for Fourth loading cycle due to development, compaction of Gob 5E and, extraction of Panel 6E (pressure arch represents the distribution of load in two-dimensional model).

The Ground Reaction Curve, determined from two-dimensional overburden model, due to the fourth loading cycle is shown in Figure 137 as GRC4. The GRC4 starts from the peak stress value of 36.8 MPa for no convergence, to the lowest stress value of 3.68 MPa over a convergence of 0.19 m. It can be noticed in Figure 137 that GRC4 is far above GRC3, representing a big increase in size of the pressure arch.
The fifth loading cycle occurs when Gob 6E is compressed (close to the instrumented abutment pillar). After Panel 6E is mined, the immediate overburden collapses into the gob region, and after a critical width of the coal panel is extracted, the gob starts to compress and take load (close to the instrumented abutment pillar). Therefore, the overburden load above the abutment pillar is distributed over Gob 6E and Gob 5E, as shown conceptually by the pressure arch in Figure 138. It should be noted that the pressure arch would decrease in size, during the fifth loading cycle. The Ground Reaction Curve, determined from first panel scale model, due to the fifth loading cycle is shown in Figure 139.
Figure 138 Conceptual diagram for Fifth loading cycle due to development, compaction of Gob 5E, and compaction of Gob 6E (pressure arch represents the distribution of load in two-dimensional model).

The Ground Reaction Curve, determined from two-dimensional overburden model, due to Fifth loading cycle is shown in Figure 139 as GRC5. The GRC5 starts from the peak stress value of 35 MPa for no convergence, to the lowest stress value of 3.5 MPa over a convergence of 0.18 m. It can be noticed in Figure 139 that GRC5 is below GRC4, representing a decrease in size of the pressure arch.
Figure 139 Ground Reaction Curve for two-dimensional model due to Fifth loading cycle (solid line compared with the dashed curves from earlier loading cycles).

6.2.2 Determining the Ground Reaction Curve for three-dimensional overburden model

To estimate the Ground Reaction Curve for the full three-dimensional overburden, the stress-displacement response of the entire overburden, over both the longwall panels, is numerically generated for different stages of longwall mining (Figure 140).
Figure 140 Entire overburden spanning over both longwall panels used to determine the Ground Reaction Curve for three-dimensional model (coal seam and floor is hidden).

The First loading cycle occurs due to development mining. After the gate road entries is developed, the overburden load above the abutment pillar is distributed to the adjacent longwall coal panels (Panel 5E coal and Panel 6E coal), as shown conceptually by the pressure arch in Figure 141. For the First loading cycle, the pressure arch would be the smallest in size.
Figure 141 Conceptual diagram for First loading cycle due to development mining (pressure arch represents the distribution of load in three-dimensional model).

The Ground Reaction Curve, determined from three-dimensional overburden model, due to First loading cycle is shown in Figure 142 as GRC1. The GRC1 starts from the peak stress value of 15.7 MPa for no convergence, to the lowest stress value of 1.57 MPa over a convergence of 0.06 m. This curve is very similar to that obtained from the two-dimensional model (Figure 131).
The Second loading cycle occurs when panel 5E is extracted (close to the instrumented abutment pillar). After panel 5E is mined, the immediate overburden collapses into the gob region (close to the instrumented abutment pillar) but has not started taking load. Thus, the overburden is hanging over the newly formed gob. The overburden load is distributed on one side to panel 6E coal and on the other side to the chain pillars, compressed Gob5E, and the remaining panel 5E coal, as shown conceptually by the pressure arch in Figure 143. It should be noted that the pressure arch would increase in size, during the second loading cycle.
Figure 143 Conceptual diagram for Second loading cycle due to development and extraction of Panel 5E (pressure arch represents the distribution of load in three-dimensional model).

