

Proceedings **32nd International Conference on Ground Control in Mining**

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July 30 - August 1, 2013

Lakeview Scanticon Resort & Conference, Morgantown, WV, USA

32nd International Conference on Ground Control in Mining

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Syd S. Peng Ground Control in Mining Award

Dr. Keith Heasley, Award Committee Chair

Subject: Letter of Recommendation for Bruce K. Hebblewhite

Dear Syd S. Peng Ground Control in Mining Committee Members:

Aug. 12, 2012

I am honored to recommend Bruce K. Hebblewhite for the Syd S. Peng Ground Control in Mining Award. Dr. Hebblewhite has had a long and distinguished career conducting research and instructional activities in the field of mining engineering. His career began in the 1970's at the University of New South Wales and University of Newcastle upon Tyne where he studied mining engineering. Currently, Dr. Hebblewhite is the Executive Directory of Mining Education Australia and the Head of School and Research Director of the University of New South Wales's School of Mining Engineering. In between, Dr. Hebblewhite has produced an outstanding publication record that has resulted in numerous mining applications, especially in the ground control area.



Publication record in the field of ground control in mining:

Dr. Hebblewhite's publication record has spanned 35 years where he has authored 71 and co-authored 37 referred and invited papers. In addition, he has 14 authored and 12 co-authored published papers and reports. These publications have covered a range of topics, including the following:

- Underground pillar design
- Evaluation/determination of mining methods
- Investigation of shaft lining failure in evaporite strata
- Mine design for subsidence prevention and/or control
- Interpretation of underground geotechnical instrumentation monitoring (of numerous types, e.g., extensometry, convergence, absolute stress, stress change, crack propagation, tilt)
- Pillar extraction methods - design, review, compliance, auditing
- Roof control system evaluation
- Introduction of longwall mining in South African coal industry under massive roof conditions
- Introduction and implementation of roof bolting systems into UK underground coal industry
- Mine planning/feasibility studies
- Geotechnical assessment of anticipated mining conditions
- Mining productivity studies
- Mining industry competitiveness
- Review of overseas mining systems - equipment, technology, methods (UK, USA, Canada, Germany, China and South Africa)
- Training programs for mine workers with respect to pillar extraction mining methods, roof support
- Mine design, reinforcement and safety audits
- Mine design, planning and operations – system and performance audits
- Mine accident investigations, expert witness reports

Acceptance of applicant's research findings by the mining industry in the form of a case study or studies:

Dr. Hebblewhite has authored in excess of 400 confidential consulting reports. These reports have been prepared for clients in the UK, Australia, New Zealand, South Africa, China and North America and have influenced the following mining industry areas:

- Mining geomechanics
- Mine subsidence
- Mine design and planning
- Mining methods
- Mine safety and training
- Mine system audits and risk assessments

Demonstrated recognition of applicant by previous activities in the field of mining engineering related to ground control issues:

One way to demonstrate Dr. Hebblewhite's standing in the field of ground control is to examine positions held:

- Head of School and Research Director, (2003 to present), University of New South Wales, while Dr. Hebblewhite has been responsible for all aspects of the School, he has

retained leadership over two important research ventures: a) monitoring program to examine escarpment stability during undermining, and b) use of satellite imagery to develop 3D subsidence profiles of active longwall panels.

- Professor, Kenneth Finlay Chair of Rock Mechanics (1995-2003), University of New South Wales, teaching mining rock mechanics and related mining engineering subjects to undergraduate and postgraduate students
- Division Manager - Mining, (1983-1995), Australian Coal Industry Research Laboratories (ACIRL), underground and surface mining, geotechnical engineering, technical services and coal preparation
- Mining Engineer, (1977-1983), ACIRL, a) development of geotechnical research activities in relation to mine design using underground, laboratory and numerical methods, b) conducted research into mining methods for Greta Seam, Ellalong Colliery, NSW, and c) project engineer for roof control and numerical modeling stability investigations

Another way is to examine his record of expert panel activity:

- Expert Witness assisting Coroner – Coronial Inquest (2002-2003): 1999 Northparkes Mine Accident
- Member (2005 – continuing): Independent Expert Review Panel (Dendrobium Mine), NSW Dept. of Planning
- Expert Witness assisting Coroner – Coronial Inquest (2007): 2004 Sydney Cross City Tunnel Fatality
- Chair: 2007 Independent Expert Panel of Review into Impact of Mining in the Southern Coalfield of NSW (Dept of Planning & Dept of Primary Industry)
- External Examiner (2008 – 2011): Mineral Resources Engineering Program, Universiti Sains Malaysia (USM)
- Specialist Advisor (Mining) to the NSW Government Thirlmere Lakes Inquiry (2012) (Office of Environment and Heritage)

Professional Involvement:

Dr. Hebblewhite has been active in Australian and International professional societies including:

- Member - Australasian Institute of Mining and Metallurgy
- Member - Australian Geomechanics Society
- Member - International Society of Rock Mechanics, (President – Mining Interest Group (2004 – 2011))
- Council Member – International Society of Mining Professors, (President for 2008/09)
- Secretary-General - International Society of Mining Professors, (from July 2012)
- Executive Director – Mining Education Australia (July 2006 – Dec 2009)
- Member of the Scientific Advisory Board – Advanced Mining Technology Centre, University of Chile (appointed from May 2012)

Committee Memberships:

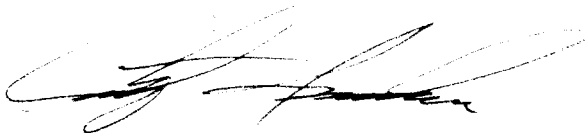
- Member (1978-1991) Underground Mining Technology Committee (formerly Thick Seam Mining Committee)

- Member (1984-1987) National Energy Research, Development and Demonstration Council (NERDDC) Technical Standing Committee No. 2 responsible for technical evaluation of proposals and project monitoring within the Program of Mine Site Technology
- Member (1985 - 1993) Editorial Board of The Australian Coal Journal
- Member (1991-1995) Standards Association of Australia, Sub-committee MN/1/5 - Coal Mining and Geology
- Member (1992-1994) Steering Committee, Coal Bed Methane Project, The Warren Centre for Advanced Engineering, The University of Sydney
- Alternate Member Underground Mining Sub-committee of the Queensland Mining Council (1993-1995) ACARP Research Committee
- Member (1998) NSW Department of Mineral Resources/RTA Advisory Management Committee for mining (Tower Colliery)
- Member (1998) I.E. Aust. (Sydney Branch) Engineering Excellence Awards (Buildings and Mineral Resource Projects Category)
- Member and I.E. Aust. (Sydney Branch) Engineering Excellence Awards (Minerals Chairman (1999) Resources Projects Category)
- Member (1997-2001) Editorial Board, The Australian Coal Review
- Member (1999 -present) Editorial Committee, International Ground Control in Mining Conference
- Member (2003 – present) Editorial and Organizing Committee: Australasian Ground Control in Mining Conference
- Member (2004 - present) Standards Australia, Sub-committee MN/1/5 - Coal Mining and Geology
- Member (2007 – present) Editorial and Organizing Committee: International Future Mining Conf. (UNSW/AusIMM)
- Member (2009 – present) Editorial and Organizing Committee: Australasian Ground Control in Mining Conf. (UNSW/AusIMM)

In closing, I had the distinct pleasure to work with Dr. Hebblewhite during his sabbatical last year. During that time I developed a deep respect for the depth and breathe of his ground control knowledge as well as his prior accomplishments in this area. I can think of no one who is more deserving of the Syd S. Peng Ground Control in Mining Award.

If you are in need of any additional information, please don't hesitate to call or write.

Sincerely,



Anthony Iannacchione, PhD, PE, PG
Associate Professor
Director, Mining Engineering Program

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Syd S. Peng Ground Control Scholarships

As a part of the Syd S. Peng Ground Control in Mining Award - an undergraduate and/or graduate scholarship, or scholarships, will be awarded annually to encourage the development of ground control engineers. The scholarship will be a minimum of \$5,000 (U.S.) and provided in a single payment to the recipient by January 1 (so the funds are available for the Spring Semester).

The 2012 scholarships were awarded and the recipients were recognized during the banquet at the Annual SME Meeting held in Denver, Colorado. The 2012 recipients were:

- Donovan Benton – Southern Illinois University
- Biao Qiu – West Virginia University
- Sean Warren – University of Nevada, Reno
- Xu Ma – Southern Illinois University



Pictured from left to right is Biao Qiu, Barb Arnold, Xu Ma, and Sean Warren.

Kazem Oraee Scholarship for Mining

The **Kazem Oraee Scholarship for Mining** award is to promote underground mining engineering and more specifically, the ground control discipline. Recent trends indicate that a shortage of competent ground control engineers may exist if this trend is allowed to continue. To help remedy the situation and promote the ground control engineering discipline, an undergraduate OR graduate scholarship, will be awarded annually at the International Conference on Ground Control in Mining (ICGCM) to encourage the development of ground control engineers. The scholarship recipient will receive \$5,000 (US) provided in a single payment and a commemorative certificate/statue.

This inaugural recipient for the **Kazem Oraee Scholarship for Mining** award was selected by the Oraee Family and goes to Behrooz Abbasi, who is a student at Southern Illinois University in Carbondale, IL, USA. Mr. Abbasi is currently working on his Ph.D. and is scheduled to graduate in December, 2014. He was selected from 28 well qualified candidates.



Professor Spearing accepting the award for Mr. Abassi at the 31st ICGCM.

32nd ICGCM – The Best of Ground Control in Mining



(From left to right) Jen Lu, Luke Stull and Marie Jaime (Consol Energy), Bob Kimutis (Alpha Natural Resources) Zara Hosseini (ESG Solutions) Syd Peng (WVU) Steve Tadolini (Minova International) Peter Zhang (Alpha Natural Resources) Essie Esterhuizen (NIOSH) Eric Westman (VPI)

The Best of Ground Control was held at the 2013 SME Annual Meeting & Exhibit on February 27, 2013, Denver, CO. This is a standing SME session at the Annual Meetings in the Coal & Energy Division. The six technical papers were selected from the 31st International Conference of Ground Control in Mining (ICGCM 2012). This was an exceptional line-up and very well attended at the conference. The papers are selected during the conference based on technical originality and merit and the quality of the presentation. In keeping with the original vision of Dr. Syd S. Peng for the conference, a diverse group of selections are sought from industry, academia, manufacturers, consultants, and government.

Mine Visit Courtesy of Consol Energy

A visit to one of Consol Energy's longwall mines gave attendees a chance to see a modern American longwall operation.



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How to Mitigate Coal Mine Bumps Through Understanding of the Violent Failure of Coal Specimens

Gamal Rashed, Graduate Research Assistant
Mining Engineering
West Virginia University
Morgantown, WV

ABSTRACT

This research is divided into two parts: the first part of this research focuses on understanding coal mine bump mechanisms, and the second part focuses on mitigating the violent failure associated with a bump. Many researchers believe that the coal bumps happen because of sudden loss of constraint between pillar/roof and pillar/floor or the hammering effect due to the breaking of thick, massive strata above the coal seam, creating a sudden impact load on the pillars and causing them to bump. However, very little work has been done to explore and understand the consequences of these proposed mechanisms for coal bumps. In the first part of this research, computer simulation of coal specimen behavior have been used to explore two issues: 1) How would an instantaneous loss of constraint or a small sudden impact load cause violent failure? 2) How does the strength of the coal specimen control coal mine bumps?

Eliminating or reducing the violent failure is the main objective of the second part of this research. The coal specimen needs to be softened either in the rib zone or the core zone and then its mode of failure studied to see whether it is violent or not. Instantaneous loss of constraint and the hammering effect are associated with sudden release of elastic energy, increase in the kinetic energy, and very rapid transient vertical stresses of the coal specimen. These are three of the major proposed mechanisms for bumps. The potential for violent failure increases with increasing the unconfined compressive strength, UCS, of the coal specimen. Failure of the rib zone is not the main factor for the violent failure of a coal specimen. The energy stored in the core zone is the key factor for the violent failure.

INTRODUCTION

Coal bumps are sudden, violent bursts of coal from a pillar or a block of coal, resulting in the whole pillar or the solid block of coal being ejected into the open entry (Peng, 2008). Because of the catastrophic nature of these sudden failures, a thorough understanding of the causes of failure and mitigating their occurrence are the main objectives of this research.

Several theories have been proposed for coal mine bumps (Babcock and Bickel, 1984; Holand, 1958b; Iannacchione and

Zelanko, 1994; Maleki, 1995; Peng, 2008; Rice, 1935). Two common causes for coal bumps receive the most attention among researchers and practitioners. The first cause was proposed by Holland (1958b): When a coal pillar is loaded close to its ultimate strength under constrained conditions, a sudden loss of interface friction would cause a coal bump. The second cause for bump was proposed by Rice (1935): It is known as a shock bump due to the breakage of a thick massive stratum or strata close to the coal seam, causing a great hammering effect similar to an impact load, resulting in a shock wave transmitted to the coal pillars underneath. Although these mechanisms were proposed many decades ago, very little work has been done to examine the impact of the first cause for the violent failure of coal pillar. To the authors' knowledge, almost no work has been done to examine the second cause. Moreover, the mechanisms of coal bumps and how energy stored in the coal is released are still not well understood. Understanding bump mechanisms is the main objective of the first part of this research.

An attempt has been made to reduce or eliminate the violent failure of coal specimens of different width to height ratios. Two hypotheses have been proposed for the violent failure of coal specimens. The first hypothesis is that the violent failure occurs because of sudden failure of the rib zone. Since coal material is brittle and the rib zone has the minimum confinement and constraint, instantaneous failure of the rib zone transfers transient stresses to the core zone which could exceed the final static strength, and violent failure takes place. The second hypothesis is that the violent failure occurs because of the core zone. Since the core zone has the highest confinement and constraint, it can store a lot of energy, which could be released violently at the moment of failure. To investigate which one of these hypotheses might be true, the coal specimens have been softened by drilling holes at specified location in the coal specimen.

THE EFFECT OF THE UNCONFINED COMPRESSIVE STRENGTH OF A COAL SPECIMEN ON THE POTENTIAL FOR VIOLENT FAILURE

The complete stress strain curve under uniaxial compression is shown in Figure 1. It can be divided into two parts: In the first part, OA represents the unfailed state, where an increase in stress is associated with an increase in the resistance of the rock. In

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the second part, AB represents the failing state, where a further increase in the strain is accompanied by a decrease in the load carrying capacity due to progressive degradation of the material stiffness. Hence the slope of the post peak curve indicates the rate of degradation of the material stiffness. A higher post peak slope means fast degradation of the coal specimen and fast crack propagation and, hence, brittle failure, while a lower post peak slope means slow degradation and, hence, ductile failure.

Therefore, the slope of the post peak of a complete stress strain curve is a good indicator of whether the coal specimen will fail violently or nonviolently.

tests were performed under a constant strain rate of 10-4 in/sec. Coal specimens of W/H ratios greater than 4 could not be tested because of the limiting load capacity of the MTS machine used in this research.

According to the laboratory test results shown in Table 1, it is obvious that the unconfined compressive strength, the modulus of elasticity, and the slope of the postpeak vary considerably within the same W/H ratio. The variation in both the strength and modulus of elasticity decreases with increasing width to height ratio of the specimen. A large value of the postpeak slope is not restricted to a

Table 1. Laboratory test results of coal specimens at different width to height ratios.

W/H ratio	Specimen #	UCS, psi	E, psi	Slope _{postpeak} , psi	UCS _{Avg} , psi	St. Dev. UCS	E _{Avg} , psi	St. Dev. E
2	1	5,728	323,190	466,733	5,853.5	1,291	304,768	66,186
	2	6,888	350,169	3,685,092				
	3	8,170	404,220	4,537,144				
	4	5,842	297,846	875,042				
	5	5,336	333,163	127,628				
	6	5,108	297,659	95,819				
	7	7,133	385,775	1,093,309				
	8	4,479	206,728	151,453				
	9	4,843	323,204	195,612				
	10	3,783	182,771	88,297				
	11	5,611	307,604	412,861				
	12	7,321	244,887	2,981,888				
3	1	5,257	202,611	139,780	6,445	1,071	254,752	31,610
	2	5,679	229,923	90,749				
	3	7,218	267,814	956,548				
	4	4,750	234,695	76,265				
	5	7,674	268,243	1,737,336				
	6	7,172	250,434	695,831				
	7	6,045	243,824	268,357				
	8	5,735	247,703	99,907				
	9	7,227	286,786	646,613				
	10	7,689	315,486	1,459,073				
4	1	8,591	251,536	196,851	8,783	669	224,594	23,066
	2	8,985	223,637	816,663				
	3	9,941	228,336	1,575,136				
	4	8,016	195,593	18,670				
	5	8,857	247,448	301,116				

Table 1 shows the unconfined compressive strength, the modulus of elasticity, and the slope of the post peak measured clockwise from the horizontal of the stress-strain curves. The modulus of elasticity was calculated at 50% of the ultimate strength of the coal specimens. The coal specimens have a square cross-section. The height of all specimens was kept constant at 1 in, while the width to height ratio of the test specimens ranged from 2 to 4. All

certain width to height ratio (i.e., violent and nonviolent failures could happen at any W/H ratio). Therefore, the width to height ratio may not be the best strategy to control violent failure. There is a strong positive correlation between the strength of the coal specimen and the slope of the postpeak of the stress-strain curve. Hence, among coal specimens of the same W/H ratio, the higher the strength, the higher the potential for violent failure. This is

not a new finding since Rice (1935) suggested that “structurally strong coal” not prone to crushing easily would favor bumps. There is also a positive correlation between the modulus of elasticity and the post peak slope of stress-strain curve at a width to height ratio equal to 2 and 3. At a width to height ratio equal to 4, the correlation coefficient between the modulus of elasticity and the post peak slope is insignificant. So it has no effect on whether the failure will be violent or not.

The Effect of the Sudden Loss of Friction on the Potential for Violent Failure

According to Cook (1963), Meikle (1965), and Prassetyo (2010), the strength of coal or rock decreases with decreasing interface friction. The constraint by interface friction provides the inherent strength of a pillar with supplemental strength, which increases with increasing the coefficient of friction. Babcock & Bickel (1984) proposed that the sudden loss of friction might cause violent failure, and most coals can be made to burst if stress and constraint are present. This finding was one step further than that of Holland (1958), who had first proposed the concept. However this finding did not answer important questions such as how the pillar was affected by sudden loss of friction, where the critical regions in the pillar are that are more susceptible to violent failure, and how energies change due to loss of constraint.

3D elastic models have been conducted using ABAQUS 6.10 (Dassault Systèmes, 2013). The elastic modulus, Poisson’s ratio, and the density of the modeled coal specimen are 250,000 psi, 0.25, and 84 lb/ft³, respectively. The purpose of these models is to examine the energy released and transient stress associated with sudden loss of constraint between the modeled coal specimen and rigid platens. To consider how a sudden release of elastic energy could cause a bump, it is assumed that the coal specimen is similar to a spring under compression. The more load is applied to compress the spring, the more elastic energy is stored in that spring. If the load is released from the spring instantaneously, the spring will rebound and elastic energy will be released suddenly. Losing constraint for the coal specimen is similar to releasing a load from a spring under compression. So the elastic energy stored in the coal specimen will be released instantaneously and violently.

Two coefficients of friction (0.25 and 0.10) and two W/H ratios (4 and 8) were used to examine the influence of different interfaces friction and W/H ratios on the violence of coal specimen failure during sudden loss of interface friction. The complete load history of computer simulation is divided into two stages. During the first stage, static analysis was performed. The load was ramped up gradually and uniformly by forcing the lower platen to move 0.1 in in the vertical direction. The initial value for coal/platens interface friction was either 0.25 or 0.1. The linear Coulomb friction model without limit for the interfacial shear stress was assumed. During the second stage of loading, the coal/platens interface friction in the whole contact surface was removed completely and suddenly from the initial value. The short-term response of the coal specimen to the induced transient stress resulting from losing constraint is a dynamic event. That is why ABAQUS implicit dynamic was used in the second stage of loading. The time period for that stage was 0.001 sec. The final load reached in the first stage was kept constant and propagated through the second stage.

Figure 3 shows the variation of the elastic strain energy with time at different coefficients of friction and W/H ratios. At time zero, 0% of the external load was applied, and at time1, 100% of the load was applied. During the first stage of loading, the elastic strain energy increases with increasing the applied load, while the kinetic energy is zero. As shown in Figure 3, the elastic strain energy increases with increasing friction and W/H ratios. During the second stage, there is a rapid energy change and a dynamic response associated with sudden loss of constraint. Examples for these transient effects are sudden release of elastic energy and sudden increase of kinetic energy. The non-zero value for the kinetic energy associated with losing constraint indicates that losing constraint is not a static process.

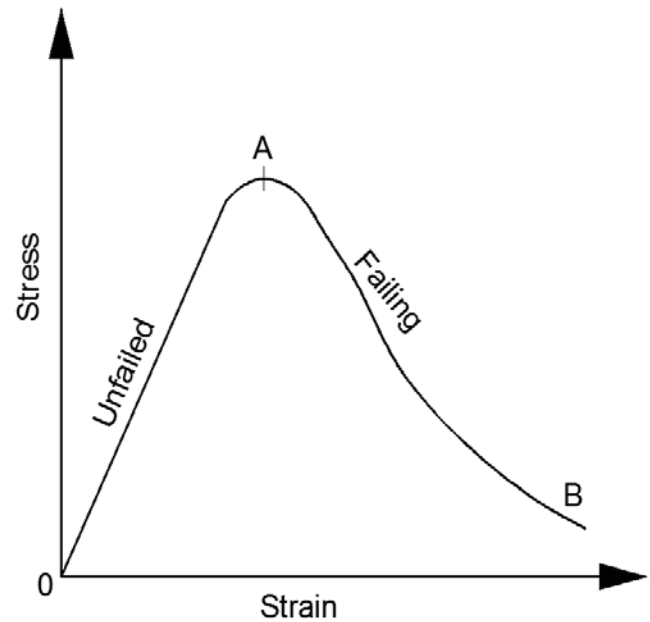


Figure 1. The complete stress strain curve under uniaxial compression.

By analogy, this kinetic energy would propel the coal particles to eject away from the coal pillar if bumps occurred. The amount of energy released, and the kinetic energy increased with increasing the coefficient of friction. Hence the degree of violence increases with increasing interface friction. Griffith (1921) realized that a crack in a body would not grow unless energy is released to overcome the energy needs of forming two new surfaces, the crack resistance. Therefore, both the released and kinetic energies are necessary for the occurrence of coal bumps. The released energy is consumed in the initiation and/or propagation of existing cracks inside the body, while the kinetic energy is providing the edges with sufficient energy for ejection from the pillar.

To get a complete picture about the adverse impact of sudden loss of interface friction, it is necessary to study how the vertical stress changes in the coal specimen before and after losing constraint. The vertical stress was studied at three specified paths of the modeled coal specimen of W/H ratio = 4, and coefficient of friction equal to 0.25, as shown in Figure 4A. The path 1-1 is located at the middle of the top surface where the vertical load was applied; the path 2-2 is located at the edges corners, where the front

and side surfaces meet, and path 3-3 is located at the middle of the front surface of the modeled coal specimen.

As shown in Figure 4, there is a very rapid change in the vertical stress due to sudden loss of interface friction. Some regions of the modeled coal specimen are subjected to transient concentrated stress that would cause violent failure when they exceed the final static strength, while other regions are subjected to stress relief. Figure 4B shows the variation of the vertical stress through path 1-1. Before losing the constraint, the maximum vertical stress occurs at the boundary (edges) and the center of the pillar. Since the strength of the central portion of the coal specimen is higher than the strength of the boundary, failure will commence at the pillar boundary and migrate towards the center (Wagner, 1980). After losing constraint, there is sudden rebound from maximum axial compression. The final vertical stress on the top surface is not the same at the edges and the center of the modeled coal specimen; the central portion is subjected to more vertical stress, which indicates that the constraint was not lost completely in the central portion because the time period proposed for losing constraint in the whole surface is not long enough. Unfortunately, increasing the time period changes the analysis from dynamic to static, (i.e., the dynamic response could not be captured if the time period is long). Figure 4C shows the variation of vertical stress before and after losing constraint through path 2-2. After losing constraint, the top and the bottom portions of the coal specimens near the platens are subjected to very high stress relief, 10 kpsi. While the middle portion of the vertical edge across path 2-2 is subjected to very high stress concentration, 6 kpsi. These transient changes in the stress state generate shock waves, which might cause violent failure if the coal specimen is loaded near its ultimate static strength. It is worth mentioning that, after losing constraint, the vertical stress is the same everywhere, which indicates that the edges can lose their constraint faster than the core. Figure 4D shows the vertical stress at section 3-3. It is quite obvious that this portion of the coal specimen is subjected to very high stress concentration, especially at the edges due to losing constraint. These edges will definitely fail.

Hence releasing the interface friction between the modeled coal specimen and the platens is similar to applying an impact load on the coal specimen. The moment the load is applied, the elastic energy will be maximum, and, when the specimen has rebounded from its maximum displacement, moving back in the opposite direction, the coal specimen relaxes, and strain energy stored in the coal specimen becomes available to perform work.

The Effect of the Sudden Impact Load on the Potential for Violent Failure

Rice (1935) proposed this mechanism and called it a shock bump. However, so far, no work has been done to advance this theory. Bump-prone coal seams are always overlain by a strong and stiff stratum or strata, such as sandstone or conglomerate (Campoli, 1987; Peng, 2008). Iannacchione stated that, among 95 bump sites studied, 86 sites had the presence of sandstone. In Utah, coal bumps occur in nearly all coal seams with thick sandstone in the overburden. In this part of this research, the effect of impact load on the potential for violent failure has been studied.

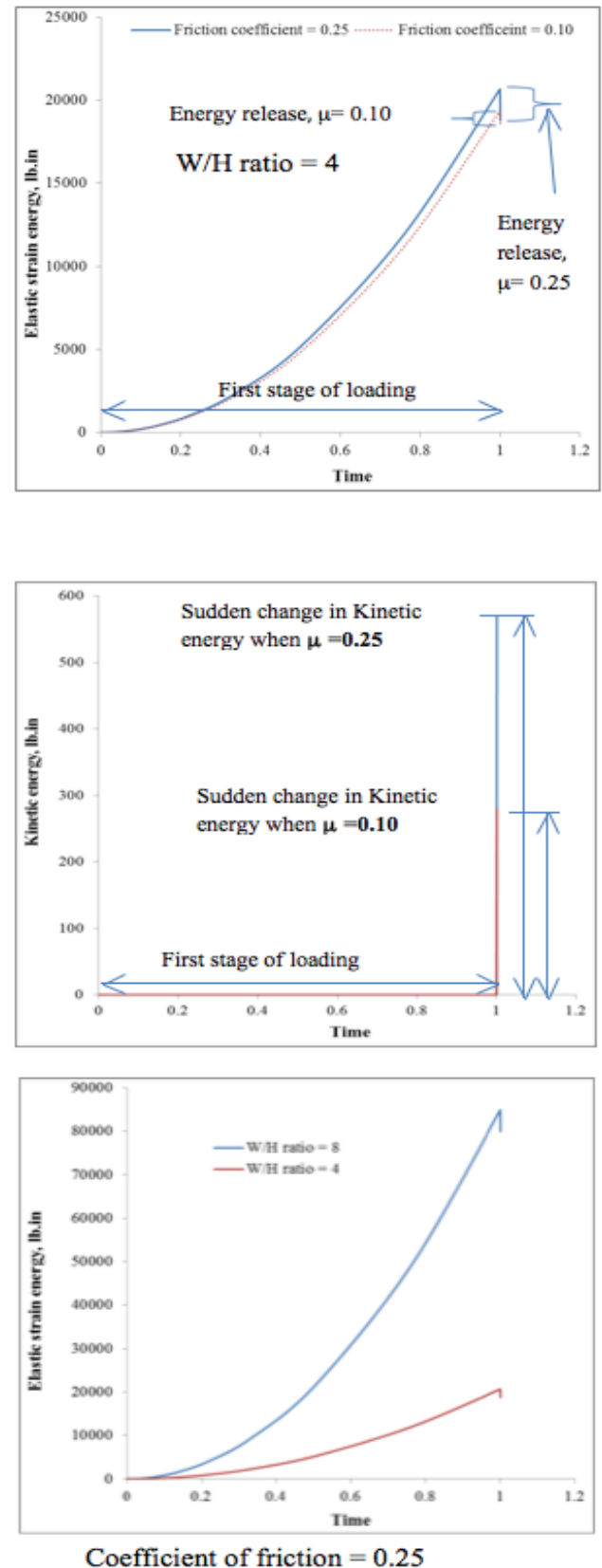


Figure 2. Variations of released and kinetic energies due to sudden loss of friction.

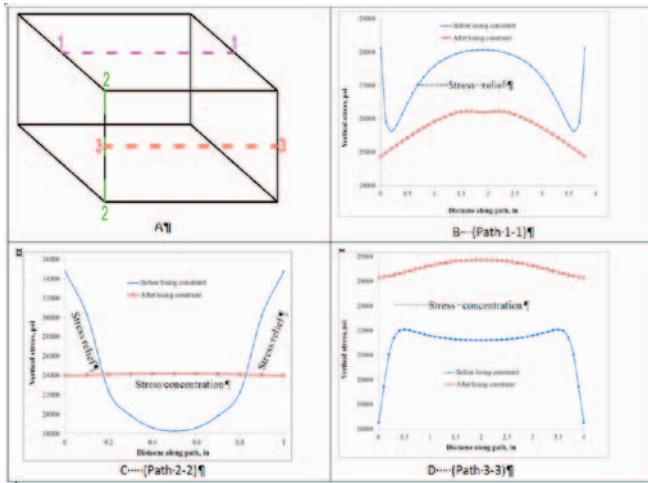


Figure 3. The variation of the vertical stress at different paths due to losing interface frictions.

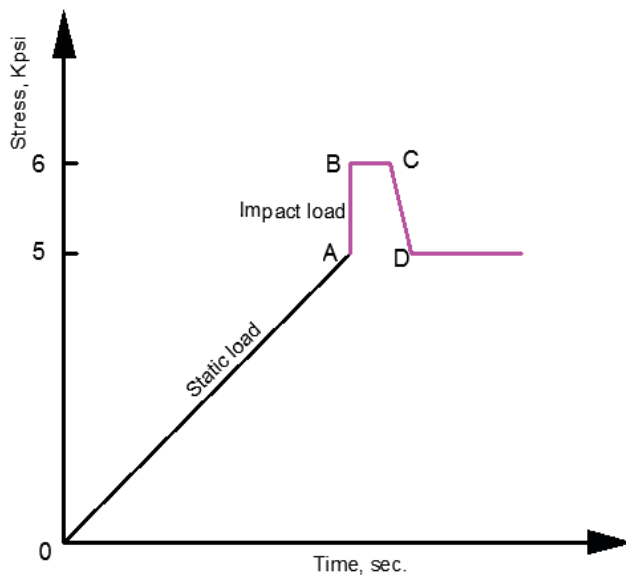


Figure 4. The applied load history of modeling from the start to the end of the analysis.

A 3D elastic model has been conducted to study the effect of impact load on released and kinetic energies for the coal specimen with a width to height ratio equal to 4 and the coefficient of friction between the coal specimen and the platen equal to 0.25. Figure 5 shows the load history from the start to the end of the analysis. Explicit dynamics analysis is required to analyze high-speed dynamic events, such as impact loads. In dynamic analysis, the load is time dependent. The boundary conditions are similar to the former models used in the previous section, where the upper rigid platen is fixed, and the lower platen can move only in the vertical direction.

The load history can be divided into two stages. In the first stage where static analysis was adopted, the overburden load was applied, and the load was ramped smoothly up to point A, from 0 to 5,000 psi. In the second stage of loading, an impact load = 1,000 psi, from

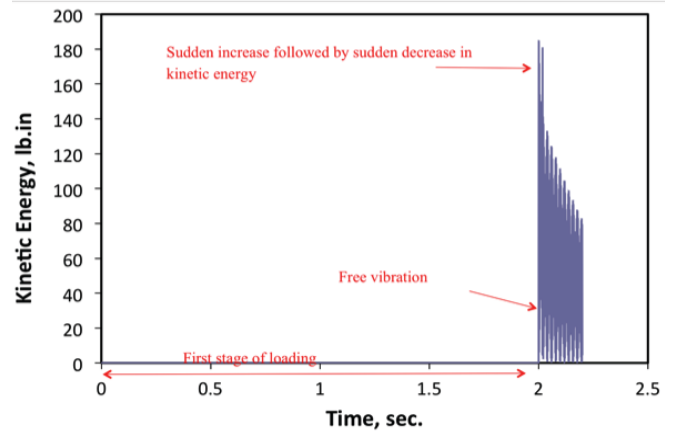
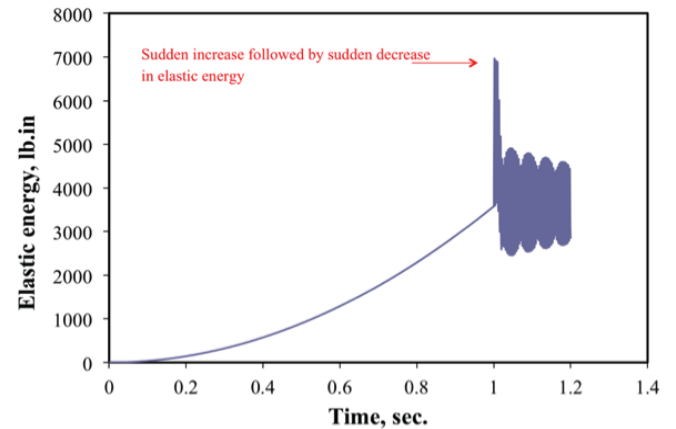


Figure 5. Variation of the elastic and kinetic energies before and after applying the impact load.

A to B, which is equivalent to breaking an overhanging roof of unit width with its length, thickness, and density equal to 55 ft, 15 ft, and 175 lb/ft³, respectively. The impact load was applied for a very short period of time, 0.01 sec. It remains constant for 0.01 sec, from B to C, before dropping back to 5,000 psi in 0.01 sec to point D, from where the load remains at 5,000 psi for the remainder of the analysis. The proposed load history shown in Figure 5 is expected to simulate the situation of pillar in the field, where the load increases gradually with the progress of mining. Since the roof is very stiff and strong, it will not cave readily, and it will overhang for certain length. When the overhanging roof breaks, an impact load will be transmitted to the adjacent pillars.

Figure 5 shows the variation of the elastic and kinetic energies with time. During the first stage of loading (static analysis) where the load is ramped up gradually, elastic energy increases gradually, while the kinetic energy is zero. In the second stage of loading, once the impact load is applied, there is a sudden increase in the elastic energy. Once the impact load has been removed, both the kinetic energy and the elastic energy decrease instantaneously so that a lot of energy is released, so cracks initiate and/or propagate inside the coal specimen. Since material damping has not been considered in the analysis, the coal particles vibrate freely around their equilibrium position. As the work done in deforming an elastic material is the area beneath the stress-strain curve, it has been stored as elastic strain energy. When the stress is released,

the work is recovered causing damage (Cook, 1963). In several instances of bumps, it was observed that unfilled cribs had been moved a number of feet laterally and left standing intact. This indicates a violent vibration of strong immediate roof (Rice, 1935).

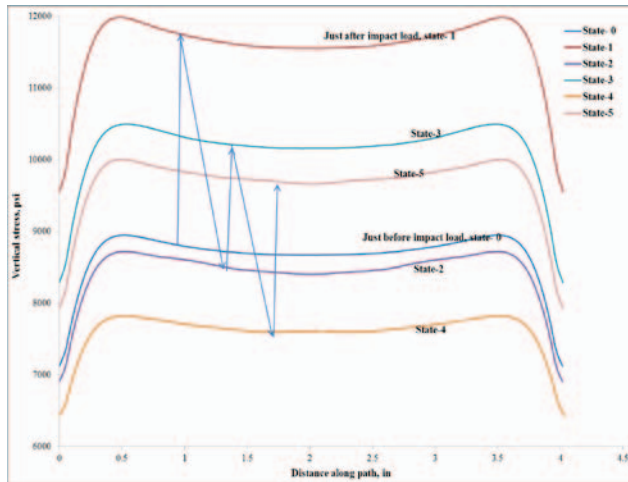


Figure 6. The variation of the vertical stress due to impact load at path 3-3.

Figure 6 shows the variation of the vertical stress due to impact load at path 3-3. It is obvious that, once the coal specimen is subjected to the impact load, it experience a sudden cyclic loading and unloading. The state of stress changes from state-0 (just before applying the impact load) to state-1 (just after applying the impact the load) then to state-2, all the way up to state-5. Notice the direction of the arrows. All these events occur in a short period of time. Such changes in the state of stress result in the generation and transmission of body waves (P & S waves) in the medium (Brady and Brown, 1993). By analogy, these sudden loading and unloading events result in contraction and expansion of the pillar. If the pillar is loaded near its ultimate strength, the induced stress from the impact load might exceed the strength of the pillar and cause a violent failure. Although the impact load = 1,000 psi, the induced stress is more than three times the impact load, it is expected that the intensity of the impulsive changes depends on the magnitude of the impact load (i.e., if the length and thickness of the overhanging roof are larger, the destructive effects of the impact load will be more significant). Rice (1935) observed two characteristic features of bumps over those pillars that had been crushed where the immediate roof is strong: 1) an open space usually found between the crushed coal and the immediate roof, and 2) the immediate roof is neither broken nor thrown down. These observations refer to the hammering effect of the immediate roof.

Therefore, a strong overlying stratum, such as sandstone, which occurs immediately above or close to the coal bed can cause bumps because such a stratum does not cave readily and throw an impact load on the adjacent pillar when it breaks suddenly.

MITIGATING THE VIOLENT FAILURE OF THE COAL SPECIMENS

When the coal specimen failed under the uniaxial compression test, three zones were recognized on the top surface of the failed coal specimen, depending on the degree of constraint (Prasetyo, 2010; Morsy, 2003). These zones are rib, transition, and core zones.

The rib zone is completely crushed, while the core zone has little or no damage, and the transition zone experiences intermediate damage. From the damage point of view, the three zones can be reduced to two zones: 1) elastic zone, usually called the core zone, which experiences little or no damage and 2) yielded zone, usually called the rib zone. As the potential for violent failure increases with increasing the UCS, if the strength of coal specimens can be decreased, coal mine bumps may be avoided. Two hypotheses have been assumed to eliminate or reduce the violent failure of coal specimens. These hypotheses are that the violent failure can be mitigated by softening the rib zone or the core zone of the coal specimen.

TESTING THE FIRST HYPOTHESIS FOR MITIGATING THE VIOLENT FAILURE

The first hypothesis was tested by softening the rib zone by drilling horizontal holes 1/10-in in diameter by 1/2-in deep. These holes drilled in the ribs of the coal specimen help initiate new cracks during the loading process and, therefore, might release the stored energy gradually. The distance between these holes is 1.0 in. Coal specimens of square cross-section and width to height ratio equal to 3.0 and 4.0 were tested under constant strain control where the rate of loading was 10-4 in/sec. The height of the specimen is kept constant and equal to 1 in. In order to quantify violent failure, several parameters have been determined. Some of these parameters are measured quantitatively, such as the unconfined compressive strength, the modulus of elasticity, and whether there is a sudden loss of strength or not. Other parameters have been measured qualitatively, such as the degree of ejection, the magnitude of noise. The qualitative scale employed ranges from low to very high and depends on the authors' experience in testing coal specimens. Failure of the coal specimen is violent when there is sudden loss of strength accompanied by high ejection and high loud noise.

Table 2 shows the test results for coal specimens of W/H ratio = 3 and 4, subjected to uniaxial compression test, where the ribs of these coal specimens were softened by horizontal holes. The test results show that, for coal specimens of W/H ratio = 3, only specimen #3 fails violently among seven specimens. It lost 85% of its strength suddenly after it reached the peak strength. The amount of ejection and the degree of noise were very high during the test. Specimen #3 attains the highest UCS and modulus of elasticity among all coal specimens of W/H ratio = 3. As mentioned before, in the first part of this research, the higher the UCS of the coal specimen, the more potential for violent failure. Coal specimens #1 and #5 have the lowest ejections and noise during failure. When the strength of the coal specimens increases to more than 6,439 psi, the amount of ejection and/or the noise level increase. This might provide an indication about the limit of strength which separates the violent and nonviolent failure for this coal.

As shown in Table 2, the strength increases and Young's Modulus decreases with increasing the W/H ratio. The degree of uncertainty about the strength and the modulus of elasticity decreases with increasing width to height ratio. All coal specimens of W/H ratio = 4 failed violently with very high ejection and noise, except specimen #6. Specimen #6 had the lowest UCS among coal specimens of W/H ratio = 4. Although specimen #6 did not fail violently, its failure was not smooth. It was accompanied by moderate ejection and high noise. This indicates that, for a coal

Table 2. Test results for coal specimens where the ribs were softened by horizontal holes.

Specimen #	W/H ratio	UCS, psi	UCS _{avg} , psi	St. Dev. UCS	E, psi	E _{avg} , psi	St. Dev. E	Ejection	Noise	Sudden loss of strength
1	3	6,331	6,864	551	298,630	301,167	19,650	Low	Low	No
2		6,707			323,610			Moderate	Moderate	No
3		7,873			318,990			V. high	V. high	Yes, 85% of UCS
5		6,439			268,125			Low	Low	No
6		6,862			298,590			High	High	No
7		6,974			299,056			moderate	low	No
1	4	8,880	8,800	267	235,104	240,377	14,185	V. high	V. high	Yes, 92% of UCS
2		9,000			264,472			V. high	V. high	Yes, 93% of UCS
3		8,886			228,787			High	V. high	Yes, 80% of UCS
5		8,905			232,515			V. high	V. high	Yes, 78% of UCS
6		8,330			241,009			Moderate	High	No

specimen of W/H ratio = 4, the strength should be less than 8,330 psi to fail smoothly. Therefore, softening the ribs is effective in mitigating the violent failure for coal specimens of W/H ratio = 3. Conversely for W/H = 4, softening the ribs is not effective in mitigating the violent failure. Note that for coal specimens of W/H ratios 3 and 4, the depth of the drilled holes is the same, 0.5 in. A 0.5 in deep hole is not enough to soften the coal specimen with a width to height ratio equal to 4. It is very likely that the failure of the ribs does not cause the violent failure. The failure of the core might be the main reason for the violent failure. To mitigate the violent failure, the holes must reach inside the core zone. For coal specimens with a width to height ratio equal to 3, a 0.5-in-deep hole is enough to penetrate the core zone and mitigate the violent failure, but, for coal specimens of width to height ratio equal to 4, a 0.5 in deep hole is not enough to penetrate the core zone.

TESTING THE SECOND HYPOTHESIS FOR MITIGATING THE VIOLENT FAILURE

Since the core has the highest confinement, it can store more elastic strain energy, which could be transformed into fracture energy and released violently at the moment of failure. Therefore, softening the core zone through drilling vertical holes will help the core zone to expand laterally and convert most of the input energy into inelastic energy (i.e., some of the input energy dissipated in closing these vertical holes instead of stored as elastic energy in the core zone). To the authors' knowledge, there has been no experimental work done before about how to soften the core of the coal specimen. Prasetyo (2010) studied the variation of the "core zone" at different width to height ratios and concluded that, the higher the width to height ratio, the larger the core zone. Hence, the larger the core zone, the higher the number of holes needed to soften that core. Therefore, for a small width to height ratio, such as 2, only one hole at the center of the specimen might be enough to soften the core. However, for higher width to height ratios, such as 3 and 4, more holes are needed.

Figure 7 shows the arrangement of the vertical holes for specimen of W/H ratio = 4. The diameter of these holes is 1/16 in. During the first attempt to soften the core of the coal specimens, when the depth of the holes was equal to the thickness of the coal specimen, which was 1 in, the coal specimens were badly fractured.

That is why the depth of the holes was reduced from 1 in to 0.8 in to reduce the amount of disturbance to the coal specimens. Table 3 shows the difference in strength for the specimen of W/H ratio = 4 when the depth of the holes is 1.0 and 0.8 in. Coal specimens of square cross-section and width to height ratio equal to 2, 3, and 4 were tested under a strain control of 10-4 in/sec. Table 3 shows the test results for the coal specimens where the core zone was softened by vertical holes of diameter 1/16 inch and 0.8 in deep.

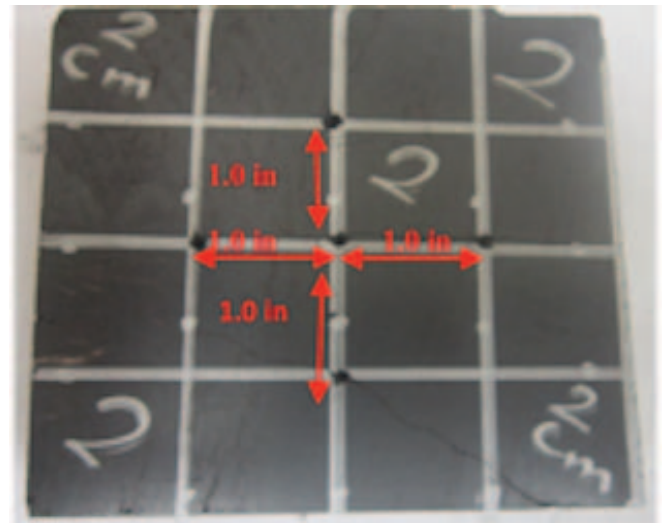


Figure 7. The arrangement of the vertical holes for specimen of W/H ratio = 4.

As shown in Table 3, all coal specimens fail smoothly without sudden loss of strength, and the amount of ejection and the noise level during testing were very low. This was because the strength of the coal specimens was reduced significantly when the core was softened compared with other coal specimens without softening (see Tables 1 and 3). The reduction in the UCS due to softening the core zone was 20% for specimens of W/H ratio = 2 and 3, while it was 17% for specimens of W/H ratio = 4. One important observation is that a pillar of W/H ratio 2, 3, or 4 would not fail

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violently as long as its strength does not exceed the average strength listed in Table 3.

cause violent failure if the pillar were already loaded close to its full bearing strength.

Table 3. Test results for coal specimens of different W/h ratios where the core was softened by vertical holes.

W/H ratio	Specimen #	UCS, psi	UCS _{avg} , psi	St. Dev. UCS	E, psi	E _{avg} , psi	St. Dev. E	Ejection	Noise	Hole depth, in
2	1	5,039	4,709	318	313,818	301,508	24,645	Low	Low	0.8
	2	4,890			326,988			Low	Low	0.8
	3	4,579			270,217			Low	Low	0.8
	4	4,328			295,009			Low	Low	0.8
3	1	5,430	5,210	438	216,105	227,373	44,046	Low	Low	0.8
	2	5,494			275,958			Low	Low	0.8
	3	4,705			190,055			Low	Low	0.8
4	1	6,441	6,362	201	213,474	211,967	9,609	Low	Low	1.0
	2	6,511			220,734			Low	Low	1.0
	3	6,133			201,693			Low	Low	1.0
	4	7,493	7,290	618	236,223	220,872	15,805	Low	Low	0.8
	5	7,576			235,266			Low	Low	0.8
	6	6,055			191,028			Low	Low	0.8
	7	7,906			218,542			Low	Low	0.8
	8	7,010			218,979			Low	Low	0.8
	9	7,533			226,140			Low	Low	0.8
	10	6,874			206,652			Low	Low	0.8
	11	7,872			234,147			Low	Low	0.8

Figure 8 compares the stress-strain curves for some coal specimens of width to height ratio equal to 4 with and without core softening. Solid lines denote coal specimens with core softening, while dotted lines represent those without core softening. Coal specimens with core softening do not lose their strength suddenly after the peak strength is reached. They keep deforming after the peak strength. While coal specimens without core softening lose their strength suddenly and completely in a violent way, when the peak strength is reached, there is almost no plastic deformation afterward. Softening the core zone delays crack propagation. Moreover, the post peak slope of the specimens with core softening is smaller than those without core softening. (See β_1 vs. β_2 .) According to the local mine stiffness criteria introduced by Salamon (1970), which is used to distinguish between stable and unstable pillar failure, the coal specimens with core softening are more stable than those without core softening.

CONCLUSION

Both the postpeak slope of the stress-strain curve and the elastic energy released increase with increasing UCS. Therefore, the potential for violent failure increases with increasing the UCS of the coal specimen. The effect of modulus of elasticity on the potential for violent failure is not as significant as the UCS.

A sudden change in interface friction or sudden breaking of the overhanging immediate roof is associated with sudden release of elastic and kinetic energies and very rapid change in the vertical stress. These transient changes are affected by the difference between the static and the dynamic friction angles and the size of the overhanging immediate roof. These transient changes would

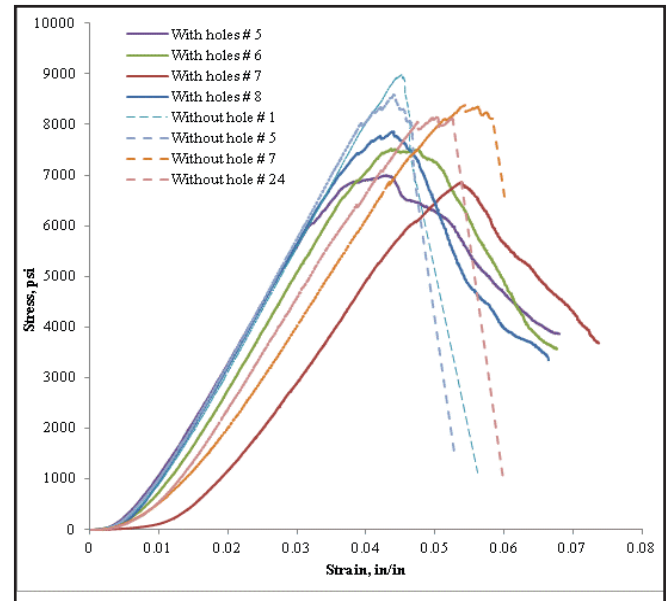


Figure 8. Stress-strain diagrams for coal specimens of W/H ratio = 4 with and without core softening.

Softening the ribs of the coal specimen is not effective in mitigating the violent failure especially for coal specimens of higher width to height ratios, while softening the core of the coal specimen is very effective in mitigating the violent failure. Hence, the energy stored in the core zone is the main cause for violent failure.

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ACKNOWLEDGMENT

This research was sponsored by the National Institute for Occupational Safety Health's Office of Mine Safety and Health Research.

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Biaxial Horizontal Swelling Strains in West Virginia Coal Mine Roof Rocks in Response to Moisture Adsorption

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ABSTRACT

Study of moisture-induced swelling strain in Pennsylvanian Period rocks of the Appalachian Plateau in West Virginia shows a 2:1 biaxial horizontal anisotropy which trends NW-SE. Oriented in-situ slabs of coal mine roof rock were collected across West Virginia, dry-sawed to size, air-dried to stability, and then monitored for moisture changes and deformations in the horizontal plane through one wetting cycle and drying cycle. Biaxial swelling strains were exhibited in nearly all rocks tested, with maximum horizontal deformations occurring normally-incident to the strike of regional geologic fold structures. This relationship persists as fold strike varies across the study area. A connection with Alleghanian fold development and/or post-Alleghanian fold relaxation is suggested. Deformations were largest in shales, but occurred in sandstones and siltstones as well. During testing, moisture-induced fractures (MIF) developed parallel to the regional fold structure and ranged in size from hairline cracks to sample-destroying cracks that were orthogonal to the observed biaxial swelling strain direction. Preferential wetting and swelling of clay minerals along NE-trending fractures, related to a fold-parallel fracture fabric of regional extent across West Virginia, is the most likely reason for the observed systematic NW-oriented biaxial swelling strains.

The testing methods used in this study are simple and inexpensive and produce robust results that offer insight into a regional, anisotropic mechanical weakness that imparts a dynamic, asymmetrical horizontal stress to moisture-sensitive coal mine roof rocks over time. Because a similar mechanical fabric has been reported in Devonian-age shales, these findings may have implications for shale gas developers in the Appalachian Basin challenged by rapid aperture closure rates along horizontally-drilled induced fracture networks in Devonian rocks like the Marcellus Shale. The paper concludes with an example of how the concept of a preferred swelling strain direction might be applied to improve the effectiveness of roof trusses.

INTRODUCTION

When clay minerals in rocks adsorb moisture they expand, creating swelling strain and stress that is separate and distinct from in-situ ground stress. This study tests oriented rock slabs pulled from the mine roof of approximately twenty (20) West Virginia

coal mines operating in twelve (12) different coal beds to quantify the horizontal component of strains associated with the swelling stresses. The locations and stratigraphic positions of test samples appear in Figure 1, and span most of the Upper Carboniferous Period (Pennsylvanian Period), ranging from the Pocahontas No. 3 coal bed (Early Pennsylvanian) to the Sewickley coal bed (Late Pennsylvanian).

Pennsylvanian-age rocks of (286-320 million years) in the central Appalachian Basin are fluvial-deltaic successions of sandstone, shale, and minor limestone. The overall gentle northwestward regional dip is interrupted by a number of broad, open anticlines (Figure 1) with limbs dipping generally less than five (5) degrees and axes trending in a northeasterly direction. In West Virginia structural symmetry of these folds extends to deeper sedimentary rocks, including the well-studied gas-bearing Upper Devonian rocks (360-408 million years) that include the Marcellus Shale. Our test results show that during moisture adsorption, sedimentary Pennsylvanian rocks across West Virginia respond with biaxial deformations that are oriented NW-SE (Figure 1). This trend is orthogonal (approximately perpendicular) to a directional strength anisotropy which is aligned parallel to anticline fold axes, and imparts a NE-trending weakness anisotropy to moisture-sensitive shale roof rocks.

This study has two purposes. First, to investigate the possibility that horizontal, moisture-induced swelling strain in sedimentary rocks is related to the present-day horizontal in-situ stress field. Such a relationship has been previously reported for copper-bearing Precambrian (>600 million years) sedimentary rocks in the Michigan Basin (Parker, 1973; Fohey, 1976; Vermuelen, 2011).

The second purpose is to quantify, specifically, the horizontal component of swelling strains which accompany moisture changes in coal mine roof rocks. Shales, in particular, undergo relatively large volume expansions and contractions in response to changes in moisture content, leading to progressive unraveling and eventually roof failure. Prior studies have examined the problem volumetrically, and a few have examined the vertical component of swelling strain (e.g. Chugh, et.al., 1980; Hakan and Peng, 2006), but no previous work has examined the orientations and magnitudes of horizontal swelling strain across West Virginia using oriented samples. A distinct NW-SE oriented anisotropy has been

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found and an example of how knowledge of this anisotropy may be put into practice to improve the effectiveness of roof trusses will be presented.

TEST PROCEDURE

Experiments were performed using a 3-point rosette procedure (Parker, 1973, 2011-2012; Haas, 1982) for measuring the deformation of unconfined rock slabs, using a mechanical strain gauge. Approximately 69 oriented roof samples were collected from underground mines throughout West Virginia between October 2011 and June 2012. In-situ roof samples were collected, as close to the active section as practical. Sample selection targeted rocks that were free of visible cracks or joints, and had not experienced a seasonal humidity cycle. The minimum desired slab size was 16 x 16 x 2 inches, although often larger slabs were pulled, from which several samples could be cut. Most samples were shale although sandstone and sandy shale samples were also tested.