The Ground Reaction Curve, determined from three-dimensional overburden model, due to Second loading cycle is shown in Figure 144 as GRC2. The GRC2 starts from the peak stress value of 20.4 MPa for no convergence, to the lowest stress value of 2.04 MPa over a convergence of 0.079 m. It can be noticed in Figure 144 that GRC2 is above GRC1, representing an increase in size of the pressure arch. This curve has small variations in the maximum stress and maximum roof convergence as that obtained from the two-dimensional model (Figure 133).
The Third loading cycle occurs when gob 5E is compacted (close to the instrumented abutment pillar). After panel 5E is mined, the immediate overburden collapses into the gob region, and after a critical width of the coal panel is extracted, the gob starts to compress and take load. Therefore, the overburden load above the abutment pillar is distributed over Panel 6E coal and compressed Gob 5E, as shown conceptually by the pressure arch in Figure 145. It should be noted that the pressure arch would decrease in size laterally and increase in size vertically. However, during the Third loading cycle, the overall size of the pressure arch will increase for the three-dimensional model (contrary to the results obtained from two-dimensional model).
Figure 145 Conceptual diagram for Third loading cycle due to development and compaction of Gob 5E (pressure arch represents the distribution of load in three-dimensional model).

The Ground Reaction Curve, determined from three-dimensional overburden model, due to Third loading cycle is shown in Figure 146 as GRC3. The GRC3 starts from the peak stress value of 22.5 MPa for no convergence, to the lowest stress value of 2.25 MPa over a convergence of 0.087 m. This curve has similar maximum stress but an increased maximum roof convergence as that obtained from the two-dimensional model (Figure 135). It can be noticed in Figure 146 that GRC3 is above GRC2, representing an increase in size of the pressure arch.
Figure 146 Ground Reaction Curve for three-dimensional model due to Third loading cycle (solid line compared with the dashed curves from earlier loading cycles).

The Fourth loading cycle occurs when Panel 6E is mined (close to the instrumented abutment pillar). After Panel 6E is mined, the immediate overburden collapses into the gob region (close to the instrumented abutment pillar) but has not started taking load. Thus, the overburden is hanging over the newly formed gob. The overburden load is distributed on one side to compressed Gob5E and on the other side to the chain pillars, compressed Gob6E, and the remaining panel 6E coal, as shown conceptually by the pressure arch in Figure 147. It should be noted that the pressure arch would increase in size, during the fourth loading cycle.
Figure 147 Conceptual diagram for Fourth loading cycle due to development, compaction of Gob 5E and, extraction of Panel 6E (pressure arch represents the distribution of load in three-dimensional model).

The Ground Reaction Curve, determined from three-dimensional overburden model, due to Fourth loading cycle is shown in Figure 148 as GRC4. The GRC4 starts from the peak stress value of 29.1 MPa for no convergence, to the lowest stress value of 2.91 MPa over a convergence of 0.11 m. This curve has decreased maximum stress and decreased maximum roof convergence compared to that obtained from the two-dimensional model (Figure 137). It can also be noticed in Figure 148 that GRC4 is above GRC3, representing an increase in size of the pressure arch.
The Fifth loading cycle occurs when Gob 6E is compressed (close to the instrumented abutment pillar). After Panel 6E is mined, the immediate overburden collapses into the gob region, and after a critical width of the coal panel is extracted, the gob starts to compress and take load. Therefore, the overburden load above the abutment pillar is distributed over compressed Gob 6E and compressed Gob 5E, as shown conceptually by the pressure arch in Figure 149. It should be noted that again the pressure arch would decrease in size laterally and increase in size vertically. However, during the Fifth loading cycle, the overall size of the pressure arch will increase for the three-dimensional model (contrary to the results obtained from two-dimensional model).
Figure 149 Conceptual diagram for Fifth loading cycle due to development, compaction of Gob 5E, and compaction of Gob 6E (pressure arch represents the distribution of load in three-dimensional model).

The Ground Reaction Curve, determined from three-dimensional overburden model, due to Fifth loading cycle is shown in Figure 150 as GRC5. The GRC5 starts from the peak stress value of 35 MPa for no convergence, to the lowest stress value of 3.5 MPa over a convergence of 0.18 m. This curve has decreased maximum stress and decreased maximum roof convergence as that obtained from the two-dimensional model (Figure 139). It can be noticed in Figure 150 that GRC5 is above GRC4, representing an increase in size of the pressure arch.
Figure 150 Ground Reaction Curve for three-dimensional model due to Fifth loading cycle (solid line compared with the dashed curves from earlier loading cycles).