First, slabs were marked for true north using a Brunton compass and double plumb lines. They were then carefully scaled down, wrapped in ventilation curtain, and transported to the lab, where they were cut into octagon-shaped samples, with diameters of 11, 13, or 15 inches, using a dry masonry saw. Prepping the samples in this way minimized rock spalling during testing, provided a clear view of the bedding structure before and during testing, and minimized the potential of rock shape to bias the test results.

Samples were air-dried at 30-50% relative humidity, and their moisture losses recorded over the next 2-6 weeks (Figure 2) by tracking weight change using a digital scale with +/- 2-gram precision. Room humidity levels were monitored with a digital psychrometer and regulated with a dehumidifier.

After drying to a stable weight, monitoring points were installed in a 3-point strain rosette arrangement, at vertices A, B, C of a 6-inch equilateral triangle, with Leg AB always being aligned with true north (Figure 3). The slab side with the most level surface was usually chosen to serve as the side for monitoring points (spad pins), so samples tested upside down had an inverted spad pin arrangement.

Spad pins were fashioned by drilling a hole of 1/16-inch diameter through the head and into the shanks of 1.75-inch finished galvanized roofing nails using a drill press (Figures 3C, 3D). A masonry bit and impact drill were used to mount the spad pins. Holes were drilled normal to the bedding, usually to a depth of half the rock thickness, to ensure the spadps terminated near a common bedding plane. Pins were trimmed for length and then installed using epoxy, turning and pumping the pins for thirty seconds each to work out air bubbles.

After epoxy cured 8 hours or more, baseline measurements were made between pins using a digital mechanical strain gauge (Figure 4D). When used properly, this simple instrument is accurate to 1/10,000 inch. The name "strain gauge" is a bit of a misnomer, because the measurements actually record a *deformation*, from which *strain* is calculated, and subsequent measurements are relative to the initial baseline readings. An invar steel bar was used to calibrate the strain gauge and compensate for temperature.

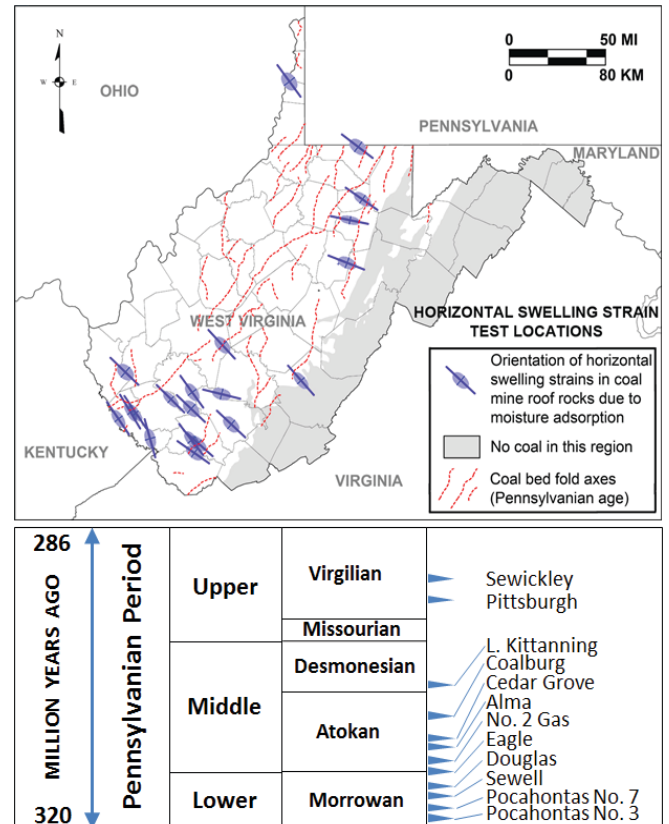


Figure 1. Approximately 69 oriented roof rock samples were pulled in-situ from underground coal mines across West Virginia, ranging upward in age from the Pocahontas No. 3 seam to the Sewickley seam.

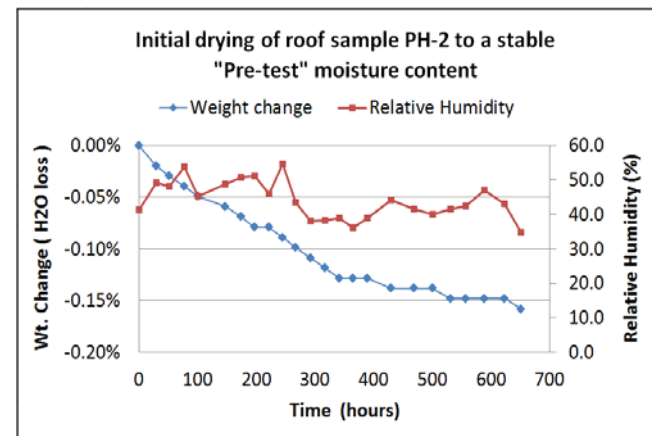


Figure 2. After sawing to size, test samples were air-dried at 30-50% ambient relative humidity. Relative changes in the rock's moisture content were determined by weight change.

The method of introducing moisture to the samples was a medium-weight cotton bath towel, wetted with approximately one-half gallon of water, and covered with a plastic sheet to limit evaporation (Figures 4B, 4C). This allowed the samples to take on moisture evenly and at their own rate without saturating them. Either aged water or discharge water from the dehumidifier was used. Two (2) tablespoons of vinegar per gallon were added to

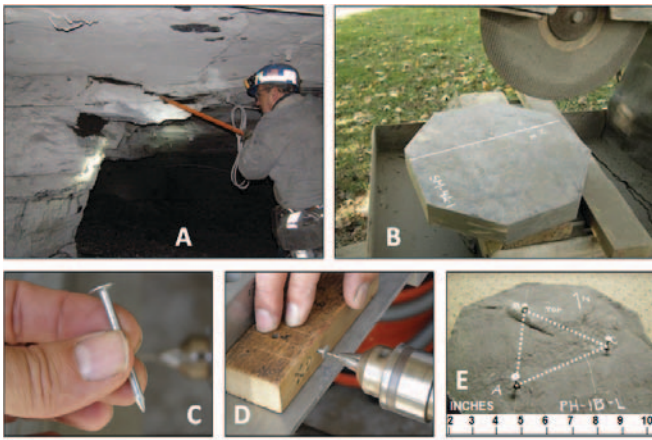


Figure 3. Rock slabs were first marked for true north, then (A) scaled from mine roof and (B) dry-sawed to standard geometry. Spad pins were fashioned (C,D) by drilling a 1/16-inch diameter hole into the top and shank of finished roofing nails, then (E) installed as an equilateral “3-pt. rosette” arrangement into each test sample.

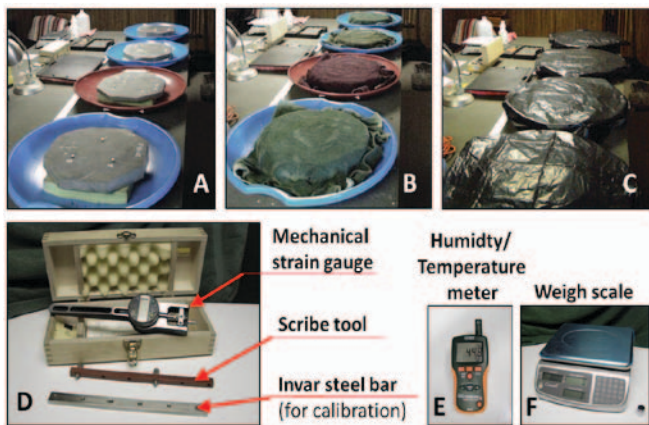


Figure 4. Oriented test samples are (A) placed in containers where (B) they are covered with a wetted towel then (C) covered by plastic sheets to restrict evaporation. Instrumentation includes (D) a mechanical strain gauge, (E) temperature and humidity meter, (F) digital scale.

retard mildew. If the presence of carbonate minerals was suspected in specimens hydrogen peroxide was substituted for vinegar.

Horizontal, rock deformations were monitored at appropriate time intervals, and redundant measurements were made to verify accuracy. The strain gauge was checked before each session, and periodically during testing, using the invar steel calibration bar. Room temperature and humidity were tracked with a digital temperature/humidity meter (Figure 4E) and rock samples were weighed on a digital scale (± 5 -grams) to track changes in moisture content (Figure 4F). Measurements were recorded on standard forms, together with the date, time, humidity, temperature, and visual observations. These data were then keyed into an Excel spreadsheet where temperature correction (ASTM, 1999) and charting operations were performed.

Samples were put through one wetting cycle, followed by one air-drying cycle. The wetting phase took 4-45 days (average 18 days) and was considered complete when an expansion plateau was achieved. Corresponding rock weight increases due to moisture uptake ranged 0.12 to 3.91 percent (average 1.06 percent) as shown in Figure 8. When rocks reached this point they were removed from humidity, and the measurements continued for an additional 5-60 days (average 23 days) until the samples had air-dried back to their “pre-test” moisture. Approximately 69 rock samples were tested, of which 57 provided sufficient data to compute strain ellipses to quantify moisture induced deformations in the horizontal plane (see Figure 8).

SUMMARY OF TEST RESULTS

Study and comparison of the compiled charts gives useful information about the horizontal swelling behavior. The measured deformations were generally proportional to the weight gain (moisture uptake), but comparisons between sample sets showed differences in the rate, magnitude, and elastic response. Sample sets for each mine ranged from 1 to 5 samples (average 3). An example is shown in Figure 5 which took 451 hours to reach its full expansion at a relative humidity range of 70-90%. During this time the triangle legs a, b, and c expanded by the amounts shown in Figure 5A, as the sample gained 0.35% of its original weight in moisture (Figure 5B). The sample was then allowed to air-dry at 30-50% RH, until its pre-wet moisture was achieved, which occurred in this example after an additional 348 hours. As the moisture gain reverses, the swelling strain generally follows, but there were exceptions.

Insights into elastic response appear in Figure 5A, which shows that approximately 26% of the rock deformation did not recover after air-drying to the starting (pre-test) moisture. This is an example of a residual horizontal deformation that becomes the new starting point for the next humidity cycle.

When rock samples were temporarily removed from humidity for weighing, the surface evaporation tended to highlight and reveal subtle fractures on rock surfaces that were not obvious before wetting. As testing progressed it was noticed that these cracks developed on samples with a dominant northeast orientation (see Figures 11, 13 for examples). The significance of these fractures will be discussed later.

The actual amount and types of clay minerals in the test samples were not measured, but samples were retained and may be tested at a later date. Sandstone response showed less correlation with moisture content than the shale samples. While sandstones initially showed a correlation between rock expansion and moisture increase, sometimes the expansions ceased, and sometimes reversed, even as moisture uptake continued. Sandstones have a greater porosity than shales, and their aberrant expansion response compared to shale is attributed to the permeability of sandstone samples we tested being dominated by pores rather than microcracks. Shales on the other hand appear to utilize the subtle NE-trending fractures and microcracks as conduits for transmitting water into the core of rock.

Deformation measurements were converted to strain ellipses using equations derived from U.S. Bureau of Mines research of the 1960's (Obert and Duvall, 1967) that is summarized in Figure 6.

The solution is sometimes referred to as the “3-point strain rosette,” and we used those parameters to compile and graph our strain statistics, in Excel.

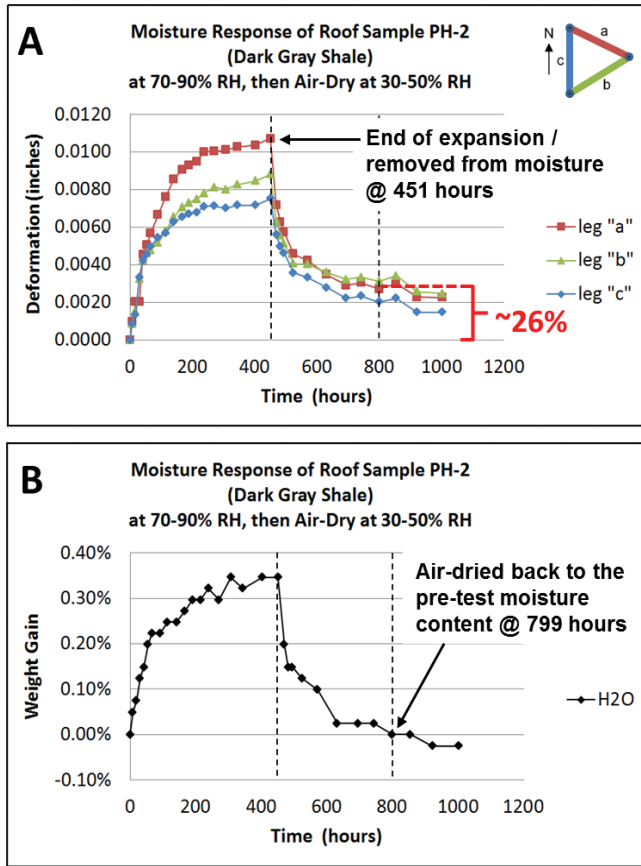


Figure 5.(A) Oriented rock samples were put through one wetting cycle and one drying cycle. Their 2D horizontal deformations were monitored at spad pins installed at the vertices of an equilateral triangle (legs are identified a,b,c). (B) Corresponding changes in moisture content were also monitored. Approximately 69 series of charts were compiled from over 16,000 measurements for this study.

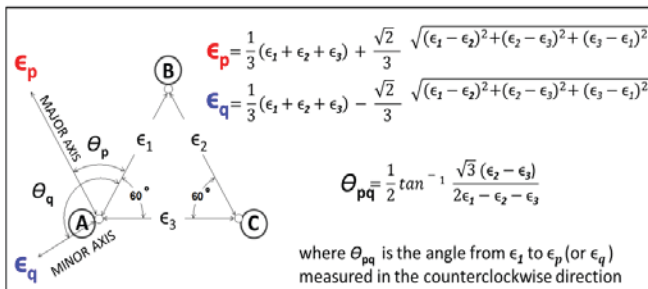


Figure 6. Equations for resolving the strain ellipse for deformation between three monitoring pins at vertices A, B, C of an equilateral triangle (adapted after Obert and Duvall, 1967; Haas, 1982).

Strain ellipses can be constructed at any selected point during testing. Typically, the reference point used for strain comparison was at the end of moisture expansion. The measured deformation in the example shown in Figure 7A was evaluated at the end of moisture expansion (804 hours) and the calculated values for azimuth, deformation, and strain appear in Figure 7C. The resulting strain ellipse is shown Figure 7B. The strain values were checked using an independent geometrical solution developed for this project and found to be in close agreement.

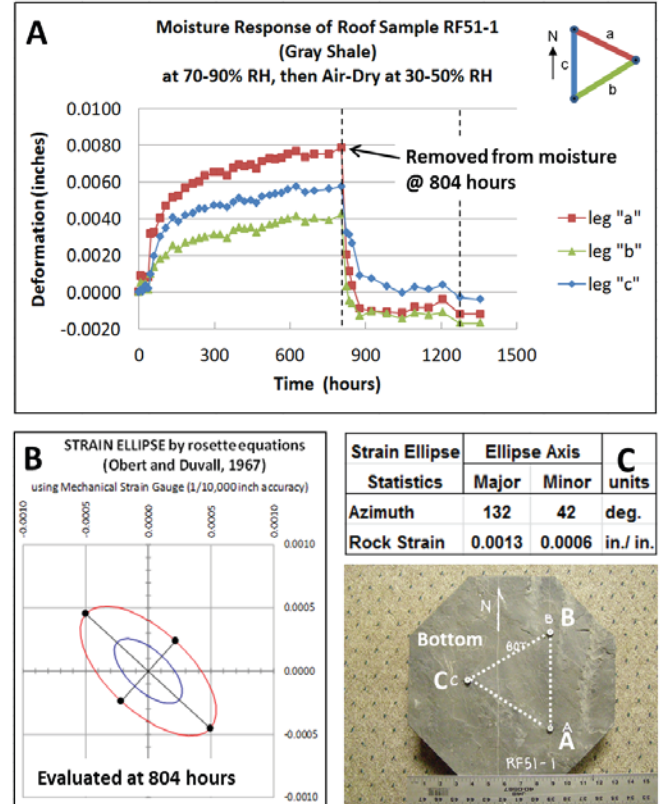


Figure 7. Example results of (A) measured deformations along legs a, b, and c that (B) can be resolved to a strain ellipse using the 3-pt. strain rosette equations, and (C) from which the azimuth and strain along the major and minor axes are readily computed.

Test results (Figure 8A) show that the swelling strains are biaxial, with the ratios of the ellipse axes ranging from 1.3 to 3.0 (average 1.9). Interestingly, all test locations demonstrated northwest-oriented biaxial strains. Sandstones exhibited the smallest horizontal swelling strains, with major axes between 500 and 1,000 micro-strains (0.0005 to 0.0010 inch/inch). Shales exhibited the highest horizontal swelling strains, typically 2,000 to 8,000 micro-strains, with one shale sample set having a swelling of 12,100 micro-strains. For comparison, 4,000 micro-strains over a 20-ft.-wide mine opening would produce nearly 1 inch of horizontal rock expansion.

Compared to strains associated with horizontal in-situ rock stress reported for the Appalachian Basin (Dolinar, 2003), our swelling strains are non-trivial. Figure 8B details previously published values of “excess horizontal strain” in the Central Appalachian

A		HORIZONTAL SWELLING STRAIN from MOISTURE					
Sample		Swelling Strain (in./in.)				$\epsilon_{\max}$	% H2O
Location	Rock Type	n	MAX	MIN	Ratio	Bearing	Gain
1	SS with siderite cement	4	0.0006	0.0003	2.0	N 78 W	0.11
2	Med.-gr. SS; micaceous	3	0.0008	0.0005	1.6	N 51 W	0.57
3	Dark gray shale w/siderite bands	3	0.0009	0.0005	1.8	N 78 W	0.27
4	Fine gr. SS w/ siderite clasts	1	0.0011	0.0005	2.2	N 58 W	0.59
5	Gray silty shale / gray SS	3	0.0018	0.0010	1.8	N 36 W	---
6	Dark gray shale	3	0.0019	0.0010	1.9	N 40 W	0.35
7	Dark gray shale	4	0.0020	0.0011	1.8	N 69 W	0.38
8	Gray shale	3	0.0024	0.0012	2.0	N 49 W	0.35
9	Dark gray shale	3	0.0025	0.0013	1.9	N 54 W	0.43
10	Dark shale	4	0.0028	0.0019	1.5	N 37 W	1.07
11	Light gray shale	1	0.0029	0.0018	1.6	N 47 W	1.61
12	Gray shale	5	0.0029	0.0017	1.7	N 44 W	0.63
13	Dark gray shale	2	0.0030	0.0019	1.6	N 43 W	0.70
14	Gray shale	4	0.0034	0.0023	1.5	N 47 W	1.39
15	Dark gray shale	3	0.0043	0.0021	2.0	N 40 W	1.14
16	Dark gray shale	1	0.0047	0.0026	1.8	N 16 W	1.08
17	Soft gray shale	4	0.0057	0.0031	1.8	N 53 W	1.18
18	Dark gray shale	5	0.0068	0.0030	2.3	N 46 W	2.08
19	Dark gray shale w/ FeCO3 nodules	5	0.0079	0.0029	2.7	N 39 W	1.23
20	Dark gray shale	5	0.0097	0.0041	2.4	N 71 W	2.20
21	Gray shale; soft, soapy	4	0.0451	0.0182	2.5	N 34 W	3.37
		70	AVERAGE		1.9		1.04

B	IN-SITU EXCESS HORIZONTAL STRAIN				(1) Also called "excess horizontal strain," calculated from the measured horizontal stresses by removing the expected effects of gravity (assumes 0.25 value for Poissons Ratio). Ref.: Dolinar, (2003)
	Tectonic horizontal strain in Central Appal. (1)				
	n	HI-MAX	HI-MIN	LO-MAX	
12	0.000760	0.000410	0.000370	0.000170	
	0.0008	0.0004	0.0004	0.0002	
	(rounded to 4 significant digits)				

Figure 8. Test results summarizing (A) horizontal strains from moisture-induced rock expansion, compared to (B) the in-situ tectonic component of horizontal strain (excess horizontal strain) in the "high-strain" zone of the Central Appalachian basin (Dolinar, 2003).

Region (Dolinar, 2003), calculated from measured horizontal stresses by removing the expected effects of vertical load. These in-situ rock stresses are some of the highest values in the Appalachian Basin, yet their strains were less than the horizontal swelling strains we measured due to moisture changes. This does not imply that the swelling stress is greater, however, because wetted rocks lose strength, so the elastic moduli are different (Chugh and Missavage, 1981).

CAUSE OF THE BIAxIAL SWELLING STRAINS

Relationship to In-situ stress

Contrary to initial expectations, the direction of moisture-induced horizontal swelling strain does not appear to be correlated to the horizontal stress field that exists in West Virginia today (Figure 9). Instead, it appears to be related to an earlier stress field that existed during the early Alleghanian Orogeny when NW compression initiated development of the regional NE-trending anticline fold structures in West Virginia (Figures 1, 12).

Previous conclusions by Michigan researchers (Parker, 1973; Vermuelen, 2011) of a parallelism between horizontal swelling strain direction and contemporary stress direction in northern Michigan Basin Precambrian sediments may be explained by

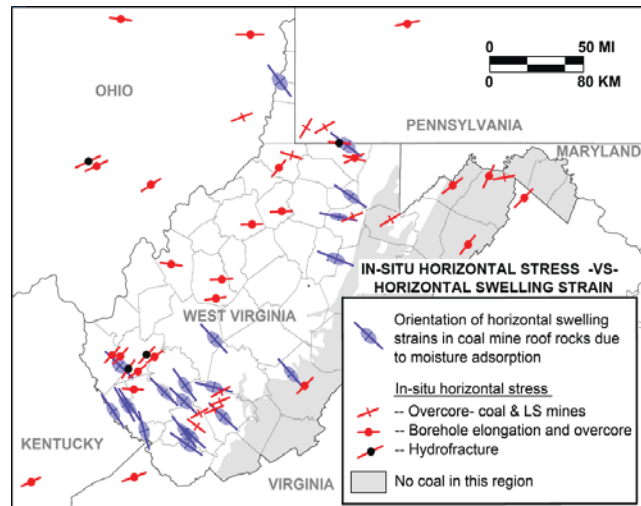


Figure 9. There is no apparent correlation between horizontal swelling strains in Pennsylvanian Period rocks and the present in-situ horizontal stress field in West Virginia.

the comparatively quiet subsequent tectonic history for the northern Michigan Basin from then to present-day. The stress field remains aligned in the same NW-SE compression direction since its inception (Vermuelen, 2011, Parker, 1973). In contrast, Pennsylvanian rocks in West Virginia have been subjected to multiple subsequent episodes of tectonic deformation, including a general counter-clockwise rotation in the stress field orientation after initial fold development (Evans, 1994). Consequently, the stress field responsible for the NW-trending biaxial swelling strain development has been over-printed, explaining the lack of a general correlation to present-day in-situ stress in West Virginia.

Relationship to Coal Cleat

Coal cleat orientation is indirectly related to swelling strain direction, but with exceptions. As shown in Figures 10A and 10B the trends of coal extension fractures (face cleats) are divided into seven (7) domains in West Virginia (Kulander and Dean, 1993). Each domain has a major cleat trend and, sometimes, a minor cleat trend. The dominant, major cleat trends are likely the first-formed (Kulander and Dean, 1993), and their strike is in the direction of maximum horizontal stress at the time formed.

Major coal cleat trends across West Virginia are predominantly oriented NW-SE, normal to regional fold axes and parallel to horizontal swelling strain. There is an exception that includes a single contiguous area along the Allegheny Structural Front in Domains 6, 7, and the eastern part of 2, where coal cleats strike predominately ENE, and are seemingly correlated with today's horizontal stress field (Figure 9), but this may be coincidental (Engelder and Whitaker, 2006). The ENE coal cleats of Morrowan-age coal beds (see Figure 1) situated along the Allegheny Structural Front are believed to pre-date Alleghanian fold development (Engelder and Whitaker, 2006). West Virginia's NE-trending regional folds developed next, and their anticline fold axes (Figure 12) appear as the common denominator between the orientation of remaining major coal cleats and orientation of the maximum horizontal swelling strain west of the Allegheny Front.

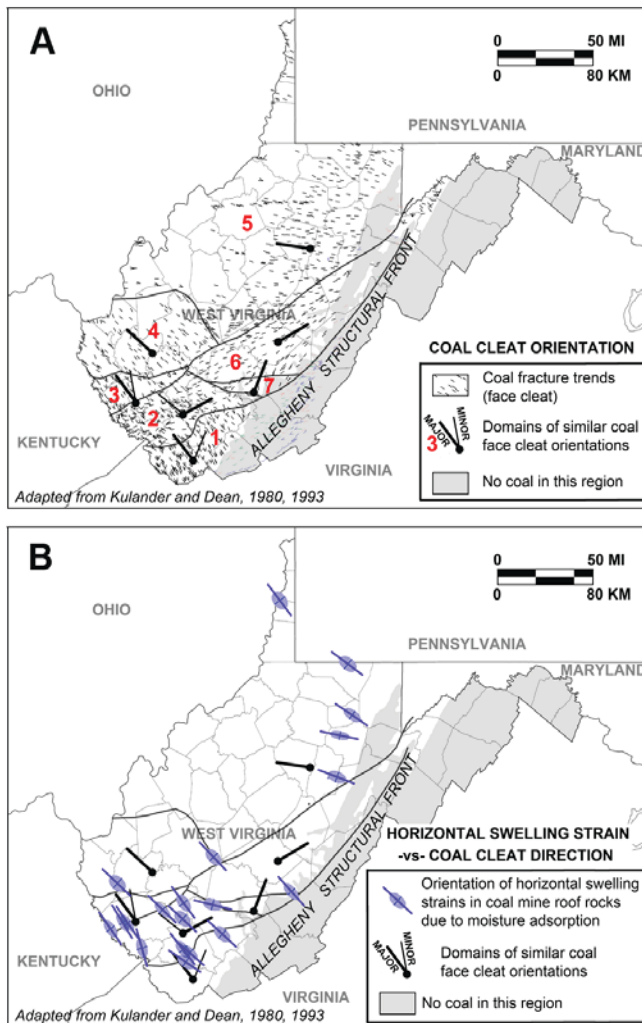


Figure 10. Coal cleat trends are divided into seven domains of similar orientations (Kulander, and Dean, 1981, 1993). (B) When generally compared to the trends of horizontal swelling strains in coal mine roof rocks they are similar, except for Domains 6, 7, and part of 2.

Relationship to Regional Anticline Fold Axes

The trends of maximum horizontal swelling strain in our dataset appear to be normally-incident to anticline fold axes, and this relationship persists as fold strike varies across the study area (Figure 12). Regional fold axes developed during the early part of the Alleghanian Orogeny when the horizontal stress field was oriented NW. A relationship to this earlier stress field is suggested, either as 1) an artifact of residual Alleghanian stress, or 2) post-Alleghanian stress unloading and relaxation.

This relationship extends to the NE-trending fabric of tensile weakness that was mentioned earlier (see Figure 11). The moisture-induced fractures that developed in about 25% of our test samples in West Virginia (predominantly shale rocks) always trended northeast. Some samples failed along these cracks, spoiling the results for strain measurement. These fractures were not discernible in the rocks until they began to take on moisture

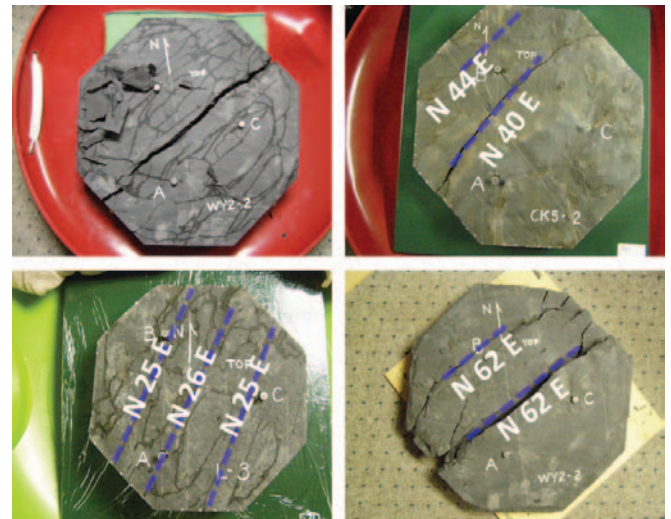


Figure 11. Examples of NE-trending cracks observed during moisture adsorption.

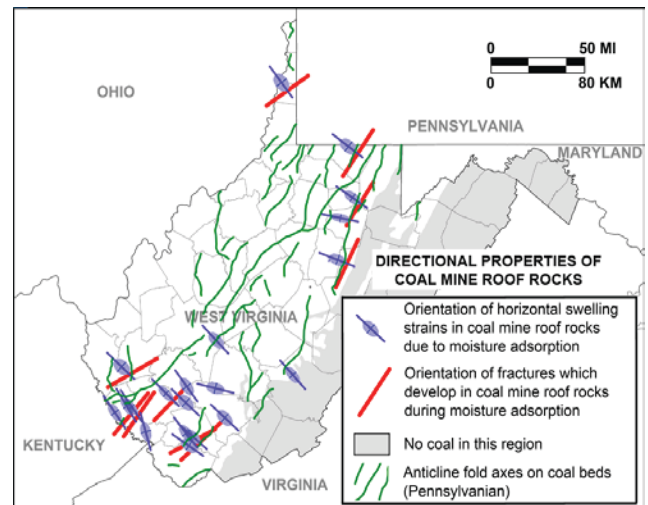


Figure 12. NE-trending fractures (red) that develop in coal mine roof rocks during moisture adsorption open along directions that are sub-parallel to anticline fold axes (green). Our tests show biaxial horizontal swelling strains develop perpendicular to these fracture trends.

during testing.

The orientation of moisture-induced fractures (MIF) from the various locations we tested varied between N 25 E and N 62 E (average N 45 E). MIF trends are aligned orthogonally to the major axis of swelling strain, which varies, by test location, between N 16 W and N 78 W, (average N 47 W). An example of this orthogonal relationship is shown in Figure 13. MIF directions follow the primary NE-trending fold axes across West Virginia and are believed to be related to the “SJW” joints in Devonian cores described by Evans (Evans, 1994) and to a direction of inferred tensile weakness in Devonian rocks (Figure 14). These are attributed to post-Alleghanian release fractures that impart a preferred direction of weakness in (Devonian) shales, facilitating

the opening of joints during stress relaxation or reorientation (Evans, et. al., 1989; Evans, 1994). Parallelism between this weakness trend and fold axes persists, even as fold strike varies across West Virginia. This rock strength anisotropy affects the direction and manner in which the immediate shale mine roof unravels and transmits moisture across bedding planes and into the internal rock mass.

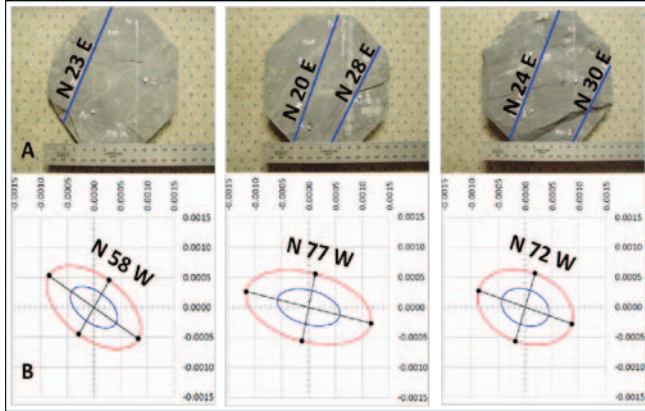


Figure 13. (A) Moisture-induced, NE-trending fractures which developed in test samples are (B) orthogonal to the major axis of swelling strain. The samples depicted came from the same mine.

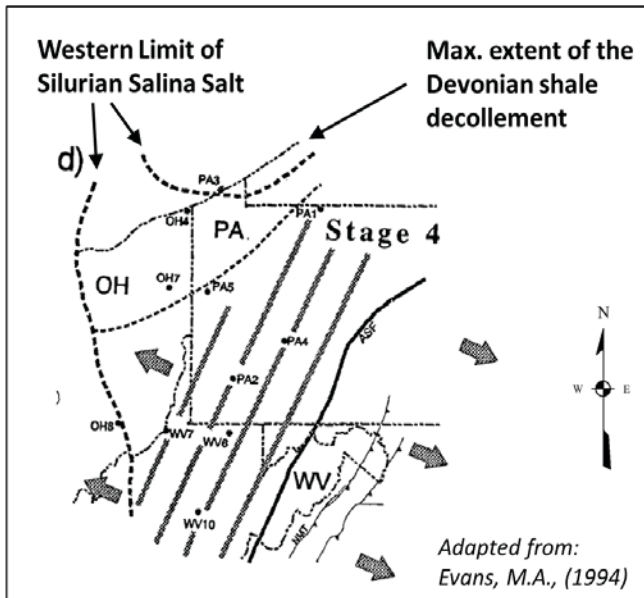


Figure 14. The NE-oriented moisture fracture fabric that developed in our test samples is related to a preferred direction of weakness that facilitated opening of release joints (Stage 4) during Post-Alleghanian stress relaxation and reorientation. Adapted from Evans, M.A., (1994).

Other insight into the mechanical fabric in Pennsylvanian Period rocks comes from the research of Devonian rocks sponsored by the U.S. Department of Energy in the 1970's and 1980's, as part of the Eastern Gas Shale Project (EGSP). An adaptation of some of that information is incorporated into Figure 15.

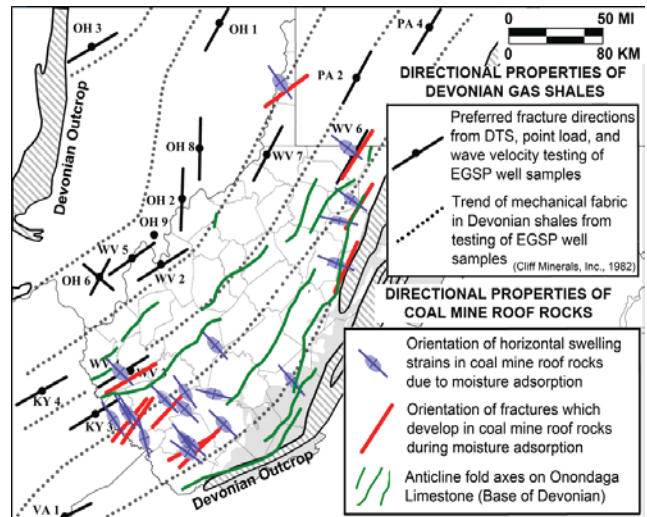


Figure 15. Directional properties observed in Devonian rock cores from the Eastern Gas Shale Project (EGSP) display a mechanical fabric which follows moisture-induced fractures (MIF) observed in coal mine roof rocks.

The NE-oriented dotted lines across West Virginia shown in Figure 15 delineate a preferred direction of weakness in Devonian rocks (based on core analyses) which manifests itself as 1) a fracture direction in point load testing, 2) as a minimum tensile strength plane in directional tensile strength testing, and 3) as an ultrasonic velocity minimum in wave velocity testing (Cliff Minerals, Inc., 1982). This “mechanical fabric” is attributable to a microstructure that appears to be an oriented array of micro-cracks on grain boundaries (Cliff Minerals, Inc., 1982).

The Devonian strength anisotropy follows the Devonian anticline fold axes (Cliff Minerals, Inc., 1982). The NE-trending weakness we find in Pennsylvanian Period coal mine roof rocks trends parallel to fold axes in those rocks. Since Devonian and Pennsylvanian fold axes are symmetrical, it appears the NE-trending MIF observed in Pennsylvanian rocks (trends indicated in red, Figure 15) are related to a common Pennsylvanian/Devonian “mechanical fabric.”

Consequently, these results suggest the interesting conclusion that there is a subtle, but pervasive, NE-trending mechanical weakness in coal mine roof rocks across West Virginia which is associated with, and perhaps the mechanism for, a NW-oriented swelling strain anisotropy that is also state-wide. Pennsylvanian rocks experienced the same folding event as Devonian rocks. Also, Pennsylvanian coals exhibit a well-developed cleat and a well-developed optical anisotropy. It follows that fine-grained rocks (shales and mudstones) in Pennsylvanian rocks would display a similar fabric (Evans, M.A., 2012). Therefore, Devonian shale rocks may very well possess a NW-SE biaxial swelling response to moisture adsorption as do Pennsylvanian rocks. This may have implications for shale gas developers in the Appalachian Basin challenged by rapid aperture closure rates along horizontally-drilled induced fracture networks in Devonian rocks like the Marcellus.

IMPLICATIONS FOR MINING

Instability in shale roof strata takes many forms, but, fundamentally, it occurs when separation develops 1) between horizontal weaknesses along bedding planes and, 2) along vertical and angled fractures where friction reduction along impinging surfaces initiates an unraveling process that once started can culminate in massive roof failure (Figure 16).



Figure 16. An example of coal mine shale roof unraveling between roof bolts in response to internal strains and reduction in rock strength caused by moisture adsorption.

Weak shale roof rock, when it is first exposed and supported with standard patterns of roof bolts, may initially give the appearance of being adequately supported. However, clay-rich roof shales are susceptible to moisture damage over time (Figure 17). The moisture may be introduced either from recurring seasonal humidity changes brought into the mine via the ventilation system (Unrug and Nandy, 1988; Stateham and Radcliffe, 1978) or by roof bolt penetration into overlying wet strata that introduces dripping water into the annulus where it then distributes moisture throughout the rock mass. The result is a loss of rock strength that appears as 1) lost cohesion and fracture surface friction, and 2) a reduction in the elastic modulus (softening) of the rock. Eventually massive roof failures can develop, in intersections and elsewhere, where pressures from swelling strain, combined with the reduction in rockmass strength, culminate in an unraveling and “dead weight” roof that no longer has sufficient internal strength to remain supported by vertical supporting bolts.

Successfully supporting moisture-sensitive weak shale roof in critical entries over the long term remains one of the unsolved challenges in coal mining. While it is unlikely that a magic bullet exists that can fix the problem, supplementing our understanding of the geo-mechanical behavior of weak shale roof rock facilitates management of the problem. In this regard, the recognition that, when roof rocks cycle through humidity changes, they develop preferential NW-trending swelling strain and preferential NE-trending mechanical weakness gives insight that might inspire refinements that can enhance the effectiveness of new and existing roof control strategies.

Variation in Moisture-Induced Swelling Strain, by Lithology Type, compared to In-situ Tectonic Strain

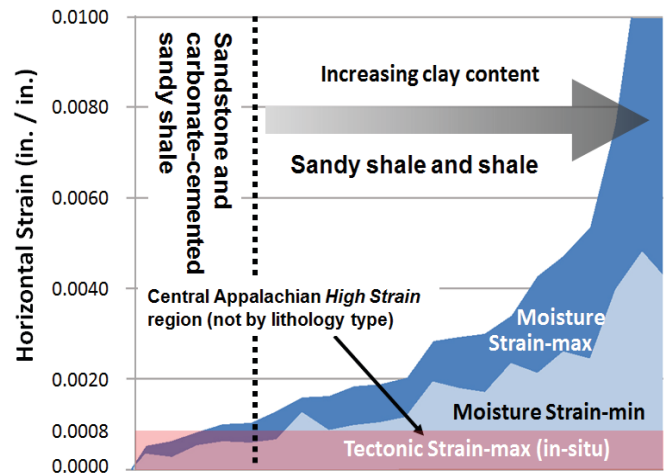


Figure 17. Swelling strains increase with apparent clay content. Shale rocks exhibit moisture-induced strains exceeding calculated in-situ tectonic horizontal strain for the Central Appalachian High-Stress Region (refer to Figure 8).

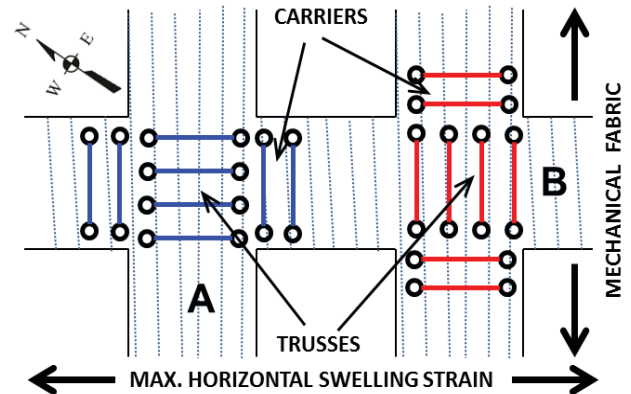


Figure 18. Roof trusses installed in 4-way intersections to arrest unraveling of moisture-sensitive weak shale roof might be more effective (A) installed perpendicular to the NE-trending strength anisotropy (mechanical fabric) versus (B) in parallel with it.

An example of this might include unconventional installations of trusses, whereby lateral compression is directed where it does the most good (Figure 18).

Roof trusses are sometimes installed in intersections to control weak shale ground movement. When trusses are installed after moisture damage has already occurred, they function basically as a “sling.” However, if trusses are installed early, before damage occurs, they are more effective, and their effectiveness may be further enhanced by installing them normal to the NE-trending “mechanical fabric.” Further, trusses that are so oriented are automatically in alignment with maximum horizontal swelling strain, thereby increasing rock compression by increasing truss tension. The idea is that the lateral compression of the truss helps keep vertical joints and fractures locked together.

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In theory, trusses installed (A) perpendicular to the mechanical weakness direction (MIF) would tend to maximize their effectiveness in reducing roof unraveling during humidity uptake. Conversely, trusses installed (B) parallel to the NE-trending fabric (MIF) would provide less benefit.

In 4-way intersections where entries have been driven in a NE-SW direction, conventional truss installation (across the entries and with carriers installed in cross-cuts) would be appropriate. However, for intersections in which entries are developed NW-SE, a non-conventional installation, one in which trusses are aligned in the direction of the entries, might be more effective. Headings driven N-S or E-W would be installing trusses oblique to the fabric so would see comparatively little difference.

Solid bar trusses that are 7/8 inch in diameter or greater are typically more desirable in this application than cable trusses because they are more rigid. During fall and winter months when humidity subsides and swelling strain reverses, it would be useful if the trusses could be re-torqued, as needed, in preparation for the next spring/summer humidity cycle.

While this is a hypothetical example, and has not been tested to our knowledge, it illustrates one way that design enhancements incorporating knowledge of directional rock anisotropy in mine roof might be put into practice.

CONCLUSIONS

Tests of oriented roof rocks in underground West Virginia coal mines show the following:

1) Evidence of a regional NE-trending fracture fabric in roof rocks that is coupled with a dynamic, asymmetrical horizontal stress caused by NW-SE swelling strain in response to a moisture uptake. The swelling strain anisotropy appears to be related to preferential hydration and swelling of clay minerals along NE-trending fractures, but more study will be needed to confirm this conclusion.

2) The NE-trending fracture fabric appears in both Pennsylvanian and Devonian rocks and is likely related to post-Alleghanian stress relaxation facilitating tensile crack development oriented sub-parallel to anticline fold axes. Moisture adsorption exploits these unhealed fractures, weakening roof rocks preferentially in a NE direction as it induces a 2:1 biaxial expansion in a NW-SE direction.

3) Because their mechanical fabrics are similar, the NW-trending horizontal swelling strains may exist in Devonian rocks as well. This has implications for shale gas developers in the Appalachian Basin who are challenged by the rapid aperture closure rates along horizontally-drilled induced fracture networks in Devonian rocks like the Marcellus.

4) The concept of a NW-oriented swelling strain anisotropy perpendicular to a regional mechanical weakness trend may be a useful new addition to the geo-mechanical toolkit.

ACKNOWLEDGEMENTS

The author would like to thank the West Virginia Office of Miner's Health, Safety and Training roof control inspectors Regions 1,2,3,4, Jack Parker, M.A. Evans, M.J. Sapko, B.M. Blake: West Virginia Geological and Economic Survey; and the participating coal producers for their assistance in this project.

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A Comprehensive Analysis of Cable Bolt Anchorage Characteristics

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ABSTRACT

Cable bolts anchored with traditional two component polyester resin cartridges have successfully supported difficult roof conditions in underground mines. Resin cartridges provide a cost-effective cable bolt anchoring system with consistent anchorage performance characteristics when properly installed. The most commonly used cable bolts in the U.S. mining industry are 270 ksi (1,862 MPa), 0.7 and 0.6 inch (18 and 15 mm), seven-strand smooth cable with resin mixing bulbs (also known as bird cages), and 40- and 30-ton nominal capacity, respectively. However, there are other types of cable bolt anchoring systems aside from bird cages. Recently, many questions have been raised regarding the anchorage characteristics of various types of cable bolts. Keystone Mining Services, LLC (KMS), the affiliated engineering company of Jennmar Corporation, conducted a variety of controlled tests developed to closely model actual underground installations. These tests compared the performance of three commonly used cable bolt varieties, including plain, bird caged and indented seven-strand cable. Details of the laboratory testing and comparative analysis of the results are provided.

INTRODUCTION

Traditional cable bolts anchored with two component polyester resin cartridges are a proven and cost-effective cable bolt anchoring system with consistent performance characteristics when properly installed. Non-tensioned and tensioned cable bolts are now used for both primary and supplemental bolting. Cable bolts are typically installed in either a 1 in (25.4 mm) or 1-3/8 in (35 mm) diameter drill hole with 4 - 5 equivalent feet (1.2 - 1.5 m) of two-component polyester resin (Faulkner, 2012). With the introduction of various new cable supports to the U.S. mining market, tests were conducted to compare the performance characteristics of each support. These tests were highly controlled to limit any unforeseen variables from affecting the results. These tests were developed to simulate actual installations in an underground mine using two-component polyester resin cartridges.

These tests compare the three commonly used cable bolt varieties including plain, bird caged, and indented seven-strand cable, and will compare the following performance characteristics for each respective cable:

- Anchorage capacity per linear length of encapsulation;
- Angle bolt capacity;
- Effects of reverse spinning on installation; and
- Effects of long-term creep.

TEST SAMPLES

All tests were conducted using 270 ksi (1,862 MPa), 0.6 in (15 mm) diameter, left-hand lay cable in plain, bird caged and indented styles. Samples of these cables are shown in Figure 1. The plain and bird caged samples used the Jennmar Cable Bolt Safe-T-Cap while the indented samples used a steel end button. The end configurations were cut off of all samples and therefore did not affect the length of encapsulation or the capacity of the anchorage for each sample. The lay of the cable can be determined by the direction in which the outer strands of the cable are wrapped around the center king wire. For left-hand lay cable, the outer strands wrap in a counter-clockwise direction, down the length of the cable away from the viewer. The most widely used cable type in the U.S. is left-hand lay cable.



Figure 1. Test Samples, top to bottom: indented cable, plain cable with a single bird cage and plain cable.

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ANCHORAGE CAPACITY

The anchorage capacity of grouted cable bolts was determined by performing tensile tests on cable samples with various lengths of resin encapsulation. These tests were conducted to determine the minimum amount of resin anchorage needed to reach the ultimate strength of each respective cable type. A standard two-component, 60-second J-LOK resin cartridge was used as the anchorage component. Standard operating procedures were followed to install the cables and resin to simulate actual field installations. A slow gel time was used to limit factors that may affect the final results, as a faster acting gel time may have incurred.

In order to limit the variables affecting the cable anchorage tests, 1 in (25 mm) ID by 1.75 in (45 mm) OD steel pipe was used as the anchoring device and to simulate the drill hole. The steel pipe was internally threaded to a 27 mm x 3 metric thread in accordance with ASTM F432 A3.1.2 for Chemical Grouting Materials (ASTM International, 2010). The 3/8 inch (10 mm) steel wall thickness provided the necessary strength and confinement needed to guarantee that the resin and cable bolt interface would fail before the drill hole and resin interface, and provide adequate strength during the tensile tests (Hyett, Bawden, & Reichert, 1992). Failure of a grouted anchorage by load transfer failure will occur by one of four methods: failure at the cable-to-grout interface; failure at the grout-to-rock interface; failure through the grout column; or by failure through the rock around the borehole wall (Thomas, 2012).

The samples were prepared at the Jennmar Research and Development Laboratory facility with a Jennmar test lab bolter developed by Fletcher. The bolter is shown in Figure 2. The Jennmar bolter is custom designed to control and record a wide range of parameters. Either clockwise (forward spin) or counter-clockwise (reverse spin) rotation was used to mix the resin. The clockwise and counter-clockwise tests were conducted as some ground support manufacturers and products recommend counter-clockwise mixing of the resin during installation. No published tests were found comparing the two installation methods so a comparison was necessary. The generally accepted method in the U.S. is clockwise rotation for left-hand lay cable.

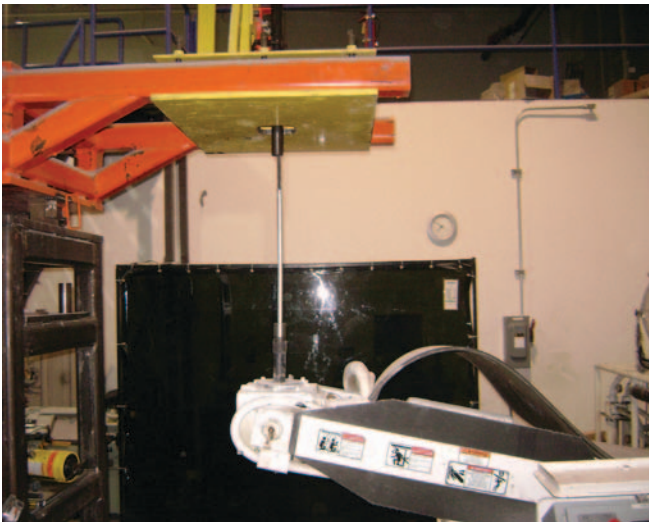


Figure 2. Insertion of a cable sample into a 48" bore tube with a Jennmar test lab bolter.

After the samples were installed and allowed to cure for a minimum of 48 hours, ultimate tensile tests were conducted as shown in Figure 3. Even though 60-second J-LOK resin can obtain the maximum capacity of the cable support within minutes after installation, 48 hours was used as a way to limit variables affecting the test results. The cable samples measured 36 in (91 cm) in length plus the encapsulation length for each respective cable type. A standard cable bolt housing with a two-piece wedge assembly and 1-1/8 inch square drive nut was set on the opposite end. A 0.6 in (15 mm) diameter 270 ksi (1,862 MPa) cable is required to have a minimum ultimate tensile strength of 58,600 lbf (261 KN) as per ASTM F 432-10. This specification requires that all cable bolt assemblies must obtain 90% of the minimum ultimate tensile strength, which is 52,740 lbf (235 KN). The specification for a Jennmar produced cable bolt assembly is 95% of the minimum ultimate tensile strength, or 55,670 lbf (247 KN) as per in-house quality control procedures. The 95% minimum ultimate tensile strength was adopted to create a higher standard as compared to industry minimum standards. During the tensile tests, the samples were loaded at a consistent rate until a cable strand failure occurred or a load transfer failure occurred between the cable and resin anchorage interface. The average anchorage capacity results for each linear length of encapsulation are shown in Figure 4 and Table 1. Four samples were tested for each individual category to develop the average anchorage capacity.



Figure 3. Tensile test of plain cable sample with 12" of resin encapsulation.

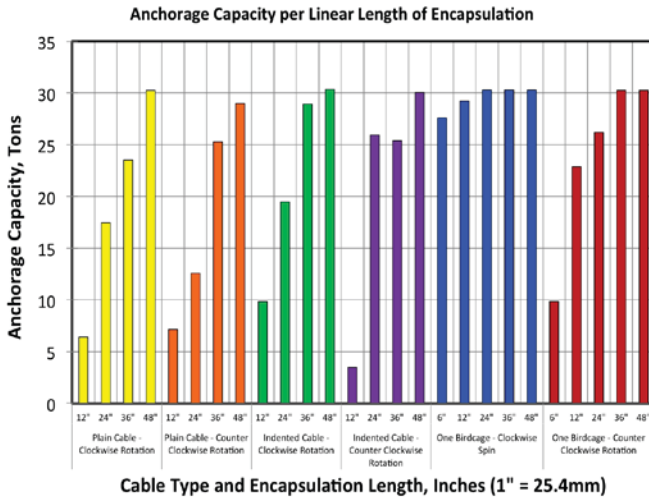


Figure 4. Average anchorage capacity per linear length of capsulation (n=4).

As detailed in Table 1, a cable bolt with one birdcage installed with clockwise rotation and 6 in (15 cm) of resin encapsulation exceeds the ASTM minimum break strength of a cable bolt by 4%. For a plain cable installed with clockwise rotation a minimum of 48 in (1.22 m) of encapsulation was required and the indented cables installed with clockwise rotation required 36 in (0.91 m) of resin encapsulation to exceed the minimum breaking strength specified by ASTM. For counter-clockwise installations, the single birdcage, plain and indented cable samples each required 36 in (0.91 m), 48 in (1.22 m), and 48 in (1.22 m) of resin encapsulation respectively to exceed the minimum capacity.

SAFETY FACTOR FOR RESIN ENCAPSULATION

The objective of the safety factor for resin encapsulation (SF_{re}) is to categorize the anchorage capacity and assign a safety factor for each individual arrangement. The SF_{re} does not rate the cable, but rates the total amount of anchorage capacity available. The SF_{re} is based on the obtained load or potentially attainable load, and is compared against the minimum design load of the cable which is 58,600 lbf (261 KN).

To determine the SF_{re} two equations are required. Equation 1 is used when a load transfer failure ("slippage") occurs between the resin and cable bolt anchorage. Equation 2 is used when a strand failure occurs which will take into account the additional encapsulation length of the resin anchorage after the ultimate capacity has been reached.

$$SF_{re} = \frac{\text{Obtained Load}}{\text{Designed Load}} \quad (1)$$

Equation 1. Safety Factor for Resin Encapsulation when a load transfer failure occurs.

$$SF_{re} = \frac{\text{Obtained Load} + ((c - p) * l)}{\text{Designed Load}} \quad (2)$$

Equation 2. Safety Factor for Resin Encapsulation when a strand failure occurs.

Equation 2 utilizes data obtained from Equation 1 to determine the SF_{re} . The variables in this equation are as follows.

- c - The encapsulation length of the sample.
- p - The encapsulation length when the SF_{re} is equal to one.
- l - The load capacity per inch of encapsulation calculated from a 12" cable sample.

Analyzing the safety factor chart (Figure 5), one birdcage rotated in the clockwise direction achieved the highest range of safety factors. As little as 12" (305 mm) of resin encapsulation obtained a safety factor of 1 and a SF_{re} of 1.69 was obtained with 48 in (1,220 mm) of encapsulation, which is the highest safety factor for any system tested. For the plain and indented cable samples with either clockwise or counter-clockwise rotation required a minimum of 48" (1,220 mm) of resin encapsulation to achieve a safety factor above 1. In underground systems, three birdcages are standard with 48 in (1,220 mm) of encapsulation. The safety factor for this system was taken as the single birdcage multiplied by three which equates to a 5.07 safety factor. In the previous section, the difference of clockwise and counter clockwise rotation was discussed. The safety factor chart correlates to those findings that counter-clockwise rotation produces inconsistencies, thus achieving a lower safety factor of 3.76 with 48 in (1,220 mm) of encapsulation.