### 6.2.3 Results and Discussion

The five Ground Reaction Curves corresponding to five loading cycles in case of two-dimensional model is shown in Figure 151. The curve (GRC4) corresponding to the Fourth loading cycle has the highest value of stress and is above all other curves. The maximum value of stress is 36.8 MPa. It is computationally faster to determine the Ground Reaction Curves for a two-dimensional model, since a large portion of the overburden is excluded from the numerical computation.
Figure 151 Five Ground Reaction Curves obtained for all five loading cycle in case of two-dimensional model of the overburden.

The five Ground Reaction Curves corresponding to the five loading cycles in case of three-dimensional model is shown in Figure 152. The curve (GRC5) corresponding to the Fifth loading cycle has the highest value of stress and is above all other curves. The maximum value of stress is 32.4 MPa. It is computationally intensive to determine the Ground Reaction Curves for a three-dimensional model, since the full overburden is included in the numerical computation.
Figure 152 Five Ground Reaction Curves obtained for all five loading cycle in case of three-dimensional model of the overburden.

By comparing Figure 151 and Figure 152, it can be deduced that the results obtained from the two-dimensional and three-dimensional overburden models are not identical. In case of the two-dimensional model, the abutment pillar will be loaded and unloaded during various loading cycles. This is not the case with the three-dimensional model where the abutment pillar is progressively loaded during subsequent loading cycles, as expected. This difference in results can be attributed to the fact that in case of the three-dimensional model, the modeling geometry is more accurate than the two-dimensional model, where a large portion of the overburden geometry is excluded.

Therefore, it can be concluded that the Ground Reaction Curves for the overburden of a longwall coalmine should be determined from a three-dimensional model. In addition, it can be noted that the pressure-arching concept can be used to explain the formation of different Ground Reaction Curves corresponding to various loading cycles.
6.3 Comparing the Ground Reaction Curves and Support Reaction Curves

The Support Reaction Curve is the graphical representation of average stress developed in the abutment pillar for a given value of roof convergence. The Ground Reacting Curve for the overburden gives the internal stress needed to hold it stable for a given amount of convergence. The ingenuity of the GRC concept is to uncouple the support behavior and overburden behavior to study them individually, and then compare their geomechanical responses together to determine the support-overburden interaction mechanism.

For the purpose of this study, the Support Reaction Curve for the instrumented abutment pillar has been numerically determined (Figure 126). In addition, the Ground Reaction Curves for the overburden at the abutment pillar location has been numerically estimated (Figure 152). The strata-pillar interaction mechanism at the given location of the instrumented abutment pillar is determined by superimposing the Support Reaction Curve over the Ground Reaction Curves (Figure 153).
Figure 153 Support Reaction Curve (SRC) for the abutment pillar superimposed over the Ground Reaction Curves (GRC) for the overburden.

In Figure 153 it can be seen that the Support Reaction Curve intersects all the five Ground Reaction Curves. Each intersecting point on the graph represents the static equilibrium between the overburden and abutment pillar. These points also give the values of stresses and displacements for each equilibrium condition. The static equilibrium between the overburden and abutment pillar is attained at a stress of 14 MPa and a roof convergence of 0.0072 m for the first loading cycle. The static equilibrium between the overburden and abutment pillar is attained at a stress of 18.6 MPa and a roof convergence of 0.0096 m for the second loading cycle. The static equilibrium between the overburden and abutment pillar is attained at a stress of 20.3 MPa and a roof convergence of 0.0104 m for the third loading cycle. The static equilibrium between the overburden and abutment pillar is attained at a stress of 26.1 MPa and a roof convergence of 0.0136 m for the fourth loading cycle. Finally, the static equilibrium between the overburden and abutment pillar is attained at a stress of 28.8 MPa and a roof convergence of 0.0152 m for the fifth loading cycle.
6.3.1 Results and Discussion

The longwall coalmine under study uses bleeder ventilation system. The gate roads have to be stable for the bleeder ventilation system to perform effectively. Thus, the chain pillars are designed to keep the gate roads stable. The chain pillars are designed using a yield-abutment-yield pillar design system. Therefore, the abutment pillar is designed not to fail.