ANGLE BOLT CAPACITY

The objective of the angle bolt capacity test was to simulate cable bolts installed at an angle over the rib, such as in a cable truss as shown in Figure 6 or as a stand-alone angled bolt. The test fixture shown in Figure 7 is capable of aligning the bearing plate of the cable at either a 30- or 45-degree angle offset from the cables linear axis by interchangeable face plates.

Six samples each of standard plain and indented cable were tested. The plain cable with birdcages was not tested because the birdcage does not affect the angle bolt capacity as no resin is involved. The interchangeable face plates utilize T-5 channel and T-3 channel plates to simulate how the system would be installed underground, such as with a B-Rigid truss system. Tension was steadily applied to the cable samples by a hydraulic ram and the peak load was recorded once the first failure occurred in the strand. The two types of cables were both tested at 30 and 45 degrees and the results are shown in Table 2. The capacities of the plain cable at 30 and 45 degrees and indented cable at 30 degrees achieved loads above the ASTM minimum standard for cable bolt assemblies. The indented cable at 45 degrees only reached 45,500 lbs (202 kN), or only 78% of the ASTM minimum standard.

Effects of Counter-Clockwise Rotation

As stated previously, the samples were installed with J-LOK resin and mixed with either clockwise or counter-clockwise rotation. The difference in the two directions is that when mixing the resin in the clockwise direction, the resin will be augered to the

Table 1. Anchorage Capacity per Linear Length of Encapsulation (n=4).

Anchorage Capacity per Linear Length of Encapsulation				
Sample Type	Rotation Direction	Encapsulation Length	Average Peak Capacity, Tons	Percentage of ASTM Minimum Break Strength ¹
Plain Cable	Clockwise	12"	6.4	22%
		24"	17.5	60%
		36"	23.5	80%
		48"	30.3	103%
	Counter-Clockwise	12"	7.1	24%
		24"	12.6	43%
		36"	25.3	86%
		48"	29	99%
Indented Cable	Clockwise	12"	9.8	34%
		24"	19.5	66%
		36"	28.9	99%
		48"	30.4	104%
	Counter-Clockwise	12"	3.5	12%
		24"	25.9	89%
		36"	25.4	87%
		48"	30	103%
One Birdcage	Clockwise	6"	27.6	94%
		12"	29.2	100%
		24"	30.3	103%
		36"	30.3	103%
		48"	30.3	103%
	Counter-Clockwise	6"	9.8	34%
		12"	22.9	78%
		24"	26.2	89%
		36"	30.3	103%
		48"	30.3	103%

¹ ASTM minimum break strength of a 0.6 in(15mm) cable is 29.3 tons, per ASTM F432-10.

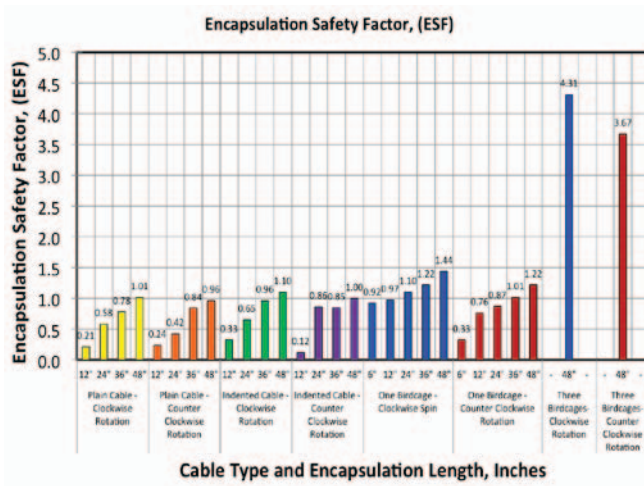


Figure 5. Encapsulation Safety Factor.

back of the drill hole due to the lay of the cable. Essentially, the resin is compressed and compacted into a tighter annulus. When mixing the resin in the counter-clockwise direction, the resin has the potential to be augered to the open end of the drill hole which has a tendency to create voids within the resin anchorage as shown with the indented cable sample in Figure 8. These voids can reduce the linear length of encapsulation and anchorage capacity. Figure 9 shows a sectioned bore tube sample of a single birdcage with clockwise rotation that was sectioned at the birdcage.

A comprehensive comparison between clockwise and counter-clockwise rotation is detailed in Table 3. The peak capacity and displacement at peak capacity differences were determined by Equations (3) and (4).

$$\left(\frac{Peak\ Capacity_{Counter-Clockwise} - Peak\ Capacity_{Clockwise}}{Peak\ Capacity_{Clockwise}} \right) * 100\% \quad (3)$$



Figure 6. B-Rigid truss system detailing angled cable bolts.



Figure 7. The Off-Axis test fixture allows cable to be aligned at 30 or 45 degrees as load is applied.

$$\left(\frac{\text{Displacement at Peak Capacity}_{\text{Counter-Clockwise}} - \text{Displacement at Peak Capacity}_{\text{Clockwise}}}{\text{Displacement at Peak Capacity}_{\text{Clockwise}}} \right) * 100\% \quad (4)$$

Table 2. Average test results for each offset angle. (n=6).

Off-Axis Capacity Results			
Test Cable	Offset Angle, degrees	Average Peak Load, lbs (kN)	Percentage of ASTM Minimum Break Strength ²
Plain	45°	55,000 (245)	94%
	30°	60,500 (270)	103%
Indented	45°	45,500 (202)	78%
	30°	59,500 (264)	102%

²ASTM minimum break strength of a 0.6 in (15 mm) cable is 29.3 tons, per ASTM F432-10.



Figure 8. Void detected in sectioned bore tube sample of indented cable with counter clockwise rotation.

Table 3 shows that for the indented and plain cable samples with an encapsulation length of 12 in – 36 in (15.2 cm - 91 cm), the capacity results proved to be inconsistent and for 48” of encapsulation the capacities were about equal. For the birdcaged cables with shorter encapsulation lengths of 6 in - 24 in (15.2 cm - 61 cm) and counter-clockwise rotation, the peak capacity was reduced by 14 to 64 percent. For the samples with 36 in and 48 in (91 cm and 122 cm) of encapsulation, the peak anchorage capacities were greater than the ultimate strength of the cable samples.

When comparing the average displacement at peak capacity results, it should be noted that only the samples that exceeded the minimum designed capacity of the cable, 58,600 lbf (261 KN), were considered. If none of the samples in the test set reached this requirement, comparisons are not possible. After examining the results, the counter-clockwise samples with encapsulation lengths of 24 in and 36 in (15.2 cm and 91.4 cm) tended to require more displacement than the clockwise rotated samples. With 48 in (122 cm) of resin encapsulation, the opposite was noted. The clockwise



Figure 9. Sectioned bore tube sample of single birdcage sample with clockwise rotation (sample was cut at the birdcage).

EFFECTS OF LONG-TERM CREEP

The creep test was designed to maintain a constant load as a function of time on underground support systems with the ability to adjust the load, as seen in Figure 10. Standard “pull tests” conducted in a lab or in the field utilize a high-capacity tensile testing machine or portable pull test equipment. Both methods apply the desired load within seconds or minutes without determining the supports ability to handle long term loads. The creep test provides a key advantage by allowing a load to be applied to a sample for weeks or months at a time. This load can be steadily increased or decreased in accordance to the test procedure. This advantage allows observations of how holding the system at critical loads over time affects the overall performance of the system.

During the tests, load was applied by a “building method”, which means load was steadily increased over an extended period of time. The first level was 20,000 lb (89 kN), then 30,000 lb (133 kN), increasing to 34,000 lb (151 kN), 40,000 lb (178 kN), and ending at 50,000 lb (222 kN). Each load level was held for three to five days after no increases in deflection was detected, totaling a 25-day test period for each sample.

Three samples were subjected to the creep test. The samples consisted of indented cable with no birdcages, plain cable with one birdcage, and plain cable with three birdcages, all of which were encapsulated in 40 in (1.0 m) of J-LOK resin with standard

Table 3. Comparison between clockwise and counter-clockwise rotation.

Comparison of Clockwise to Counter-Clockwise Rotation					
Sample Type	Encapsulation Length, Inches	Peak Capacity Difference, %	Average Displacement @ Peak Capacity, Inches		
			Clockwise Rotation	Counter-Clockwise Rotation	Displacement Difference, %
Plain Cable	12”	11%	-	-	-
	24”	-28%	-	-	-
	36”	8%	2.06”	2.36”	15%
	48”	-4%	2.16”	1.47”	-32%
Indented Cable	12”	-64%	-	-	-
	24”	33%	-	-	-
	36”	-12%	2.23”	2.20”	-1%
	48”	-1%	2.2”	1.73”	-21%
One Birdcage	6”	-64%	1.83”	-	-
	12”	-22%	1.83”	-	-
	24”	-14%	1.82”	3.30”	81%
	36”	0%	1.8”	2.30”	28%
	48”	0%	1.53”	1.40”	-8%

rotated samples required more displacement at peak capacity than the cables that were installed with counter-clockwise rotation. This condition may be a result of the counter-clockwise rotation causing an auguring effect which can pump the resin to the open end of the drill hole.

clockwise rotation. The sample with three birdcages was tested because it is a standard system used in underground mines. The cables were installed in a 1 in (25.4 mm) ID steel pipe that was “rifled” to simulate a drill hole underground.

Displacement readings were continuously measured at two locations to monitor movement in the cables. The first location was



Figure 10. Long term creep tester.

the end displacement of the cable (top of the borehole), which monitored the “pull” from the resin (Figures 11 and 12). The second location determines the housing and wedge assembly movement of the cables by measuring the base of the cable to a specific stationary point.

The final results indicated that three birdcages in 40 in (1.0m) of resin produced an end displacement of just 0.004 in (0.10 mm), one birdcage produced an end displacement of 0.190 in (4.80 mm), and the indented cable produced 0.80 in (20.3 mm) at 50,000 lb (222

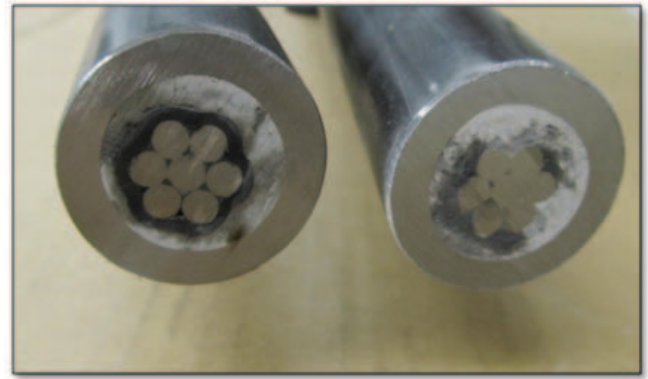


Figure 11. Creep test samples before testing. Left to right: plain cable with one birdcage and indented cable.

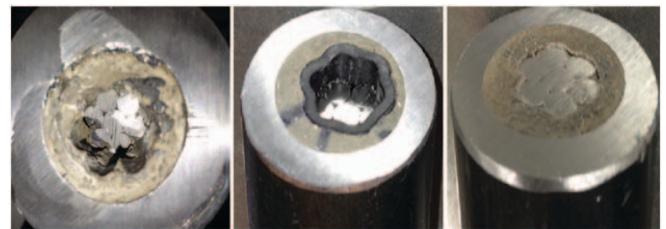


Figure 12. Creep test samples after testing. Left to right: Indented plain cable with one birdcage, and plain cable with three birdcages.

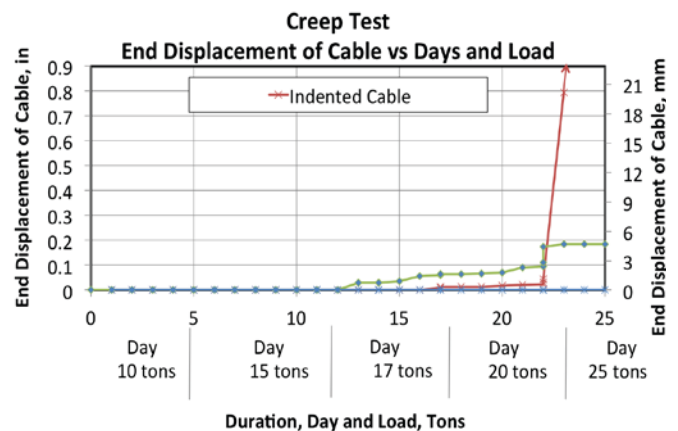


Figure 13. Creep test - end displacement of cable samples as a function of time and load.

kN). On Day 2 of the 50,000 lb (222 kN) load level, the indented cable pulled 0.750 in (19 mm) from the resin (Figure 12). This excessive movement indicates that creep occurred in the support and the creep test had to be stopped on day 23 for this sample. The sample with three birdcages, which is a standard cable anchorage, incurred minimal movement from within the resin, producing a lower cable deflection. Future testing is planned to determine the affects of creep on indented cables encapsulated with 48 in and 60 in (1.20 m and 1.52 m) of resin.

SUMMARY AND CONCLUSIONS

It was determined that traditional cable bolts incorporated with birdcages are a proven system with consistent anchorage performance characteristics when properly installed. As little as 6 in (15.2 cm) of resin encapsulation is needed to break the cable with a single birdcage. The indented and plain cable samples required 36 in of encapsulation to break the cable. The method of counter-clockwise rotation of a cable bolt on installation substantially reduced the anchorage capacity of each respective system when compared to clockwise rotation. These reductions are confirmed by the anchorage capacity per linear length of encapsulation test results. Therefore, it is recommended that counter clockwise rotation should not be used due to its reduced anchorage capacity. The angle bolt capacity test determined that a plain cable is more effective than the indented style cable. With an angle of 30 degrees, the capacities were relatively equal. However, with an angle of 45 degrees, the plain cable obtained a 20% higher capacity than the indented cable. The long-term creep tests again showed that the samples with birdcages are more effective than the indented cables. Future testing is planned to increase the sample set of the creep test results and to develop a more definitive conclusion.

ASTM F432-10 currently requires short encapsulation tests for thread-deformed, reinforcing bar; however, no such specification exists for cable supports. After the analysis and comparison of these results, the industry should consider the development of a specification for encapsulation tests of grouted cable bolts. With the influx of various cable supports to the mining industry and the results of the tests shown here, a specification would permit the end user to know the actual capacity associated with each type of cable bolt being installed into their mines. The developed strength index would compare each type of cable support by a consistent and ASTM certified test procedure.

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Investigation of Pillar Loading Considerations in Determination of Pillar Stability Factors for Longwall Gateroad Design

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ABSTRACT

Recent field data providing measured pillar loading from a deep western mine revealed that an increased load transfer distance existed compared to expected values (Larson et al., 2012). Additionally, recent work by Tulu and Heasley (2012) has shown that the abutment angle in deep longwall mines can be significantly reduced as the depth of mining increases. The load transfer distance and abutment angle are both used as parameters to estimate pillar loading in longwall design. To investigate the potential implications of these observations on pillar loading in deep longwall mines, a parametric study using ALPS loading estimation procedures was performed. A supplemental study was conducted using LaModel to further investigate pillar loading at the tailgate. The study found that the abutment angle is particularly important for panels with supercritical geometries. As panels reach a subcritical geometry with increasing depth, the primary influence of the abutment angle is reduced and matched by other factors, including load transfer distance for tailgate loading. An extended load transfer distance was shown to potentially reduce loading on pillars adjacent to the longwall as the loading is spread over a larger area. The results of these studies indicate a need for further field verification and research toward regional geologic influences on pillar loading and how these may impact ground stability.

INTRODUCTION

Recent work by Larson and Whyatt (2012) indicates that the load transfer distance or abutment extent, D , at a deep western longwall coal mine, may, in fact, be nearly four times the estimated distance indicated by the Peng-Chiang equation (1984). The Peng-Chiang equation,

$$D=9.3\sqrt{H} \quad (1)$$

where H , the depth of overburden in feet, is widely used to estimate the load transfer distance in US coal mines (Mark, 1987, Peng and Chiang, 1984, Tulu and Heasley, 2012).

Larson and Whyatt (2012) used borehole pressure cells (BPC's) installed along two sets of gateroads on either side of an advancing panel to determine the distance in advance of the face that load was experienced by the pillars. The load transfer distance was

calculated from these measurements. The average value between two sets of gateroads was determined to be 538 meters (1,764 feet). This number is significantly larger than the estimated distance of 135 meters (442 feet) calculated by the Peng-Chiang equation. Visual observations made during ground deterioration mapping support these measurements, and indicated that visible load was apparent at 305 meters (1,000 feet) outby the advancing longwall face. The exceptional load transfer distance at this mine may be due to the local/regional geology compared to the case studies used to generate the empirically derived Peng-Chiang equation. The stiff, competent stratum at the case mine may have the capacity to transfer load for a greater distance than those used to develop the Peng-Chiang method.

Additionally, a recent study by Tulu and Heasley (2012) of 30 mines in the United States and Australia, whose overburden depths are greater than 275 m (900 feet) indicates that the abutment angle, β , values ranged between 6° and 18° , with a mean value of 10° . This study serves to augment observations made by Mark (1992) of abutment angle in mines ranging in depth between 455 and 760 feet, where the average angle was determined to be 21° . Abutment loading depends heavily on the abutment angle, β , which defines the wedge of overburden suspended over the gob (Figure 1). The weight of this wedge must be supported by adjacent pillars and regions of unmined coal in the abutment area as the wedge is undermined.

Furthermore, their work shows that abutment angle values potentially decrease with increasing depth. As the relative criticality of a panel is heavily dependent upon the abutment angle, this observed reduction of the abutment angle serves to delay the transition to a subcritical geometry with increasing depth. An empirical relationship tying β to depth of overburden, valid for depths between 274 m (900 ft.) and 625 m (2,050 ft.), was determined as:

$$\beta=21^\circ (H \text{ (ft)}/900)^{-1.59} \quad (2)$$

These findings raise the question of how regional variations in load transfer distance and abutment angle might affect pillar loading as mining moves into deeper deposits. This paper examines the potential impact of these parameters on pillar loading as predicted by the ALPS load estimation methodology (Mark,

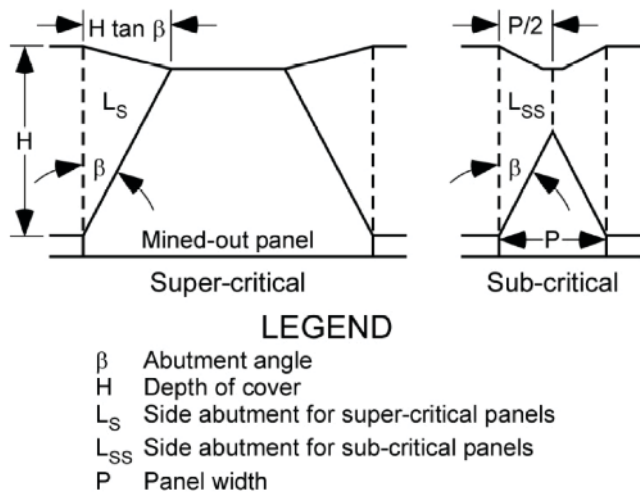


Figure 1. Abutment angle concept (Mark, 1992).

1987), supplemented by the LaModel boundary element software (Heasley, 2010) within the context of two hypothetical longwall panels. This type of sensitivity study may guide the analyst in designing ground response studies which would most effectively incorporate the relevant ground behavior characteristics. This is important since ground response instrumentation studies can be expensive and time consuming, yet insight into the relative influence of input parameters on pillar loading may have a significant impact on design considerations. It is important to note that these investigations examine only the pillar loading component of gateroad stability, and other mitigating components, such as innate pillar capacity and roof/entry support are not taken into account.

THE ALPS METHODOLOGY

The implications of abutment angle and load transfer distance variations on pillar loading was evaluated using the Analysis of Longwall Pillar Stability (ALPS) methodology (Mark, 1987; 1992; Mark et al., 1994). The most recent development in ALPS was the inclusion of Coal Mine Roof Rating (CMRR) as part of the analysis (Mark et al., 1994). The ALPS methodology was verified against successful and unsuccessful longwall extraction cases at 44 US coal mines which has been key to the acceptance of the method by the mining industry. The ALPS methodology provides a useful tool for the analysis of longwall gateroad pillar loading. The following is a brief summary of the relevant aspects of the ALPS methodology.

The Abutment Angle

The abutment angle (β) is effectively a measure of how easily strata overlying the seam caves into the gob, and its value may be affected by regional geology. Clearly, strong sandy strata will cave differently than a weak, layered mudstone unit, and the resulting β angle will vary accordingly. Weak units, as an example, are likely to cave more completely, resulting in a smaller volume of suspended overburden and hence, a smaller abutment angle. Mark (1992) found β angle values ranging from 10° and 25° , with an average value of 21° , in the field studies used to formulate the ALPS procedure. As previously stated, a recent study of mines

deeper than 275 m (900 feet) indicates that at greater depths, β angle values ranged between 6° and 18° (Tulu and Heasley, 2012) and that there is an apparent dependency between β and depth.

The Load Transfer Distance

ALPS makes use of the Peng-Chiang equation (equation 1) to estimate the extent of the load transfer (D) adjacent to a longwall panel. The shape of the load transfer curve is based on field observations of load decay with distance from the edge of the longwall panel (Mark, 1987).

Pillar Loading

Pillar loading in ALPS is determined using several simple calculations for key locations relative to the longwall panel extraction. These locations, shown in figure 2 (after Mark, 1990), include development loading, headgate loading, bleeder loading, isolated loading, and tailgate loading. Pillar loads at these locations are a mix of initial development loads, estimated by the tributary area method, and abutment loading. Abutment loads from mined-out areas are added to the development loading in varying degrees as mining progresses. Abutment loads are estimated by a relatively simple set of equations incorporating a number of empirical factors (Mark, 1987).

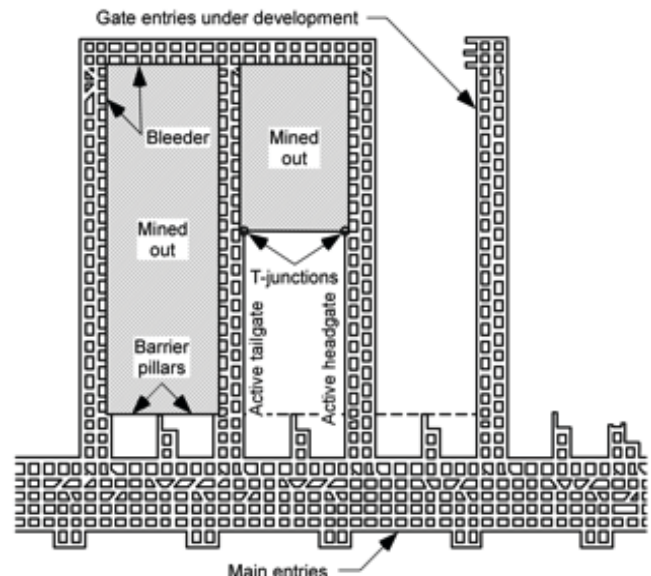


Figure 2. Loading condition locations, modified after Mark (1990).

Abutment Descriptions: Inby the face, load transferred to the adjacent gateroad pillars and adjacent unmined panel is called the side abutment load, while load transferred to the longwall face and adjacent pillars is called the front abutment load. Mark (1987) specifically defines “front abutment” as the abutment loads “experienced by the pillars at face ends,” or “the abutment load when the distance to the face is zero”. These headgate and tailgate locations are represented by the “T-junctions” in Figure 2.

Bleeder Pillars: Bleeder pillar loading, L_B , is defined as the load to which pillars will be subjected while protecting the bleeder

entries along the perimeter of a longwall district, where it is “subjected to the development load and the first full side abutment” as shown in Figure 2 (Mark, 1987). L_B is determined by the development loading and a fraction (R) of the full side abutment. This abutment fraction, R, is defined by Mark (1987) as “the percent of the abutment load applied to the chain pillars,” and given by the equation

$$R=1-((D-W)/D)^3 \quad (4)$$

where W is the width of the pillar system. R is therefore sensitive to changes in load transfer distance, D. Consequently, headgate and bleeder loading calculations, which include R, will also be sensitive to D.

Bleeder loading is given by

$$L_B=L_D+(L_S)(R), \quad (5)$$

Where L_D represents the development loading and L_S represents the full side abutment loading.

Headgate Pillars: Headgate loading (Figure 2) refers to the load on the gateroad bounded on one side by an unmined panel during the advance of the active panel at the junction with the longwall face. Inby the longwall face it is a bleeder loading scenario. It is defined by the sum of the development loading and a fraction of the side abutment loading multiplied by the headgate front abutment factor, F_H . F_H is assumed by ALPS software to be 0.5 based upon pillar stress measurements conducted during the initial development of the ALPS method. Headgate loading is given by

$$L_H=L_D+(L_S)(F_H)(R). \quad (6)$$

Tailgate Pillars: Tailgate loading, described by Mark (1987) as “the most severe longwall loading”, is experienced during the mining of the second and subsequent panels in a district (Figure 2) when the gateroad is bounded on one side by the mined out panel and on the other by the active panel. The gateroads are now subject to a combination of development loading, side abutment loading from the previously mined panel and the front abutment loading from the current panel. Front abutment loading is considered to be a fraction of the side abutment loading. Mark (1987) found the tailgate abutment factor, F_T , to be 0.7 for the single available case study. This calculation departs from headgate and bleeder loading equations by omitting the factor R (which incorporates load transfer distance), since there is no adjacent panel to accept the far-field load. Tailgate loading is given by

$$L_T=L_D+(L_S)(1+F_T). \quad (7)$$

Note that when F_T is equal to 0.7, the calculation of tailgate loading may also be rewritten:

$$L_T=L_D+(L_S)(1.7). \quad (8)$$

HYPOTHETICAL LONGWALL CASES USED IN ANALYSES

Two hypothetical cases were chosen with the intention of approximating different mining geometries and depths, so that

results will illustrate to what degree pillar loading is influenced by variations in the abutment angle and load transfer distance.

Case 1: Shallow Longwall

The first hypothetical case approximates a shallow eastern mine with 152 m (500 ft) maximum cover depth and is modeled after sample problem 6.2.2 in “Analysis of longwall stability” (Mark, 1987). This example is representative of a mining scenario common during the developmental stages of ALPS in the 1980s and early 1990s. It uses a three-entry gateroad system as illustrated in Figure 3. Pertinent parameters as used in the analyses are summarized in Table 1.

Table 1. Parameters for hypothetical Case 1: Shallow Longwall.

Parameter	Value
S_1 (in situ coal strength)	6.2 MPa (900 psi)
h_p (pillar height in m)	1.8 m (6 ft)
H_{max} (maximum overburden depth in m)	152 m (500 ft)
P (panel width in m)	244 m (800 ft)
w_e (entry width in m)	5.5 m (18 ft)
w_p (pillar width in m)	22 m (72 ft)
l_p (pillar length in m)	28 m (92 ft)
D (Peng-Chiang load transfer distance)	63 m (208 ft)

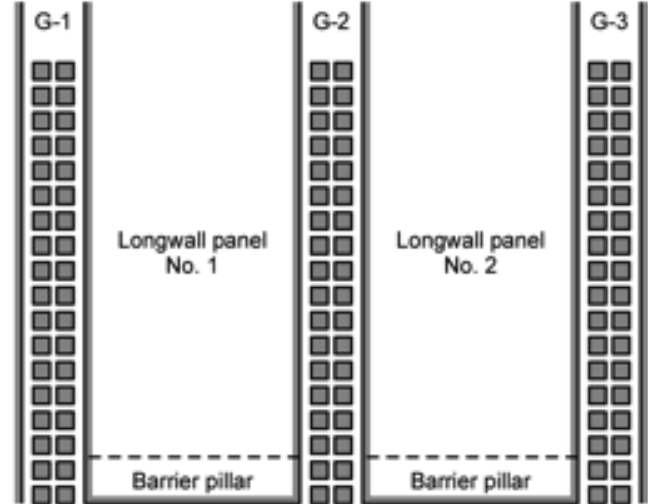


Figure 3. Hypothetical Case 1: Shallow Longwall (Mark, 1987).

Case 2: Deep Longwall

A second case was evaluated to represent a contemporary deep western coal mine similar in design to that investigated by Larson and Whyatt (2012), illustrated in Figure 4. It also uses a three-entry gateroad system, but the overburden depth is significantly greater at 716 m (2,350 ft). Correspondingly, pillar dimensions are also significantly larger as shown in Table 2. The case includes a set of gateroads used as the tailgate of a first panel (i.e. adjacent to a

barrier panel) that are only required to serve as a bleeder following the longwall advance and not as a subsequent isolated tailgate. Due to the presence of the barrier pillar, it experiences a headgate loading condition, despite its role as a tailgate and subsequent bleeder. The second gateroad must serve as a headgate for the first panel and a tailgate and bleeder for the second panel. These pillars must be designed to survive the tailgate loading condition and, as illustrated, need to be significantly larger to accommodate the increased load associated with it.

Table 2. Parameters for hypothetical Case 2: Deep Longwall.

Input	Gateroad 1 (Headgate loading)	Gateroad 2 (Tailgate loading)
S_i (in situ coal strength)	6.2 MPa (900 psi)	6.2 MPa (900 psi)
h_p (pillar height in m)	3 (10 ft)	3 (10 ft)
H_{max} (maximum overburden depth in m)	580 (1900 ft)	610 (2000 ft)
P (panel width in m)	183 (800 ft)	183 (800 ft)
w_e (entry width in m)	6 (20 ft)	6 (20 ft)
w_p (pillar width in m)	23 (75 ft)	52 (170 ft)
l_p (pillar length in m)	55 (180 ft)	55 (180 ft)
D (Peng-Chiang load transfer distance)	123 m (405 ft)	126 m (415 ft)

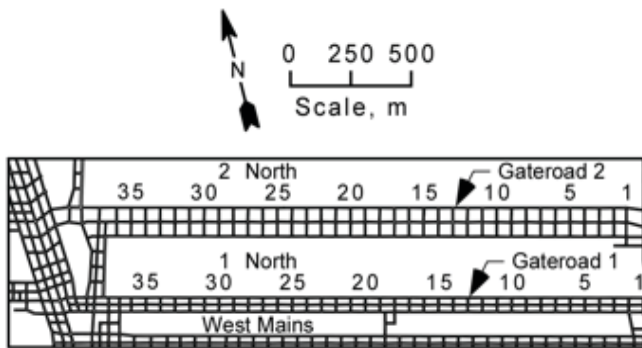


Figure 4. Hypothetical Case 2: Deep longwall (Larson and Whyatt, 2012).

ALPS SENSITIVITY ANALYSIS

A sensitivity study was conducted to investigate the relative dependence of pillar load estimates on the values of the abutment angle (β) and the load transfer distance (D). ALPS pillar loading estimation procedures were used to conduct the analysis of parameter sensitivity. The initial value of the abutment angle was set at 21° and the initial load transfer distance was calculated using the Peng-Chiang equation. Sensitivity was examined by using parameter values 20% less than and 20% greater than the initial values. Results were evaluated by expressing the resulting range of pillar loads as a percentage of the pillar loading using the initial values. Sensitivity is described as follows:

- Insensitive (I), having no effect of loading;

- Slightly sensitive (S), where the examined range of values comprised a percentage of the default loading of up to 5%;
- Moderately sensitive (M), where the examined range of values comprised a percentage of the default loading between 5 and 15%;
- Very sensitive (V), where the examined range of values comprised a percentage of the default loading of greater than 15%.

Headgate, tailgate and bleeder loading were considered individually, as loading of pillars at the different locations may be impacted differently by parameter variation.

Case 1: Shallow Longwall

The results for the hypothetical Case 1 are tabulated in Table 3. Headgate loading is moderately sensitive to changes in β and slightly sensitive to changes in D. Bleeder loading is also moderately sensitive to changes in β , and slightly sensitive to changes in D. Tailgate loading is very sensitive to changes in β , insensitive to changes in D. Note that Case 1 is supercritical for all variations considered.

Table 3. Relative influence of abutment angle and load transfer distance for Case 1: Shallow Longwall.

Parameter	L_H		L_B		L_T	
Abutment angle, β	M*	7%	M	12%	V	21%
Load transfer distance, D	S	>1%	S	>1%	I	0%

*I represents “insensitive”, S represents “slightly sensitive”, M represent “moderately sensitive”, and V represents “very sensitive”.

Case 2: Deep Longwall

The results for Case 2 are tabulated in Table 4. For Case 2 the panel is subcritical for variations of the abutment angle centered around 21° . In this situation the headgate loading is only slightly sensitive to changes in β while tailgate and bleeder loading are moderately sensitive.

In light of the findings of Tulu and Heasley (2012), a series of hypothetical initial β angles were assessed to determine at what point loading would shift from a subcritical regime to a supercritical one. To state the problem more explicitly, at what value of β would $P/2$ become greater than $\tan(\beta) \cdot H$. In this hypothetical case study, the value was established as 10.3° . Therefore, a value of β of 8° was chosen to create a supercritical condition.

Table 4 presents the results for these two selections of initial values of β . When the panel becomes supercritical, headgate, bleeder and tailgate loading are very sensitive to changes in β .

Headgate and bleeder loading is only slightly sensitive to changes in D while tailgate loading is insensitive. Insensitivity to D is the result of the absence of the abutment fraction, R (which incorporates D), in the tailgate loading calculation.

Table 4. Relative influence of abutment angle and load transfer distance for Case 2: Deep Longwall.

Parameter	L_H		L_B		L_T	
Abutment angle β , centered on 21° (subcritical)	S*	4%	M	6%	M	8%
Abutment angle β , centered on 8° (supercritical)	V	17%	V	30%	V	42%
D, (subcritical)	S	>1%	S	1%	I	0%
*I represents “insensitive”, S represents “slightly sensitive”, M represent “moderately sensitive” and V represents “very sensitive”.						

DISCUSSION OF SENSITIVITY ANALYSIS RESULTS

The sensitivity study indicates that the relative influence of each parameter is linked to both mine design and loading condition. The influence of the abutment angle (β) was found to be most significant when the panel is supercritical. This is particularly true in gateroads experiencing a tailgate loading condition. The degree of sensitivity of the parameter β to panel criticality is illustrated by influence on loading above and below the transitional angle, 10.3° . The influence of β changes markedly, from dominating while the panel width is supercritical to relatively minor while it is subcritical. The significance of β in supercritical regimes is also noted to correspond with depth (up to the point at which the panel shifts to subcritical loading) because the abutment load becomes a larger part of the total pillar loading as the depth increases and the weight of the suspended overburden becomes proportionally larger. Therefore, if the panel is deep and supercritical, improving the β angle estimate is a clear priority.

Variations in the load transfer distance (D) have a significant impact on headgate and bleeder loading, but the tailgate loading is unaffected by changes in D. This highlights an interesting feature of ALPS noted earlier, that tailgate pillar loading is completely insensitive to load transfer distance. In addition, the influence of D at the headgate and bleeder locations is diluted because the variation of D spans the gateroad width (W). In ALPS there is no variation in loading with load transfer distance when $D < W$, as all the abutment load is carried by the gateroad pillars.

DEMONSTRATION OF POTENTIAL IMPACT OF LOAD TRANSFER DISTANCE VARIATIONS

The potential impact of load transfer distance variations on pillar loading was further explored using the ALPS design approach for both the shallow and deep mine hypothetical cases. The objective was to demonstrate how these variations would change the pillar loading for the specific cases.

Case 1: Shallow Longwall

For this study the load transfer distance was varied between 52 m (170 ft.) and 283 m (930 ft.), while the abutment angle was held constant at 21° . The Peng-Chiang load transfer distance is 70 m (230 ft.) for this case. Figure 5 shows how the load transfer distance variations change the pillar loading at the various abutment locations. The results show, for example, that if the load transfer distance was 283 m (930 ft.) the bleeder loading could be reduced as much as 82% of the loading based on the Peng-Chiang

load transfer distance. As observed in the sensitivity study, the ALPS method assumes the tailgate loading is unaffected by the load transfer distance.

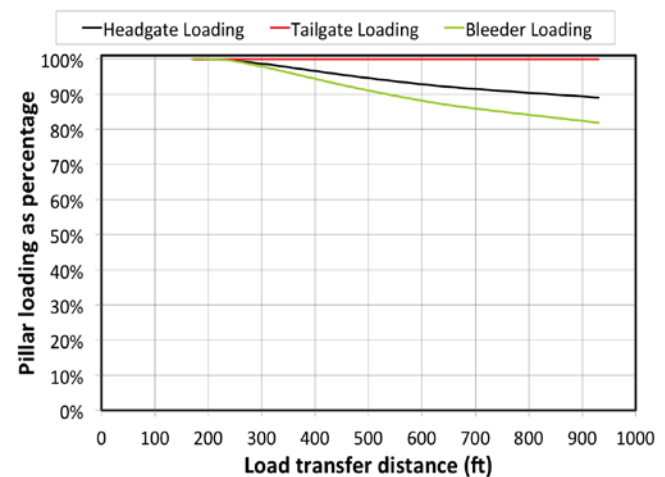


Figure 5. Impact of load transfer distance on pillar loading for the Case 1 shallow longwall using the ALPS methodology. The pillar loads are expressed as a percentage of the loading associated with the Peng-Chiang load transfer distance of 230 ft.

Case 2: Deep Longwall

The load transfer distance was again varied between 52 m (170 ft.) and 283 m (930 ft.) and the changes in the pillar loading determined. Figure 6 shows the impact of the changes on the pillar loading at the different pillar locations.

The maximum load transfer distance of 283 m (930 ft.) considered here is conservative relative to the 448 m (1,471 ft.) reported by Larson and Whyatt (2012). Nevertheless, bleeder loading at gateroad 1 is shown to reduce as much as 76% of the loading that would be predicted if the Peng-Chiang load transfer distance was used. The tailgate loading is again seen to be independent of the load transfer distance.

DEMONSTRATION OF POTENTIAL IMPACT OF ABUTMENT ANGLE VARIATIONS

As Tulu and Heasley (2012) report, variations in abutment angle (β) may exist for deep mines. The impact of abutment angle deviations on pillar loading is demonstrated in this section.

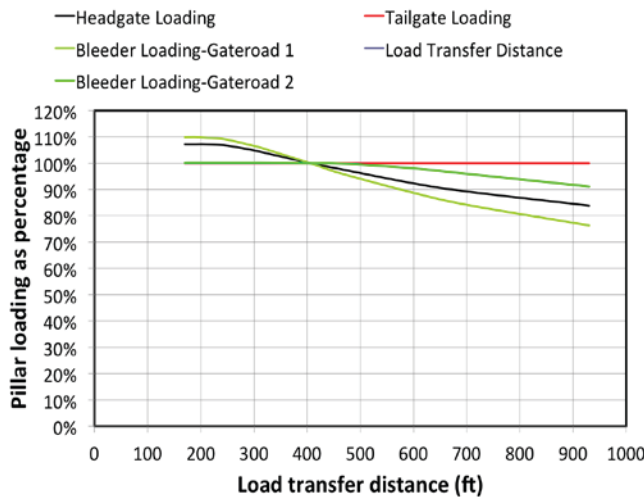


Figure 6. Normalized LaModel results for RT with subcritical (green, triangle), critical (red, circle) and supercritical (blue, square) cases. The circled cases at a normalized load transfer distance of one represent the ALPS default load transfer distance. This graph illustrates the modeled dependence of RT, similar to the abutment fraction (R) used by traditional ALPS calculations, on load transfer distance, D.

Case 1: Shallow Longwall

The variability in abutment angle described by Tulu and Heasley (2012) does not apply to the hypothetical case 1, which is representative of a shallow mining scenario. Equation 3 is derived from observations in mines whose mining depths exceed 274 m (900 ft.). At depths of less than 274 m (900 ft.), the default angle of 21° is recommended. As such, no change from default loading estimations are examined.

Case 2: Deep Longwall

Abutment angles in the range of 6° to 26° were evaluated, spanning the range of potential abutment angles reported by Tulu and Heasley (2012) and Mark (1987). For these analyses, the abutment extent was held constant at the Peng-Chiang calculated value of 405 ft. The results (see Figure 7) show that as the abutment angle reduces to 6° tailgate and bleeder loading can be as little as 63% of the loading that would be predicted when using an abutment angle of 21°.

SUPPLEMENTAL LAMODEL STUDY TO INVESTIGATE TAILGATE PILLAR LOADING

The ALPS sensitivity study was augmented by a LaModel study to further investigate how load transfer distance may affect loading of tailgate pillars. Although ALPS assumes that there is no effect, similar effects on other pillar locations suggest that this is an oversimplification, perhaps required by the paucity of field measurements available during development of the ALPS program.

We return to the Case 1 shallow longwall. For this study, only the depth of mining was varied from 152 m (500 ft.) to 762 m (2,500 ft.) to produce supercritical, critical and subcritical caving cases. LaModel input parameters were developed following the

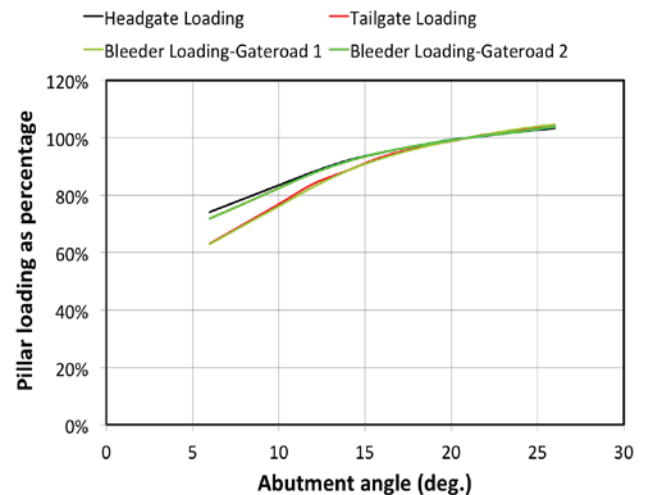


Figure 7. The impact of variation of abutment angle on calculated pillar loading for two sets of gateroads in Case 2: Deep Longwall using the ALPS methodology. Pillar loads are expressed as a percentage of the loading associated with a 21° abutment angle.

guidelines of Heasley et al. (2010) that produced LaModel results consistent with the empirical equations underlying ARMPS and ALPS. These include matching the load transfer distance equation of Peng and Chiang (1984), and an abutment (β) angle of 21° (Mark, 1990). Elastic pillars were used in all cases. For a more detailed discussion of modeling procedures, please refer to Appendix A.

Modeled pillar loads were compiled for a set of gateroad pillars that were the width of the gateroad plus one crosscut in length. The critical tailgate (T-Junction) pillars were assumed to have their inby crosscut ribs parallel with the longwall face, as was used by Colwell, Frith, and Mark (1999a; 1999b). One difference in pillar load determination from model results, compared to the original field cases for ALPS, is that the loading on these model pillars was summed over the entire set of two pillars rather than looking at the load over a single line across the middle of the pillars, as was done by instrument measurements used during the development of ALPS.

According to the LaModel results, the tailgate loading is indeed affected by load transfer distance. The LaModel results were used to determine the relationship between the load transfer distance and the tailgate loading, as a surrogate tailgate load factor (F_T). The LaModel based load factor was subsequently applied to re-calculate tailgate loading for the hypothetical Cases 1 and 2 for a range of load transfer distances. Details of the development of the LaModel based load factor are presented in Appendix A. Figure 8 shows how the tailgate loading would be affected by changes in the load transfer distance based on the LaModel derived load factors.

The LaModel results show that tailgate pillar loading is sensitive to load transfer distance consistent with other pillar locations. This consistency could be important for integrating combinations of cases, i.e., from the first gateroad in a district (a bleeder or headgate loading geometry) and subsequent gateroads (a tailgate loading geometry), where load transfer distance departs markedly from the Peng and Chiang estimate.

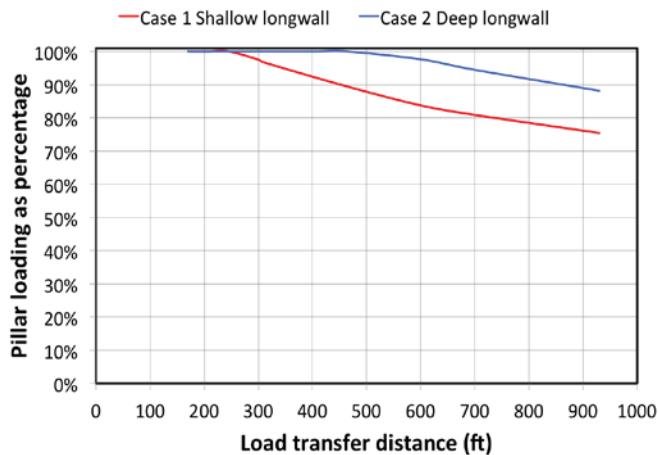


Figure 8 . Impact of load transfer distance on tailgate pillar loading using LaModel results. The results for the Case 1 shallow and the Case 2 deep longwall are shown. The pillar loads are expressed as a percentage of the loading associated with the Peng-Chiang load transfer distance of 70 m (230 ft.) and 123 m (405 ft.) for Case 1 and Case 2 respectively.

A WORD OF CAUTION

The study presented here has the sole objective to evaluate potential impact of the load transfer distance and abutment angle on pillar loading. Such arbitrary modification of parameters naturally invalidates the link between ALPS and the recommended stability factors derived through back analysis of actual longwall performance. When using ALPS for design, it is imperative to calculate the load transfer distance using the default Peng-Chiang equation and strongly recommended to use the default value of 21° for the abutment angle. Substitution of other values, even in cases where those values are explicitly known for that location, is not recommended outside of the context of an extensive local database.

CONCLUSIONS

Recent research (Tulu and Heasley, 2012, and Larson and Whyatt, 2012) suggests that increased load transfer distances and reduced abutment angles may exist, particularly in deep western longwall mines. This study explores how variations in the abutment angle and the load transfer distance might impact the calculated pillar loads in longwall coal mines, with the intention to guide future research and provide mine operators with guidance on which ground characteristics are most apt to affect pillar loading, and are therefore the most valuable to measure.

Results indicate that the abutment angle, β , is particularly important for panels with supercritical geometries. As panels reach a subcritical geometry with increasing depth, the primary influence of β is reduced and the relative influence of other factors, including load transfer distance and the headgate and tailgate abutment factors, increases. When this subcritical geometry is present, observations of long load transfer distances begin to have a significant influence on the pillar loading.

Application of load transfer distances and abutment angles consistent with those observed by Larson and Whyatt (2012) and Tulu and Heasley (2012) to hypothetical cases indicates that pillar

loading may be significantly reduced. A LaModel study shows, in addition, that loading of pillars at the tailgate junction may be favorably impacted by an increase in the load transfer distance. These findings need to be verified by actual observations in operating coal mines.

The observation of reduced abutment angles at increasing depth seems to indicate that the overburden in these cases was relatively weak, limiting the ability for strata to span over large distances and create extended load transfer distances. It appears, therefore, that reduced abutment angles and extended load transfer distances are unlikely to occur simultaneously at a particular longwall location.

This study highlights the fact that further work is needed to examine abutment angle and load transfer distance values, and the nature of loading at depth. This is particularly relevant to western coalfields, where there is a clear need for a better understanding of loading characteristics. Integration of improved estimates of ground response characteristics into design decisions can only increase quality, confidence and safety.

As these finding may have implications for recommended ALPS stability factors, great care must be taken that undue focus is not placed on the loading component of pillar stability. Pillar loading must be considered as only one part of the stability equation, and should not be used in the context of the national database; erroneous and dangerous stability factors may result. A holistic approach toward pillar integrity must be taken, which would take into account both variations in loading and capacity. Toward this end, further investigation into the effects of installed support and local geology is indicated.

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APPENDIX A—LAMODEL STUDY SPECIFICS

This modeling exercise was designed to study how overburden stiffness might impact load transfer distance and pillar loading in a longwall coal panel. It was designed in line with a LaModel analysis of Analysis of Retreat Mining Pillar Stability (ARMPS) cases published by Tulu et al. (2010).

This modeling exercise assumed elastic coal and nonlinear gob, in part to minimize the influence of pillar yielding on load estimates. We calibrated rock mass stiffness so that the distance from the gob within which 90% of the abutment load occurs in the model matches the distance within which 90% of the abutment load occurs according to the ALPS stress distribution profile for a particular load transfer distance, D. For this study, D was not always the same load transfer distance calculated with the empirical formula of Peng and Chiang (1984). We calibrated final gob modulus to reflect abutment loading corresponding to the angle, β , of 21° (Mark, 1990). These calibrations were done using the lamination thickness wizard and the gob wizard, respectively, in LamPre 3.0 (Heasley, 2010). To match the load transfer distances using the lamination thickness wizard, we had to take advantage of the concept of an effective seam depth, as did Larson and Whyatt (2012). The “zero” position of the face with respect to the pillar in the tailgate in these models was the same as that used by Colwell, Frith, and Mark (1999a; 1999b)—that is, the inby edge of the pillars being evaluated were in line with the longwall face.

Example 6.2.2 from Mark’s (1987) dissertation was used to examine the variation of tailgate loading with load transfer distance. Table A-1 lists parameters from Mark’s example problem and those used in the models with LaModel. It was determined to use 2.74-m (9-ft) elements in LaModel, which prevented an exact match in geometry.

Table A-1. Parameters used in example and model.

Parameter	Mark example 6.2.2	LaModel
Depth of cover	304.8 m (1000 ft)	153 m (500 ft), 318.0 m (1043.3 ft), 762.0 m (2500 ft)
Panel width	243.8 m (800 ft)	244.1 m (801 ft)
Critical nature	Supercritical	Supercritical, Critical, Subcritical
Number of entries	3	3
Entry width	5.49 m (18 ft)	5.40 m (18 ft)
Pillar width	21.9 m (72 ft)	21.9 m (72 ft)
Pillar length	28.0 m (92 ft)	27.4 m (90 ft)
Weight density of overburden	25.5 kN/m ³ (162 lb/ft ³)	25.5 kN/m ³ (162 lb/ft ³)
Seam height	1.83 m (6 ft)	1.83 m (6 ft)
Seam elastic modulus	NU	2.068 GPa (300,000 psi)
Seam Poisson’s ratio	NU	0.33
Overburden elastic modulus	NU	13.79 GPa (2,000,000 psi)
Overburden Poisson’s ratio	NU	0.25
β	21 °	21 °
Initial gob modulus	NU	0.689 MPa (100 psi)
Maximum gob stress	NU	27.58 MPa (4000 psi)
Gob height factor	NU	1.
NU = Not used.		

The effect of load transfer distance on the tailgate of a panel was examined by constructing a 510 by 360 grid of the seam in LaModel. Model geometry included two panels, so that the tailgate pillars measured were between a mined-out panel and a panel being mined. Tailgate pillars were located at the center of the model. Average loads were estimated for both pillars together, when the face was in the location with respect to the pillars as illustrated by Colwell, Frith, and Mark (1999a; 1999b). Three depths of 152.4 m (500 ft), 318 m (1043.3 ft), and 762.0 m (2500 ft) were chosen to represent supercritical, critical, and subcritical cases, respectively. The load transfer distances used for each depth were chosen to represent a wide range, as shown in Table A-2.

The load on these tailgate figures is the result of the following (Mark, 1990):

Table A-2. Load transfer distances used in the model using LaModel.

Depth of seam, m (ft)	Load transfer distances used
153.0 (500)	0.817D, 0.866D, 0.914D, 1.00D, 1.491D, 2.000D
318.0 (1043.3)	0.566D, 0.599D, 0.692D, 1.00D, 2.000D
762.0 (2500)	0.529D, 0.667D, 1.00D, 1.505D, 2.000D
D refers to the load transfer distance calculated by $D=9.3\sqrt{H}$, where H is the overburden thickness.	

$$L_T = L_D + L_s(1 + F_T) \quad (A-1)$$

where L_T = total load on the tailgate pillars in the “zero” position,

L_D = load on the same pillars from development,

L_s = abutment load, as defined in figure 1 (or L_{ss} if subcritical), and

$(1+F_T)$ = the multiplier of L_s (or L_{ss} if subcritical) representing the portion of the abutment load that is transferred to the pillars in the zero position.

Therefore, the tailgate loading factor, $(1+F_T)$, can be solved by rearranging equation (A-1):

$$(1 + F_T) = \frac{(L_T - L_D)}{L_s} \quad (A-2)$$

where L_T is determined from the model, L_D is calculated by tributary weight as did Mark (1990), and L_s (or L_{ss} if subcritical) is determined from Figure 1, as did Mark (1990).

Figure A-1 shows the tailgate loading factor, $1+F_T$, determined by using the models. By comparison, the ALPS tailgate loading factor ($1+F_T$) is 1.7, also shown in the figure. The difference between the model $1+F_T$ and the ALPS amount (1.7) is significant, and shows that more of the abutment load is transferred to the unmined portion of panel 2 (and possibly the gob) than was envisioned in the ALPS program. However, Mark (1990) states that the factor 1.7 was based on a set of measurements at one site, without describing how or where those measurements were made or how those measurements were used to estimate the pillar load.

Figure A-2 shows a version of Figure A-1, but the tailgate factor has been normalized so that the tailgate factor for the cases having a load transfer distance matching the Peng and Chiang equation (equation 1) are made equivalent to the ALPS tailgate factor. Herein, the load transfer distance calculated by equation 1 will be referred to as the ALPS load transfer distance, or ALPS LTD. To be more clear, a normalization factor N_D was determined from

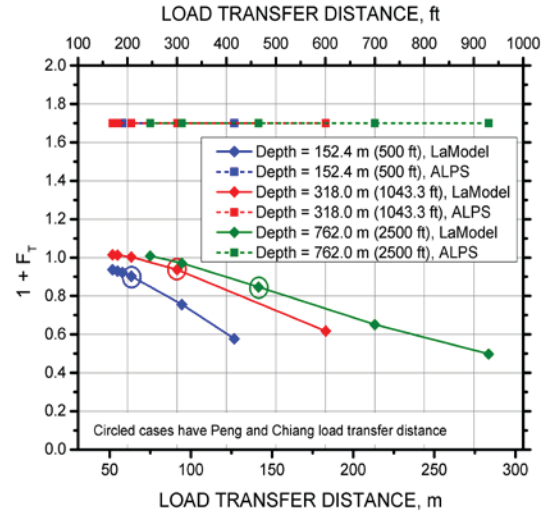


Figure A-1. Tailgate loading Factor, $1 + F_T$, vs. load transfer distance, as determined using LaModel for example 6.2.2.

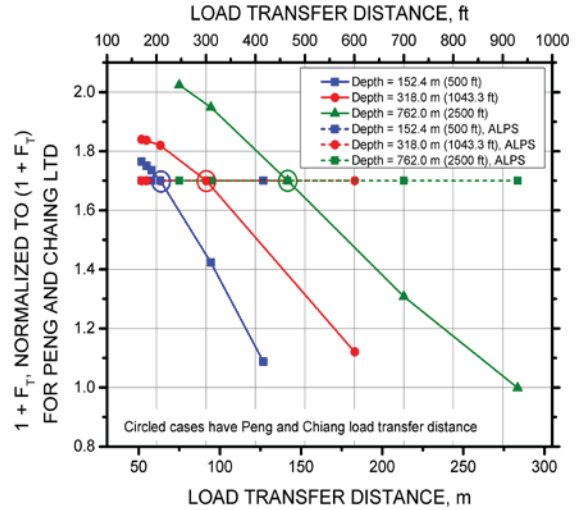


Figure A-2. Tailgate loading factor, F_T , normalized to ALPS loading factor ($1 + 0.7$).

$$\frac{1 + F_T}{N_D} = 1.7 \quad (A-3)$$

at the ALPS load transfer distance for each seam depth. Then $1+F_T$ was divided by the N_D determined for that seam depth. The figure clearly shows that the tailgate factor varies with load transfer distance.