In Figure 153 it can be seen that the Support Reaction Curve extends much above the five Ground Reaction Curves indicating that the abutment pillar can take a much higher load than all the loads applied on it during the five loading cycles due to longwall excavation. In summary, the abutment pillar remains stable for the full longwall mining cycle, as it is designed for.

6.4 Comparing the results with an established ground control software, ACPS

Analysis of Longwall Pillar Stability (ALPS) was the first empirical pillar design technique to consider the abutment loads from full panel extraction (Mark, 1990). It was calibrated using an extensive database of mining case histories. Later the Analysis of Retreat Mining Stability (ARMPS) and the Analysis of Multiple Seam Stability (AMSS) were released (Mark and Chase, 1997; and Mark, Chase, and Pappas, 2007). They were built upon a larger case history and used more sophisticated statistical methods.

The Analysis of Coal Pillar Stability (ACPS) integrates the three older software packages to make a single pillar design framework (Mark and Agioutantis, 2018). Figure 154 to Figure 156, shows the steps to calculate the stability factor for the chain pillar system in ACPS for the longwall coalmine under study.
Figure 154 Plan view of the chain pillars in the gate roads, as used in ACPS.
Figure 155 Pillar bearing capacities calculated by ACPS for a default value of CMRR and abutment angle.
6.4.1 Results and Discussion

The calculated stability factor for tailgate loading, as determined by ACPS is much smaller than the suggested safety factor of 1.44 (Figure 156). This difference in the value for stability factor can be attributed to the great depth of the mine, and the presence of strong sandstone layers in the overburden roof. It should be noted that for a coalmine at a depth of 2000 feet, mine regulators accept the stability factor less than 1, as determined by ACPS. Therefore, it can be concluded that the chain pillar system will be stable for the longwall coalmine, as is suggested by the GRC concept. Moreover, it can be argued that the GRC concept is a better tool to evaluate the stability of pillar systems for such cases, where the empirical data is sparse.

Figure 156 Stability factors calculated by ACPS for different loading conditions.
SUMMARY AND CONCLUSION

This research study endeavors to improve the understanding of the role of overburden mechanics in mine pillar design and global ground stability. More specifically, the study focuses to enhance the pillar-support design methodology allowing for the consideration of in-seam and overburden stress and displacements in the evaluation of underground pillar-support stability. This is achieved by re-evaluating the overburden response and pillar response for evaluating the stability of underground excavations. Originally developed within the civil tunneling industry, the GRC concept has provided a more integrated method of evaluating ground support. This approach, however, has found limited application in typical mine design and mine stability evaluation scenarios. By integrating our current knowledge and understanding of material and ground behaviors, this study looks to modernize the GRC concept for the improvement of underground stability analysis.

This study further describes the development of the GRC approach as an important index for pillar design methodology, and presents various technical arguments about the ability of this approach in bridging the limitations in numerical, analytical or experimental methods. Based on this philosophy, a conceptual flowsheet to improve pillar design is outlined. This flowsheet forms the basis for the instrumentation plan and numerical modelling plan to further support the proposed pillar design methodology.

To validate the proposed pillar design methodology, a deep longwall coalmine in Virginia has been instrumented with Borehole Pressure Cells and roof extensometers. The BPCs measured the stress changes in the abutment pillar, while the roof extensometers measured the adjacent roof deformation, due to longwall mining activity. Analysis of instrumentation data shows that the pressure readings from the BPCs are free from any noise, and hence can be used for calibration purposes. On the other hand, the extensometer readings were inconclusive.
In subsequent study, numerical simulations were performed in FLAC3D, to create pillar scale models and panel scale models. The modeling method is validated based on recent publications in this field. The pillar scale models are calibrated using the instrumentation results. The panel scale model is further calibrated using the subsidence profile obtained from SDPS. The purpose is to use the calibrated pillar scale models to determine the Support Reaction Curves, and the calibrated panel scale model to determine the Ground Reaction Curves. It is concluded from this exercise that numerical models of longwall coalmines could be validated using geomechanical data published in recent publications. It is further concluded that the numerical models of longwall coalmines can be calibrated using a minimal of in-mine instrumentation data and subsidence profile.

While determining the Ground Reaction Curves for the overburden, it is noticed that for a longwall coalmine, the response of the full three-dimensional model of the overburden must be accounted for, instead of a two-dimensional model. The strata-pillar interaction mechanism, at the instrumented abutment pillar, is studied by comparing the Ground Reaction Curves for the overburden and the Support Reaction Curves for the pillar. The study concluded that the abutment pillar is able to withstand the loads applied due to longwall mining operations, similar to what is observed in the field.

The dissertation establishes that a unified method of using a combination of numerical approach, analytical approach and data-driven approach can be devised for mine pillar design. The application of the GRC concept in mine pillar design will lead to a safer work environment at a longwall section. In addition, the study delivers a full three-dimensional view of the various loading cycles experienced by the chain pillars in a longwall coalmine. The proposed pillar design flowchart will help researchers to optimize the design of mine pillars and predict effects of any geomechanical changes made in the longwall coalmine.
7.1 Future Work

To improve and further validate the proposed pillar design methodology using the GRC concept, the research results indicate the need for future study in the following areas:

1. The instrumentation experience can be utilized to develop better instrumentation plans to produce better measurements in the future. This research can be repeated by calibrating the numerical models, based on instrumentation data form different panels and different pillar support systems in the same longwall coalmine. The same exercise can be used to calibrate the numerical models, based on instrumentation data form international case studies to further reinforce the pressure arching mechanism.

2. In order to extend this proposed pillar design method to other types of mine pillar design, similar studies can be done for different underground mines (e.g. stone mine, trona mine).

3. Since the numerical models were calibrated using the subsidence profile from SDPS, a further modification to this method can be to determine subsidence using satellite data, making the calibration process easier. Similarly, roof convergence can be measured using techniques like photogrammetry.

4. The proposed design method should be modified and tested to use the same Ground Reaction Curves to design natural support systems (mine pillars), and secondary support systems (cribs, can supports) as a unified support design methodology.
Appendix 1: List of Instruments used

Table 18 and Table 19 describes the instruments, their specifications. These instruments have been installed at the Longwall coalmine in Virginia.

Table 18 Description of instruments procured from Simplified Mine Instruments.

<table>
<thead>
<tr>
<th>Serial No.</th>
<th>SMI-Instrument Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>MHR-V1 (w/o rib box &amp; warning system)- 10 ft monitoring rod, hardware and 100' signal cable. Included 1 (5/8)&quot; Dia. Hole X 6</td>
</tr>
<tr>
<td>2</td>
<td>900' extra comm cable X 6</td>
</tr>
<tr>
<td>3</td>
<td>MHL installation toolkit X 1</td>
</tr>
<tr>
<td>4</td>
<td>Shipping and handling X 1</td>
</tr>
</tbody>
</table>
Table 19 Description of instruments procured from Geokon.

<table>
<thead>
<tr>
<th>Serial No.</th>
<th>Model Name</th>
<th>Geokon-Instrument Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3200-1-35MPA</td>
<td>Hydraulic Borehole Pressure Cell, 35 MPa range. With 4500HH-35MPa transducer attached (no dial gage required). Attach 10 feet of 3200-10E and 1000 feet of 02-250V6-E to each sensor.</td>
</tr>
<tr>
<td>2</td>
<td>4500HH-35MPA</td>
<td>VW Pressure Transducer, 7/16-20 MP female thread, 35 MPa. For use in conjunction with 3200-1-35MPA</td>
</tr>
<tr>
<td>3</td>
<td>02-250V6-E</td>
<td>Blue PVC cable, 0.252&quot;, 2 twisted pairs (2000 feet)</td>
</tr>
<tr>
<td>4</td>
<td>3200-10E</td>
<td>(1/8&quot;) high pressure</td>
</tr>
<tr>
<td>5</td>
<td>3200-6</td>
<td>Charge for Encapsulation of the cell (2 cells)</td>
</tr>
<tr>
<td>6</td>
<td>3200-3</td>
<td>Borehole Pressure Cell installation tool for boreholes to 50' depth, with carrying case</td>
</tr>
<tr>
<td>7</td>
<td>HRD-A1117</td>
<td>Quick Coupler, (3/8&quot;)NPTF. For power team hand pump</td>
</tr>
<tr>
<td>8</td>
<td>8032-16-1S</td>
<td>16x4 Channel Multiplexer w/surge protection and 16 strain relief cable entries</td>
</tr>
<tr>
<td>9</td>
<td>8032-7</td>
<td>Multiplexer Cable, standard length, 10' one end with bare leads</td>
</tr>
<tr>
<td>10</td>
<td>8600-2X</td>
<td>Modified CR6 Datalogger with integral MUX board. X= Internal multiplexer for use with (6) 2&quot; dia. BEI linear potentiometers Model #9615 (1.5K Ohm +5VDC Excitation). External multiplexer to be used with 4500HH-35MPA. 16 standard cable entries.</td>
</tr>
<tr>
<td>11</td>
<td>CSI-LOGGERNET-DD</td>
<td>CSI LoggerNet Software (download &amp; CD disc)</td>
</tr>
<tr>
<td>12</td>
<td>STARTPROG</td>
<td>Starter Datalogger Program</td>
</tr>
</tbody>
</table>
Appendix 2: Calibration sheets for BPCs