Since our interest is only in how results vary with changes in load transfer distance, the factor $1+F_T$ was normalized with respect to its amount from the model having the ALPS load transfer distance. That is,

$$R_T = \frac{(1 + F_T)}{(1 + F_T)_{(ALPS LTD)}} \quad (A-4)$$

Figure A-3 shows this factor plotted with load transfer distance normalized with the ALPS load transfer distance. The plot provides an estimate of the relative change in proportion of L_s exerted on the tailgate pillars with respect to $1+F_T$, determined from the model having the ALPS load transfer distance. Thus, if the load transfer distance is 40% greater than assumed in ALPS (a factor of 1.4 on the x-axis), the LaModel predicted contribution of the side abutment L_s to tailgate pillar loading is about 95% of that predicted by the model having the ALPS load transfer distance. Mathematically, this relationship can be expressed as

$$L_T = L_D + L_s (1 + F_T)_{(ALPS LTD)} R_T \quad (A-5)$$

Again, the plot indicates that tailgate pillar loading varies with load transfer distance.

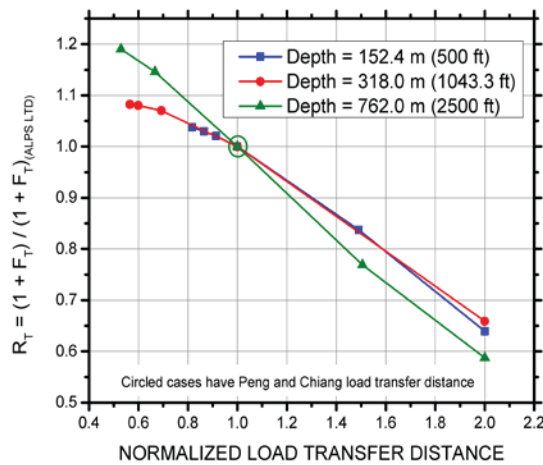


Figure A-3. Normalized LaModel results for RT with subcritical (green, triangle), critical (red, circle) and supercritical (blue, square) cases. The circled cases at a normalized load transfer distance of one represent the ALPS default load transfer distance. This graph illustrates the modeled dependence of RT, similar to the abutment fraction (R) used by traditional ALPS calculations, on load transfer distance, D.

International Practice in High Performance Underground Thick Coal Seam Extraction and Related Ground Control Challenges

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ABSTRACT

This paper provides an overview of the international “state of the art” with respect to underground coal mining methods used in seams of thickness greater than 4.5 m, through a review of a selection of high performing thick seam mining operations around the world—both Single Pass Longwall (SPL) and Longwall Top Coal Caving (LTCC). The paper discusses the recent operational experiences from these selected faces, together with the ground control issues faced, and identifies some of the major, inherent geotechnical factors that can influence the mining method selection, design criteria, and ultimate performance, from both an operational and safety perspective.

It is clear that both of these methods are developing quickly—not only from an operational and equipment perspective, but also from the understanding of critical geotechnical criteria. There is an increasing role for both methods in thick seam environments, although the higher SPL faces will only be successful in the near to mid-term under benign geotechnical environments. Regardless of the method, a thorough understanding of the critical geotechnical factors is a key to successful implementation of either method.

INTRODUCTION

Thick Seam Mining Methods

Underground mining of coal seams with thicknesses in excess of 4.5 m can present a number of opportunities along with significant ground control challenges. Many mining methods have been adopted in different parts of the world but often with only limited success. In the past ten years, conventional longwall faces have struggled to achieve optimum productivity as well as resource recovery when cutting at heights at or above 5 m, and in some cases even less. Ground control problems associated with face stability are one of the major challenges faced by these high reach single pass longwalls (SPL). However, in the past two years, panels are being mined successfully with face heights up to 7 m in parts of China.

The other major mining system that provides a viable, high performance alternative to SPL is Longwall Top Coal Caving (LTCC). LTCC avoids the problem of high faces, but relies on

good, predictable, and immediate caving of top coal behind the face in order to achieve optimum resource recovery while maintaining good productivity. In both mining systems—SPL and LTCC—apart from underground geotechnical challenges, surface subsidence is also a major geotechnical consideration that must be managed.

As a point of definition, the term thick seam has commonly been applied to any minable seam thickness greater than the reach of existing development and longwall systems. In the 1980s and 1990s, this was interpreted as 4.0 m. However, with higher reach continuous miners and longwall systems, an arbitrary figure of 4.5 m has been adopted in Australia for all recent studies into “thick seam mining.” Note: The Chinese mining industry uses a figure of greater than 5 m to define thick seam operations (Wang, 2011).

Australian Thick Seam Mining Background

There has been considerable interest in underground thick seam mining methods in Australia for decades. Thick seam bord and pillar mining has taken place in Australian coalfields for over fifty years in locations such as the Cessnock and Hunter Coalfields near Newcastle in New South Wales (NSW); the Ipswich Coalfield west of Brisbane in Queensland; and in Moura and Collinsville, in the Bowen Basin in Queensland. These early operations consisted of various forms of bord and pillar mining, commonly involving multiple slice ramping operations during panel retreat, to produce high pillar workings often characterized by problems such as pillar stability, rib stability and spontaneous combustion, together with limited resource recovery performance.

Hebblewhite et al. (2002) reported that Australia has significant reserves of thick coal seams that require the application of alternative mining methods beyond the conventional bord and pillar or longwall systems. Table 1 summarizes the available data regarding potential underground “thick seam coal” in Australia.

The international incentive for identifying or developing new methods for underground thick seam mining is commonly for optimizing resource recovery. However, the Australian coal industry is an export-dominated industry where high productivity, sustainable financial viability, and the highest safety standards are paramount. As such, the Australian requirement is for appropriate methods that meet or improve upon the Australian safety and

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Table 1. Australian thick seam reserves (after Hebblewhite et al., 2002).

AUSTRALIA		Number of Sites	Measured Resources (Mtonnes)	Measured Resources (%)	Ind. & Measured Resources (Mtonnes)	Ind. & Measured Resources (%)
Seam Thickness	4.5m – 6.0m	29	2,249	34.8	7,425	42.2
	6.0m – 9.0m	33	3,310	51.2	8,397	47.8
	>9.0m	9	597	9.2	1,113	6.3
	No Info	2	310	4.8	650	3.7
	TOTAL	73	6,466	100.0	17,585	100.0
Seam Dip	<5 deg	43	3,173	49.1	11,684	66.4
	5 deg – 15 deg	21	2,256	34.9	3,698	21.0
	>15 deg	9	1,037	16.0	2,203	12.5
	TOTAL	73	6,466	100.0	17,585	100.0
Seam Depth	<150m	28	3,312	51.2	6,303	35.8
	150m – 300m	23	1,594	24.7	5,383	30.6
	>300m	19	1,508	23.3	5,698	32.4
	No Info	3	52	0.8	201	1.1
	TOTAL	73	6,466	100.0	17,585	100.0

productivity/financial performance criteria, while at the same time achieving maximized resource recovery, as a second tier priority. On the other hand, in China, resource recovery has been the number one priority, by government decree, probably ahead of productivity performance, resulting in many of the mining system and equipment advances that have occurred in the last twenty years in that country.

Earlier studies by the University of New South Wales (UNSW) and others (Hebblewhite, 1999; Hebblewhite, 2001; Hebblewhite et al., 2002) reviewed the Australian opportunities and available methods and technologies for underground thick seam methods. From this review, four generic methods were identified as having thick seam potential:

- Extended height single pass longwall (SPL)
- Multi-slice longwall (MSL)
- Hydraulic mining (HM)
- Caving longwall systems (CL), including longwall top coal caving (LTCC).

The option of extending the height of a conventional single pass longwall was considered at the time to have limited possibilities. It was apparent that technology was already increasing both shearer and support heights from 4 m to 5 m and above (Hamilton, 1999). However, limitations such as equipment size, weight, and stability, along with face conditions, were considered at the time to limit the application of this method to no more than 6-m heights for many years to come. The method was certainly considered to have great potential in the 4.5–6.0 m mining height range.

Multi-slice longwall was also reviewed in detail, with consideration of experiences from both Europe and China. The potential to apply modern paste fill technologies for septum formation and stabilization was investigated (Palarski, 1999; Bassier and Mez, 1999). However, through a risk assessment process, other issues such as mining under goaf (gob) areas (water

and gas hazards) and general stability concerns, quite apart from the very limited gains in productivity anticipated, eliminated this method as a viable future option for Australia.

Hydraulic mining, as practiced in New Zealand and elsewhere previously, was also investigated. It was found to be a method with a significant potential in a limited range of suitable mining conditions. It offered significant financial benefits, but limited large-scale production potential. It was therefore considered to be suitable as a “niche application” method but not a universally applicable option.

This left the range of longwall caving options. Evaluation of different European and early Chinese experiences with the original “soutirage” mining concepts and equipment showed promise, but performances were below the level required to be viable in Australia, not to mention concerns over issues such as dust and spontaneous combustion. However in the 1990s, the Chinese coal mining industry developed the modern double armoured face conveyor (AFC) LTCC method, which largely replaced previous multi-slice longwalls and earlier caving systems. This method provided a significant alternative to SPL, not only for the seam thicknesses between 4.5 m and 6 m, but particularly for the thicker 6–9 m thicknesses, which constitute close to 50% of Australia’s reported underground thick seam resources (see Table 1). Xu (2001) provided an early insight into a modern LTCC face at the Chinese Dongtan Mine, owned by the Yankuang Group. Hebblewhite (2001) provided an assessment of the geotechnical challenges faced by the different thick seam mining options at that time. Kelly et al. (2003) reported to the Australian coal industry on the opportunities and issues for introducing LTCC into Australia. Hebblewhite (2005) provided a broad review of all of the above mining system options, with a focus on the LTCC potential and related challenges for application by the Australian mining industry.

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CURRENT AUSTRALIAN THICK SEAM OPERATIONS

This paper is primarily focused on international thick seam mining operations beyond Australia. However it is also important to recognize the current status of thick seam mining practice within Australia, which is summarized and briefly discussed below. Outside of China, it is believed that Australia has the largest number of high performance thick seam operations, consistent with the large extent of thick seam reserves in the country, as summarized by Table 1. Clearly there is a strong incentive for further development of thick seam mining technologies in Australia, for the reasons outlined previously.

Based on 2011 reported statistics (Australian Longwall Magazine, 2012), using the definition of a ≥ 4.5 -m mining height, Table 2 lists Australia's SPL thick seam operations (all dimensions refer to the maximum within the quoted ranges). In Table 2, Mining Height refers to the cutting height of the longwall face, whereas Shield Height refers to the maximum potential working range of the face shields. It is significant to note that even where seam thickness might permit, cutting height is always less than the maximum shield range. This is a prudent precaution against roof falls on the face, to enable the shields to operate at higher than the cut height in order to maintain roof contact at all times, even with the presence of roof cavities. Figure 1 is a photo of the longwall face at West Wallsend Colliery when it was cutting up to 4.8 m in height. It highlights one of the key geotechnical challenges for SPL faces, face spalling, which has been experienced regularly at West Wallsend.

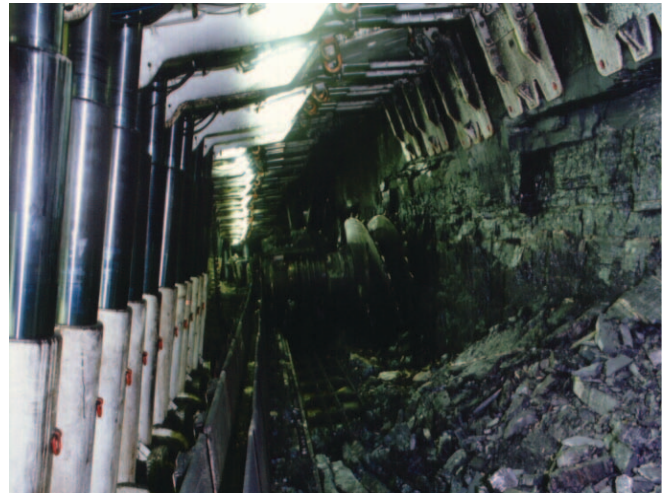


Figure 1. Longwall face at West Wallsend Colliery when it was cutting up to 4.8 m in height.

Austar mines the 6.5-m-thick Greta Seam at depths of 500–600 m, with a cutting height of 2.9 m. Figure 2 is a schematic diagram of the LTCC concept, showing the location of the rear AFC and the caving of the top coal onto the rear AFC — a process that is managed by only caving a small bank of shields at any one time, usually immediately after the shearer has passed.

Table 2. Australian thick seam longwall mines using SPL mining systems (2011/2012).

Mine	Depth(m)	Seam Thickness(m)	Mining Height(m)	Shield Height(m)	Shield Capacity(tonnes)
Broadmeadow	300	4.8	4.8	5.2	1,152
Carborough Downs	450	4.5	4.5	5.3	1,238
Mandalong	320	5.0	4.8	5.2	1,053
Moranbah North	320	4.5	4.5	5.0	1,750
Newlands Northern	240	5.0	4.5	5.0	1,040
North Goonyella	350	4.5	4.5	5.3	1,200
West Wallsend	205	4.6	4.6	5.3	900

The above is a summary of the SPL faces operating in Australia at the start of 2012. However, there is also one LTCC face that has been in operation for a number of years. Austar Colliery operates the first LTCC face in Australia. Austar is owned by the Chinese company, Yancoal Australia Ltd, and the Chinese parent company is Yanzhou, part of the Yankuang Group, which is also one of the leading LTCC performers in China. UNSW and the Commonwealth Scientific and Industrial Research Organisation (CSIRO) had previously been engaged by Yankuang to evaluate potential sites in Australia for the introduction of LTCC, and Austar is the result. Austar was the first mine in the world to introduce two-leg LTCC shields, through a collaborative development project between Yankuang and Deutsche Bergbau-Technik (DBT). Previously, all Chinese LTCC faces used 4-leg shields. This equipment has subsequently been introduced into some Chinese LTCC faces. Moodie and Anderson (2011) have provided a current assessment of the successful performance of LTCC at Austar.

It should also be noted that two further LTCC faces have been, or are in the process of being, introduced in Australia. These are at the Broadmeadow Mine, operated by BMA and also the North Goonyella Mine, owned by Peabody. In both cases, the new LTCC faces are replacing previous SPL faces.

REVIEW OF INTERNATIONAL OPERATIONS

During 2011, the author undertook a period of special study leave (sabbatical) from UNSW to conduct a review of a selection of international thick seam operations (both SPL and LTCC). The mines visited are listed in Table 3 below. All data discussed in the remainder of this paper was provided informally through personal communications with various management and operational personnel at each mine.

The following is a brief summary of key operational and geotechnical findings from the above visit program, based on

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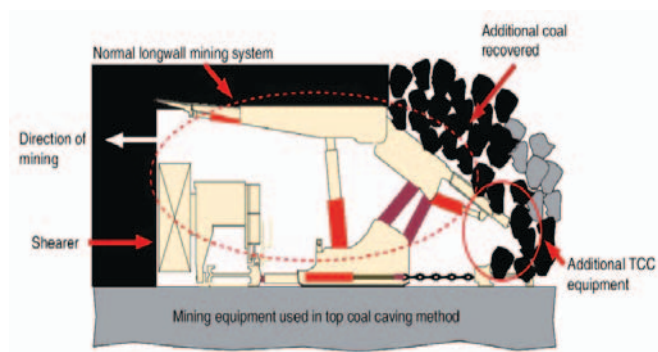


Figure 2. Schematic diagram of the LTCC concept, showing the location of the rear AFC and the caving of the top coal onto the rear AFC.

information provided by various sources during these visits. It is stressed that the data quoted here was valid data at the time of the mine visits.

contractual arrangements, rather than any operational reasons or mining conditions.

Geological conditions are a significant factor in the operational performance at Matla. The interburden between the upper 4 Seam and the 2 Seam mined in No. 2 Mine is only 19 m thick. Chain pillars in the 2 Seam are positioned directly beneath those in the 4 Seam above, a design which is considered to have improved some of the problems associated with high stress conditions in the lower workings. Other geological issues include extensive dolerite sills in some areas. The combination of the sills and some strong sandstone units has led to problems establishing the initial cave in each longwall panel. When caving does occur, this can lead to windblasts; there was a windblast-related fatality at the No. 3 Mine in 2011. To overcome the initial caving problems, drilling and blasting the immediate overburden is almost standard practice at the No. 2 Mine. As a result of the relatively shallow depth, surface subsidence is a major issue requiring a considerable amount of surface remediation and premining preparation.

Table 3. International thick seam mines visited by Hebblewhite (2011).

Mine	Owner	Country	Depth	Mining Method	Mining Height	Face Length
Matla No. 2	Exxaro	South Africa	110m	SPL	4.5 – 5.0m	126m
Daw Mill	UK Coal	UK	700m	SPL	5.2m	357m
Jining No. 3	Yanzhou	China	700m	LTCC	Cut 3m in 7m seam	290m
Dongtan	Yanzhou	China	710m	LTCC	Cut 3-3.5m in 6m seam	223m
Gaohe	Shanxi Gaohe Energy	China	470m	LTCC	Cut 2.5-2.8m in 6.5m seam	236m
Danang	Danang Energy	China	130m	SPL	4.5 – 5.0m	202m
Tashan	Datang Tashan Coal Mine Co.	China	400m	LTCC	Cut 3.5m in 12-20m thick seam	235m
Bulianta	Shenhua	China	300m	SPL	6.8 - 7.0m	286m

Matla

Matla No. 2 Mine mines the No. 2 Seam, with previous workings present in the 4 Seam above. This is part of a three-mine complex that makes up the Matla Mine, and in total, is producing approximately 12.8MTPA. No. 2 Mine was achieving up to 538,000 tonnes (t)/month (approx. 20,000 tt/day), and a long-term average of 200,000 t/month (including face moves). No. 2 Mine has supports with a shield maximum operating height of 5.5 m, but is typically cutting only 5 m or less. Face height is reduced to about 4.3 m for face recovery. Supports are two-leg shields with a maximum capacity of 1,066 tonnes. The shields are only 1.75 m wide, and the engineering/management personnel at the mine believe that 5 m is an upper limit due to lack of rigidity in the supports, leading to twisting of the lemniscates in each shield resulting in face management problems and potential equipment damage. Matla staff believe wider shields are necessary to operate at 5 m or greater.

The face lengths at Matla in all mines are of the order of 130 m due to limited capital availability under their domestic coal

In summary, the biggest challenges facing thick seam mining at Matla are the geological conditions (including relatively incompetent immediate roof and floor); the operational problems due to the low width:height ratio of the shields; and water management in a multi-seam mining operation, working beneath previous goaf areas.

Daw Mill

Daw Mill is the largest operating underground coal mine in the United Kingdom, located in proximity to Coventry. It is mining the Warwickshire Thick Seam, which is up to 7 m thick in places, due to the coalescence of up to 4 different seams. Daw Mill aims to produce approximately 3MTPA, with a peak of 3.2MTPA achieved in 2008. Average daily production is budgeted at 15,000 t/day from the longwall face. The face supports are two-leg shields with a capacity of 850 tonnes. The face cutting height is between 5.0 m and 5.2 m, although the supports have a maximum height range of 5.4 m. At least 0.6 m of coal is left in the roof to maintain roof stability. Supports are 1.75 m wide, but the management does not report any problems with shield stability at these heights. The seam is relatively flat overall, although localized seam gradients

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are experienced as steep as 1 in 5. Faults are also a problem, and the face visited (32 Panel) was about to negotiate an 11-m throw fault at the time of the visit. Face conditions were generally good, although some degree of face spalling was commonplace. The Daw Mill face height is lowered to 4 m for face recovery.

The biggest challenges for Daw Mill are not the longwall face itself, but the extremely weak floor conditions, which cause major problems, especially in the tailgate. Extensive tailgate support is required, together with frequent floor brushing. The other major challenge for Daw Mill is spontaneous combustion.

Jining No. 3 & Dongtan

These two mines were part of the Yanzhou operations in the east of China (Shandong Province). Both employ LTCC for their thick seam operations, although conventional height longwalls were also in use in other lower height seams.

Jining was producing 7MTPA at the time, with approximately 85% of this production from the LTCC face. The LTCC face was achieving up to 15,000 t/day production, and an average volumetric coal recovery of 70% of the 7-m seam being mined. The mine adopted an unusual mid-face gate road, to bring services onto the face—reportedly due to problems of water in the maingate. In spite of this complication, the mine was achieving a face retreat rate of approximately 6 m/day.

Dongtan Mine has been using LTCC since 1992, having previously used multi-slice longwall. In 2006, following the development of the DBT 2-leg LTCC shield, a new 2-leg, 850-tonne capacity shield face was introduced to Dongtan with a production capacity of 6MTPA. This yields 15,000–16,000 t/day. The cutting height at Dongtan is dictated by a mid-seam stone band at approximately 3.5 m above the floor.

Conditions on both the Jining and Dongtan faces were very good, with no evidence of any significant face or roof stability problems, and good coal caving (and good fragmentation) immediately behind the face onto the rear AFC. Dongtan only caves one shield at a time, immediately behind the shearer, and claims a volumetric coal recovery of up to 86%.

Some discussion with engineers from Yanzhou revealed that all mines in the group are now using LTCC rather than the previous multi-slice longwall system. This has resulted in a significant improvement in mining conditions and stability, and also marked improvement in production (daily average face production up from 10,000 t/day to 15,000 t/day), and significant reduction in development requirements. Depending on the caving:cutting ratio for any given seam, they are also considering higher capacity rear AFCs to handle increased caving ratios. (They believe that the maximum caving:cutting ratio is 3:1 for current LTCC technology).

On the subject of choosing SPL instead of LTCC, their view was that it was very much related to geological conditions (especially coal seam conditions). In their eastern Chinese conditions, SPL was used for any seams up to 5 m, but LTCC was used for any seams above 5 m where face and possible coal roof stability became a problem with SPL at such heights due to coal seam structure and strength. By comparison with the new SPL high faces in western China (see Bulianta discussion below), they believe that the

geotechnical differences that allow for high SPL faces are stronger coal, shallower depth, and lower horizontal stresses allowing for improved roof conditions above and ahead of the shields. Yanzhou engineers believe that the development of a relationship between depth/coal Uniaxial Compressive Strength (UCS) and LTCC mining conditions would be invaluable.

Gaohe & Daning

The Gaohe Mine is an interesting operation located near Changzhi, in Shanxi Province, south of Taiyuan city. The mine is owned by a joint venture: 55% to the local Lu'an Coal Bureau and 45% to Asian-American Coal, which is now wholly owned by Banpu from Thailand. Daning Mine was also previously managed under a similar arrangement.

Gaohe had initially been designed in the early years of the 21st century with a unique shield concept that would allow either SPL or LTCC mining options—in other words, a 6-m height range shield that also incorporated an LTCC rear configuration. While initial design work was done for such a shield, it was never completed, and the mine was subsequently developed as an LTCC mine. This was particularly due to concerns about lower than expected coal strength and the prevalence of structure within the coal. Gaohe engineers refer to a Chinese empirical coal cavability index referred to as an F Factor (no details available). According to them, Gaohe coal has an F Factor of 0.2, compared to 0.6 for other mines such as Daning, but a value of 6.0 for Bulianta—a very significant difference.

Gaohe was still in the commissioning stage of operations and was only achieving 7,000–8,000 t/day, or 3MTPA, compared to a production capacity of 6MTPA. Shields were 800-tonne capacity, cutting at heights between 2.5 m and 2.8 m. The face conditions included a mid-seam stone band plus regular cleating and jointing and a number of low angled faults (all of which mitigated against SPL as an option). Other problems for Gaohe included high gas levels (18 m³/tonne CH₄) and higher than normal subsidence due to the top 200 m of overburden being unconsolidated soils and clays.

Daning on the other hand is a generally more competent coal, and at a shallower depth (130 m), can stand up as a 4.5–5 m face height, although face and roof coal instability is still a nuisance-level problem. The two-leg, 790-tonne supports have a maximum height range of 5.5 m to allow for handling roof cavities in the soft coal tops. Use of face flippers is critical at all times to control face flushing behind the shearer. The flippers are a two-stage system providing face coverage of 1.8 m below the roof horizon. Daning at the time was producing 4MTPA, averaging 180,000 t/month from the one face.

Tashan

The Tashan Mine, located near Datong, in Shaanxi Province, was the highest performing LTCC mine in China (and hence the world) in 2011. The mine was operating two LTCC faces and one conventional height longwall, with a total annual mine production of 23MTPA; 10MTPA of that was coming from each of the LTCC faces. These were averaging 35,000 t/day, with a maximum of 50,000 t/day achieved. Seam thickness varied from 12 m to 20 m, and the mine claimed to be recovering between 70% and 80% of the seam in each panel, by volume, with a cutting height of only

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3.5 m and a web thickness of 800 mm. The overall face retreat rate was between 3 m and 5 m per day (face length of 235 m at a depth of 400 m). The Tashan LTCC faces consisted of Chinese built four-leg, 1,300-tonne capacity shields. Coal clearance was achieved by AFC capacities of 2,000 t/hr (front) and 3,000 t/hr (rear), plus a maingate belt capacity of 3,500 t/hr on a 1.4-m-wide belt. Underground conditions on the face were excellent in all respects. Typical coal fragmentation onto the rear AFC was in the range of 100 mm to 200 mm particle size, with only isolated blocks requiring any additional breakage. Face shields were rarely yielding, other than some groups of rear legs in yield, which are often associated with top coal fracturing initiating just above/ahead of the start of the goaf shield.

Bulianta

Bulianta Mine is located close to the city of Ordos, in the Province of Inner Mongolia. It is a relatively new mine within the Shenhua Group, together with its sister mines, Daliuta and Shang An, both either operating or installing 7-m SPL faces at the time of the visit. Bulianta operates a 7-m-high SPL face using Chinese built two-leg, 1,800 tonne capacity, 2.06-m-wide, 76-tonne (weight) shields with a 7.5-m maximum height and an operating range of 3.2 m to 7.0 m. The full seam section (6.5–7.0 m) was being mined by the face. Web thickness is 865 mm. A significant feature of the shields is the three-stage flippers that provide face support to almost 60% (4 m) of the face height. The total annualized mine production was 30MTPA in 2011, coming from three SPL faces: one at 7-m height and two each at 5.5-m height.

The 7-m-high face was in its first panel of production at the time, but was budgeted to produce 13MTPA, or 43,400 t/day. Production from each shear off the face was 2,170 t/shear, and the face retreat rate at the time was averaging 430 m/month. These daily and monthly rates were being achieved, on budget, at the time of the visit.

The shearer was a 3,000 kW Eickhoff SL1000, fitted with 3.5-m-diameter drums, capable of cutting at a speed of 14.5 m/min. Coal clearance was by way of a Caterpillar 6,000 t/hr AFC powered by 3 x 1.6 MW drives. The maingate belt was 1.6 m wide.

Conditions in the mine were excellent. The access travel road, together with the full length of the maingate, was concrete paved for high speed transport. The coal was an anthracitic coal with what appeared to be relatively high strength and very little sign of structure. The mine had insignificant amounts of seam gas content. Gate roads were between 3.5 m and 4 m in height with roof and rib conditions standing “as cut,” with no sign of any deformation or instability. Support density in the maingate was 6 bolts/m in the roof, some cables, and 4 bolts/m in the ribs, which were also fully meshed. On the face, the coal stood well, with just some localized blocks slabbing off, typically less than 0.5 m across. Use of the extensive flipper face coverage was a critical component of face management.