Figure 157 and Figure 158 shows the calibration sheets that were used to convert raw data from the BPCs to pressure readings.

Figure 157 Calibration sheet for BPC-1.
Vibrating Wire Pressure Transducer Calibration Report

Model Number: 450HH-35 MPa
Serial Number: 1619922
Date of Calibration: June 24, 2016
Temperature: 22.80 °C
Bareometric Pressure: 997.5 mbar

Technician: 

<table>
<thead>
<tr>
<th>Applied Pressure (MPa)</th>
<th>Gauge Reading</th>
<th>Gauge Reading</th>
<th>Average Reading</th>
<th>Calculated Pressure (Linear)</th>
<th>Error Linear (%)</th>
<th>Calculated Pressure (Polynomial)</th>
<th>Error Polynomial (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>8946</td>
<td>8946</td>
<td>8946</td>
<td>0.077</td>
<td>0.22</td>
<td>0.014</td>
<td>0.64</td>
</tr>
<tr>
<td>7.0</td>
<td>8140</td>
<td>8140</td>
<td>8141</td>
<td>6.955</td>
<td>-0.10</td>
<td>6.926</td>
<td>-0.07</td>
</tr>
<tr>
<td>14.0</td>
<td>7324</td>
<td>7323</td>
<td>7324</td>
<td>13.95</td>
<td>-0.14</td>
<td>13.80</td>
<td>0.00</td>
</tr>
<tr>
<td>21.0</td>
<td>6596</td>
<td>6594</td>
<td>6595</td>
<td>20.95</td>
<td>-0.14</td>
<td>20.80</td>
<td>0.00</td>
</tr>
<tr>
<td>28.0</td>
<td>5880</td>
<td>5880</td>
<td>5880</td>
<td>28.01</td>
<td>0.02</td>
<td>28.02</td>
<td>0.06</td>
</tr>
<tr>
<td>35.0</td>
<td>4857</td>
<td>4856</td>
<td>4857</td>
<td>35.05</td>
<td>0.14</td>
<td>34.99</td>
<td>-0.04</td>
</tr>
</tbody>
</table>

(MPa) Linear Gauge Factor (G): -0.008552 (MPa/digit)

Polynomial Gauge factors: A = -2.781E-06, B = -0.008168, C = 

Thermal Factor (K): 0.002803 (MPa/°C)

Calculate C by setting P=0 and R1 = initial field zero reading into the polynomial equation

(psi) Linear Gauge Factor (G): -1.240 (psi/digit)

Polynomial Gauge factors: A = -4.033E-06, B = -1.185, C =

Thermal Factor (K): 0.4066 (psi/°C)

Calculate C by setting P=0 and R1 = initial field zero reading into the polynomial equation

Calculated Pressures:

Linear: P = G(R - R0) + K(T - T0) + S

Polynomial: P = AR1 + BR1 + C + K(T - T0) + S

*Bareometric pressure expressed in MPa as per. Barometric compensation is not required with wired transducers.

Factory Zero Reading: 8901
Temperature: 21.5 °C
Barometer: 1009.9 mbar

Figure 158 Calibration sheet for BPC-2
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