GEOTECHNICAL FACTORS

It is clear from the above discussions that there are a number of geotechnical issues and challenges associated with both LTCC and SPL thick seam mining. In terms of LTCC, as was discussed by Hebblewhite (2001), one of the major considerations is

the assessment, and then predictability of effective, consistent and controlled top coal caving from above the goaf shield and immediately behind the face. Some advances have been made to address this challenge. Vakili and Hebblewhite (2010) developed an empirical Top Coal Caving Rating (TCCR) based on a combination of various seam geometric factors, stress conditions, coal properties, and coal seam structure. This was calibrated on a limited amount of available Australian and Chinese caving data. There is certainly room for far more refinement and development in this area, together with improved numerical modelling of top coal cavability.

A more recent and obvious area where further geotechnical research studies are required is to provide a means (empirical or numerical) for assessing SPL face stability in order to make a geotechnical judgment on the choice between LTCC and SPL. This is clearly a function of depth (hence face loading), coal properties and strength, and coal seam structure, and it may well be closely linked to any predictive capability for top coal caving.

The above two factors are the most pressing issues for consideration at the mining method selection/design stage of a project. The issue of face shield stiffness is another problem warranting ongoing consideration. The concerns expressed by Matla about high shield aspect ratios are one part of this issue. However, overall stiffness of the structure and the hydraulic system in these very tall supports is another factor worthy of consideration.

Finally, for any form of thick seam mining, surface subsidence is an inevitable consequence that must be taken into account. A 7-m-thick coal seam may result in surface subsidence between 2 m and 4 m or more, depending on depth and face width. This can require modifications to underground panel designs (e.g., narrow panels), or considerable surface preparation and rehabilitation. Thick seam operations in some Polish mines place sand backfill behind their longwall faces in order to reduce the maximum potential subsidence from a typical value of 60% of extraction height or more, to a more acceptable level of below 30%. However this comes at a great cost in terms of underground operational performance.

CONCLUSIONS

Thick seam mining technology and practice is developing at a great rate, with China leading the world in terms of high performing faces: 10MTPA LTCC mines and 13MTPA 7-m-high SPL faces. However the latter are currently only performing under quite benign geotechnical conditions. SPL operations with face heights between 4.5 m and 6 m are more common in both China and Australia, but are also not without problems. Factors such as coal strength and structure are critical, together with regional stress and depth considerations. LTCC on the other hand can operate in conditions of weaker and more structured coal, and it actually performs better under deeper and more highly stressed environments. There are significant needs for further geotechnical research and development to improve both top coal cavability assessment, and also SPL coal face stability.

ACKNOWLEDGEMENTS

The author wishes to thank all management and operations personnel at the mines visited as part of this study for their

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willingness to host underground inspections and provide valuable performance and technical information. The assistance of staff from Caterpillar (Bucyrus at the time) staff in the Beijing office for helping organizing a number of the mine visits and related travel logistics is also gratefully acknowledged.

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Industry Research Into Gas and Oil Well Protective Coal Pillar Design

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ABSTRACT

The Pennsylvania Joint Coal and Gas Committee Gas Well Pillar Study of 1956 is the basis for well protection pillar design in the US and many other countries. An update was necessary because the well construction common to the gas and coal industries as well as mine configurations, mine depth, and extraction thickness have changed considerably since 1956.

As initially envisioned the update would involve a similar empirical study that formed the basis of the initial investigation. However, there are no records of any gas or oil well failures related to coal mining since 1956 when mines followed the initial recommendation, making an empirical study problematic.

Thus, the study had to be more analytical, which in return required at least some fundamental research into the behavior of the interaction of coal mining and oil and gas wells. A call to industry about research they have undertaken resulted in obtaining excellent research on the subject from three companies; Alpha Natural Resources, Inc., CONSOL Energy Inc., and Range Resources. These research projects were initially undertaken for internal use and not previously published. John T. Boyd Company reanalyzed these studies and determined how their results would affect protective pillar designs.

These research projects pointed to the importance of horizontal movement and shear caused by subsidence and the strength of pillars and their foundations. The importance of geology is important in understanding horizontal shear, ground movement, and floor strength. The failure modes of casing and pipe needed to be understood so the threshold of these movements and stresses could be determined.

INTRODUCTION

The recommendations of the 1956 Pennsylvania Joint Coal and Gas Committee's Gas Well Pillar Study are the basis of well protection pillar design for Pennsylvania and other states. The Pennsylvania Department of Environmental Protection (PADEP), Bureau of Oil and Gas Management required an update to this study and recommendations because gas and oil well construction and coal mining have changed considerably since 1956. These

changes to common practice include well diameter, casing and pipe steel, cementing requirements, mine configurations, depths, and extraction thicknesses.

It was initially envisioned that the update would involve an empirical study similar to the initial investigation. However, there are no records of any gas or oil well failures related to coal mining since 1956 when mines followed the initial recommendations, making an empirical study problematic.

The study required an analytical approach, which in return required at least some fundamental research into the behavior of the interaction between coal mining and oil and gas wells. Requests to industry resulted in acquisition of excellent research on the subject from three companies: Alpha Natural Resources, Inc., CONSOL Energy Inc., and Range Resources. These unpublished research projects were initially undertaken for internal use and not analyzed in the way needed for this study. John T. Boyd Company reanalyzed these results and determined how they would impact protective pillar designs.

The industry research projects pointed to the importance of horizontal movement and shear caused by subsidence and the strength of pillars and their foundations. Geology is seen to be important in understanding horizontal shear and movement, and floor strength is key in understanding shear and confinement. The failure modes of wells need to be understood so the threshold of these movements and stresses can be determined.

ALPHA NATURAL RESOURCES, INC.

Cyprus Cumberland Resources Corp. (now Alpha Natural Resources, Inc. [Alpha]) completed a study (Luo, and Peng, 1997) in 1999 on the effects of longwall mining around gas well W-510 at the Cumberland Mine in Greene County, Pennsylvania. Alpha developed gate entries to this well and established a pillar around it. Figure 1 illustrates the configuration of the 3-Left Section at the well location.

The 200-ft wide (center-to-center) gate road section employed 16-ft wide entries and crosscuts. The entries were reported to be 8 ft high and the Pittsburgh Seam thickness was reported to be 6.1 ft. The crosscuts were driven 60 degrees off the entries. The two

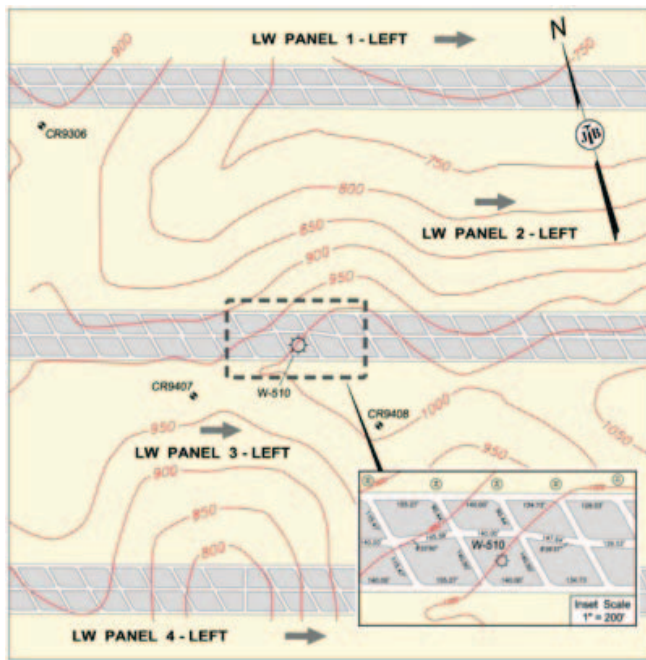


Figure 1. Cumberland Mine, 3-Left Section showing protection pillar, borehole location, and overburden thickness.

adjacent longwall panels are 900 ft wide with Panels 2-Left and 3-Left being approximately 8,000 ft and 7,600 ft long, respectively. The overburden at the well is 1,007 ft.

As the gate entries were developed toward the gas well, pillars were widened to protect the well. The pillar surrounding the well was widened from 100 ft to 120 ft but the pillar length was reduced from 130 ft to 104.3 ft.

The gas well was completed prior to mining in 1983. In the roof, pillar and floor of the gate, the well has a 4½-in-diameter steel pipe protected by an 8½-in-diameter casing. This outer casing is cemented to the borehole wall and cement fills the annulus between casing and pipe.

Before mining, Luo and Peng (1997) predicted the subsidence that could occur at the well and, after mining, the movement was surveyed (Reilly, 2002). The predicted and actual surveyed surface movements are presented in Table 1. Luo and Peng (1997) assumed that all ground movement and deformation transferred to the outer casing because this casing is cemented to the surrounding ground. The calculated strain and curvature in the outer casing after completion of the 2Left Panel indicated a compressive axial strain. Because of the lack of movement from the coal seam to 400 ft above the mine, no strain was calculated but the compressive strain increased from that point to the ground surface reaching a maximum axial strain of around 2.9×10^{-4} or about 8,500 psi compressive stress. This strain is less than the casing's permissible strain of 1.23×10^{-3} .

The WVU estimate of the parameters for casing steel in both compression and tension is shown in Table 2. Stress and strain caused by casing bending was also considered by Luo and Peng (1997). The maximum curvature on the outer casing caused by 2-Left Panel was estimated at around 2.3×10^{-6} 1/ft or a maximum

stress of 24 psi on the outer casing, which was well below the yield stress of 36,000 psi.

The axial strain and lateral curvature along the outer casing of the gas well after the extraction of 3-Left Panel was analyzed. The trend is that the axial strain increases toward the ground surface. It is still less than the permissible strain in the section within 600 ft above the coal seam. The maximum axial strain at the ground surface is about 3.25×10^{-3} which is above the ultimate allowable strain. It is above this allowable strain to a depth of 300 ft; therefore, the outer casing can plastically yield. It was noted that slippage along the outer casing and the well wall will reduce this strain.

Luo and Peng (1997) also investigated the effects on the inner production pipe, which carries the gas to the surface. He noted that the movement, stress, and strain on the inner pipe are due to surface subsidence and caused by shortening. Shortening and the resultant strain in the upper 2,210 ft of the pipe were predicted to all fall below allowable limits as shown in Table 3. The maximum strain generated in the pipe by its weight was determined to be 2.33×10^{-4} , and adding this strain and that produced by subsidence, the resulting strain was still below allowable limit.

Luo and Peng (1997), using the Analysis of Longwall Pillar Stability (ALPS) computer program (Mark, 1992), investigated the gate road pillar's stability. The highest load on the well protection pillar will occur when both longwall panels have retreated. For this condition, the ALPS determined a pillar stability factor of 1.31. This factor is reported to be larger than the stability factor of 1.17 recommended for the Pittsburgh Coal Seam in Pennsylvania indicating that the gate road pillar will be stable.

WVU determined that the gas well location in the larger gate road pillar placed it in an area where the stress distribution is relatively uniform. At that location, the predicted vertical stresses were approximated at 1,221 psi (virgin ground, no mining), 1,350 psi (when the gate road was developed), 1,790 psi (after the retreat of the first panel), and 1,930 psi (after both panels are mined). These values represent a 10%, 46% and, 58% increase over virgin vertical stress. These values were calculated assuming an overburden density of 158.4 pcf and a Mohr-Coulomb failure criterion. The values that were used to estimate the horizontal stress are shown in Table 4. Based on these pillar strengths and stress at the gas well location, Luo and Peng (1997) concluded that "This indicates that the failure of the coal surrounding the gas well is impossible."

Alpha implemented a program consisting of subsidence surveys to determine ground movement and a down-hole camera to observe the inner pipe. These surveys and observations were made before longwall subsidence affected the well, after the first longwall mined 2,000 ft past the well, and finally after the second longwall mined 2,000 ft past the well. The study reported these programs verified the predicted subsidence and effect on gas well structure (Luo and Peng, 1997).

However, what is described as plastic failure was observed to have occurred in the inner pipe 16.5 ft below the coal seam after the second longwall panel. This plastic failure was not associated with any pipe fractures and did not disrupt gas production. As a result of this plastic distortion, PADEP considers well W-510

Table 1. Predicted and Actual Surveyed Surface Movement at Well W-510.

After Retreat of	Predicted		Surveyed	
	Vertical Movement (ft)	Horizontal Movement (ft)	Vertical Movement (ft)	Horizontal Movement (ft)
2-Left Panel	-0.13	-0.3	-0.32	-0.4
3-Left Panel	-1.29	-0.1	-1.40	-0.1
<u>Note:</u> For horizontal movement, negative values are toward 2-Left Panel				

Table 2. West Virginia University Estimated Parameters for Casing Steel.

Ultimate Strength	54,000 to 60,000 psi
Yield Stress	36,000 psi
Young's Modulus	2.84×10^7 to 2.99×10^7 psi
Average Young's Modulus	2.92×10^7 psi
Average Permissible Strain	1.23×10^{-3}
Ultimate Allowable Strain	1.85×10^{-3} and 2.05×10^{-3}

Table 3. West Virginia University Predicted Shortening and Strain in W-510 Pipe.

After Retreat of	Maximum Shortening (ft)	Maximum Compressive Strain
2-Left Panel	0.13	5.88×10^{-5}
3-Left Panel	1.29	5.84×10^{-4}

Table 4. West Virginia University Predicted Stresses and Coal Strength Around W-510.

	Vertical Stress (psi)	Horizontal Stress (psi)	Confined Coal Strength (psi)
Development	1,350	579	2,640
1st Longwall	1,790	767	3,201
2nd Longwall	1,930	827	3,375

a failure since a kink resulted and the well could not be serviced or potentially properly plugged. In their opinion, the well has been altered and the integrity compromised even though the well continued to contain and produce gas.

Since this plastic failure below the coal seam was not anticipated, Luo and Peng (2007) found it necessary to evaluate the stress field in the floor strata under the pillar using a twodimensional finite element model. The model was 265 ft wide and extended from the mine floor to 40 ft below the coal seam. A single layer of sandy shale, based on the provided geological column, was modeled in the floor. The vertical stress in the model is the largest among the three stress directions with the maximum vertical stress around 2,000 psi. The maximum horizontal stresses, σ_2 and σ_3 , were determined to be 1,229 psi and 1,098 psi, respectively. The maximum stresses occurred near or at the pillar—floor contact with all stress decreasing with depth. The magnitude of these horizontal stresses are smaller than the uniaxial compressive strength of the sandy shale, typically in the range from

3,000 psi to 5,000 psi, concluding that the stresses under the pillar were not likely the cause of the inner pipe plastic failure.

Luo and Peng (1997) then assumed that weak parting, such as fireclay, is sandwiched between the sandy shale. This weak layer can fail or flow plastically under the floor stresses determined above. This flow can be toward the free surface of the drill hole. The flowing material then can compress the inner pipe equal to the vertical stress in the floor.

The inner pipe's outer diameter is 4.5 in., but the wall thickness was not known, thus Luo and Peng (1997) assumed likely thicknesses of $\frac{1}{8}$ and $\frac{1}{4}$ in. The corresponding maximum tangential stress in the inner pipe would range from around 15,000 psi to about 33,000psi. The vertical stress caused by gravity at the failure location is around 3,469 psi. The vertical stress caused by the shortening of the inner tubing in the upper 2,210 ft is approximately 17,462 psi. The principal stress on the inner surface of the steel tubing can be determined from the maximum tangential and total vertical stresses. For the thinnest pipe, the calculated principal stress at the location of the plastic failure is 39,100 psi. This stress is larger than the 36,000 psi yield stress for casing steel, and it is below the ultimate strength of pipe steel, which ranges from 54,000 to 60,000 psi. For thicker pipe, the principal stresses are smaller than the yield stress. The determination of principal stress was not discussed in the research report. The calculated stresses are shown in Table 5. Luo and Peng (1997) summarizes by noting that "... it is still unlikely for the determined principal stresses to cause the observed failure.

Discussion of Alpha's Findings

In trying to establish the cause of the plastic failure observed in the inner pipe 16.5 ft below the coal seam, BOYD requested from Alpha geologic or geophysical borehole logs near the W-510 gas well and received three driller's logs. Geophysical logs were also provided for two of those three holes. The three holes were located 600 ft (CR-94-08), 320 ft (CR-94-07), and 1,780 ft (CR-93-06) from gas well W-510, as illustrated in Figure 1. Geophysical logs for boreholes CR-94-07 and CR-94-08 show claystone (1.3 ft and 1.5 ft thick) at the depth of the noted plastic failure in the pipe as shown in Table 6. The driller's log for borehole CR-94-07 closely matched the geophysical log and it was noted in that log that the claystone layer was soft.

Claystone tends to be the weakest member of a coal sequence and it is common for a soft claystone to be located below a limestone in Greene County as it is here. A limestone bed exists in all three holes just above the location of the plastic failure. However, the drillers did not note the presence of claystone in two of the holes, CR-94-08 and CR-9306. However, a claystone was identified in the geophysical log of hole CR-94-08. Since both

Table 5. West Virginia University Predicted Stresses within the W-510 Pipe.

Wall Thickness (in.)	Tangential Stress (psi)	Vertical Stresses (psi)			Principal Stress (psi)
		Gravity	Shorting	Total	
1/8	33,029	3,469	17,462	20,931	39,102
1/4	15,059	3,469	17,462	20,931	25,785

Table 6. Stratigraphy below Pittsburgh Seam, 3-Left Section, Cumberland Mine.

CR-94-07 (geophysical log)			CR-94-08 (geophysical log)			CR-93-06 (driller's log)		
Rock Description	Thickness (ft)	Depth Below Coal (ft)	Rock Description	Thickness (ft)	Depth Below Coal (ft)	Rock Description	Thickness (ft)	Depth Below Coal (ft)
Carbonaceous Shale	0.7	0.7	Carbonaceous Shale	0.6	0.6	Gray Shale	2.70	2.70
Limestone	2.2	2.9	Limestone	2.3	2.9	Gray Sandy Shale	6.80	9.50
Shale	6.6	9.5	Shale	6.7	9.6	Gray Shale	2.60	12.10
Carbonaceous Shale	4.9	14.4	Carbonaceous Shale	5.5	15.1	Dark Gray Shale	2.65	14.75
Argillaceous Limestone	1.1	15.5	Argillaceous Limestone	1.1	16.2	Limestone	1.90	16.65
Claystone	1.3	16.8	Claystone	1.5	17.7	Gray Shale	1.55	18.20
Shale	-	-	Shale	-	-	Black Shale	0.75	18.95
-	-	-	-	-	-	Gray Limy Shale	-	-

geophysical logs identified the claystone, it is expected that a soft claystone existed at the depth of noted plastic pipe failure.

It is probably not coincidental that the pipe's plastic failure and the claystone coincide and that failure may be contributed to by the shear stress created from lateral movement of the weak claystone, as is common in pillar foundation failure. As this lateral movement in the claystone pushes against the stiffer casing, load on the pipe will increase as a clay wedge forms behind the pipe. This wedge would accumulate load increasing shear against the casing. If this occurs, the shear will dramatically increase. However, with a burial depth of 16.5 ft below the mine opening, BOYD could not model the type of free face that would allow such lateral movement.

WVU researchers noted that the vertical stress increased to 1,930 psi after both panels are mined. If this stress is projected to a depth of 16.5 ft below the coal and the Poisson's ratio of the claystone is taken as 0.3, then the horizontal stress is calculated to be 827 psi, as follows:

$$\sigma_H = \frac{\nu}{1 - \nu} \sigma_V = \frac{0.3}{1 - 0.3} 1,930 \text{ psi} = 827 \text{ psi}$$

This will only occur if the claystone moves against all sides of the pipe. If this movement entails plastic flow or failure of the claystone, the horizontal load against the pipe would approach hydrostatic. The original virgin horizontal stress, which existed before mining and drilling, would be reduced once the claystone is drilled, and dissipated once the claystone enters plastic flow or

fails. Thus, the only remaining horizontal stress is due to Poisson's ratio or hydrostatic loading.

The collapse mechanism for pipe and casing is a complex phenomenon because elasticity theory cannot directly determine the stresses in the pipe. The collapse load determination is dependent on the ratio of outer casing diameter and its wall thickness (D/t). Four collapse conditions exist, therefore, four collapse equations are needed; yield, plastic, transition, and elastic. In the case of W-510 the external pressure, P_T , that would collapse the casing would be transitional and occurs at:

$$P_T = Y_p \left[\frac{F}{D/t} - G \right]$$

Where Y_p is the steel casing or pipe yield strength, t , is wall thickness, and D , F , and G are factors specific to the type of casing and can be found the API Bulletin 5C3 (American Petroleum Institute, 1994).

Table 7 compares the applied horizontal stress to the collapse pressure of the pipe. The collapse pressure is at the elastic-plastic transition, which would generate the type of failure noted in W-510; thus it can be reasoned that the plastic failure of the outside casing caused the plastic failure of the inner pipe.

If the actual horizontal stress in the claystone is the average between the Poisson's and the hydrostatic stresses then the horizontal stress may be taken as $(827 + 1,930) \div 2 = 1,379$ psi.

Table 7. Comparison between Horizontal Stress and Casing Collapse Pressure.

Vertical Stress (psi)	Horizontal Stress (psi)		Collapse Pressure (psi)		
	Poisson's	Hydrostatic	Not Adjusted	De-rated	Allowable
1,930	827	1,930	1,610	1,290	850

As noted, the allowable stress of the casing is 850 psi, thus the horizontal stress is $1,379 \div 850 = 1.6$ times the allowable limit. To reduce the pillar bearing load, the pillar load has to be reduced by a factor of 1.6. This should come close to occurring if the stability factor is increased by this ratio from the present stability factor of 0.8 to 1.3 ($0.8 \times 1.6 = 1.3$). To account for the inaccuracies inherent in this approach, the stability factor was increased to 1.5, which is 0.2 higher than the National Institute for Occupational Safety and Health's (NIOSH) recommendation of 1.3. Finally, using ALPS, BOYD determined the required pillar size at a stability factor of 1.5. This 1.5 stability factor pillar resulted in a bearing load decrease of 1.6, which supports the above procedure. BOYD used these findings in recommending stability factors for the Mark-Bieniawski equation (ALPS and ARMPs) when designing a protection pillar.

CONSOL ENERGY INC.

In 2009, Dr. Su of CONSOL Energy Inc. (CONSOL) completed two-dimensional finite element analysis simulations on a hypothetical CNX gas well using ABAQUS (Dassault Systèmes, 2002). CONSOL modeled core hole NV-057, which was completed to a depth of 1,084 ft while intersecting the top of the 5.7-ft-thick Pittsburgh Seam at a depth of 1,032 ft. For the model, CONSOL assumed the well to be located at a hill top and drilled into an abutment pillar of an abutment-yield (three-entry) gate road system. The center-to-center measurements of the pillars are 160 ft and 70 ft with 16 ft wide entries resulting in pillar widths of 144 ft and 54 ft.

Dr. Su was the first to introduce the use of horizontal slip elements into Finite Element subsidence models, which has dramatically improved subsidence analysis. He employed such elements in this model at contacts between hard (sandstone, limestone, etc.) and softer (shale, claystone, etc.) stratum. The models were completed after longwalls passed on either side of the gate entries and the horizontal slip and horizontal and vertical (Figure 2) stress above and below the abutment pillar were reported in 20-ft increments across the pillar. These figures show the calculated ground slip and strain above and below the center of the abutment pillar, as well as the slip and strain twenty feet to either side.

BOYD reviewed the log of core hole NV-057, assigning stratigraphic names to the various layers. The contacts between hard and soft stratigraphic members were overlaid on Figure 2. The overlays show more slip and greater strain occur at the contacts between stiff and soft strata.

Discussion of CONSOL's Findings

CONSOL's maximum predicted shear above the top of the coal and center of the abutment pillar is 210 psi and occurs at the base of the Benwood Limestone where it contacts shale. The largest shear below the pillar center is 42.7 psi. Both values are well below

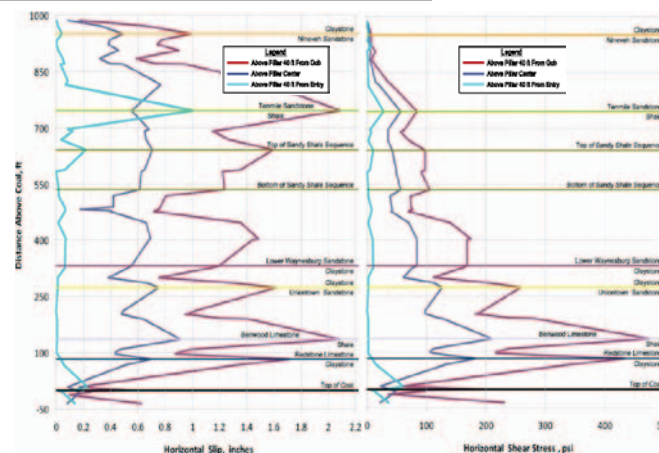


Figure 2. Predicted horizontal slip and shear stress due to longwall mining; at 40 ft from gob, pillar center, and 40 ft from entry.

the allowable shear of 1,230 psi that BOYD has recommended for oil and gas wells.

The change in vertical stress, and not the total vertical stress, affects the casing. The vertical stress that can be transferred from the surrounding rock to the casing is dependent on the amount of slip that occurs between the rock and casing and the elastic modulus of the rock and steel. If one assumes the vertical stress in the rock is transferred to the casing without change and is the same in both the rock and casing, then the maximum change in vertical stress of 1,700 psi, as determined by CONSOL, is below the casing's allowable body stress of 2,030 psi that BOYD has recommended for oil and gas wells. Still, the rock stress and allowable stress are close enough to be a concern.

RANGE RESOURCES

In 2012, Rostami, Elsworth, and Watson (2012) and others of Pennsylvania State University (PSU) completed numerical modeling studies to evaluate the distribution of stresses, strains, and deformations along a projected vertical gas or oil well caused by longwall mine subsidence. The stresses, strains, and deformations above the gate road pillars were modeled after each adjacent longwall panel was mined. Panels were projected to be 1,200 ft wide and the assumed horizontal coal seam was 6.6 ft thick and located below a horizontal surface. The induced movements determined by these models were used to evaluate longwall subsidence effects on gas and oil wells drilled into the pillars of the gate road using the following two gate road geometries:

- A three-entry system which included one row of yield pillars, 49 ft wide, and one row of abutment pillars, 98 ft wide, separated by an entry 16 ft wide resulting in an accumulated pillar width of 148 ft.
- A single row of pillars with a width of 197 ft.

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For this analysis, a vertical gas well was placed in the center of the yield pillar and at various locations within the abutment pillar. For the gate road that contained a single row of pillars, the well was placed in the center of the pillar.

Three scenarios were completed representing seam depths of 328 ft, 656 ft, and 984 ft. Rostami, Elsworth, and Watson (2012) evaluated overburden behavior after extraction of the first and then the second longwall panel on either side of the gate road. Three geologic settings were examined:

- Homogeneous strata calibrated against a measured subsidence profile (I-79 undermining)(Gutierrez, Vallejo, and Lin, 2010).
- Alternating 32.8-ft-thick layers of shale with elastic moduli of 725 ksi and sandstone with elastic moduli of 2,900 ksi.
- Layered as in previous condition but with the shale—sandstone contact having a friction angle of 20 degrees.

The two-dimensional numerical models were analyzed using both FLAC by Itasca International Inc., a finite difference program, and PHASE by Rockscience, Inc., a finite element program. FLAC was considered to be more relevant because it reproduced a subsidence profile that closely follows the ground subsidence models. A two-dimensional model, using FLAC, was constructed perpendicular to the direction of panel advance. All materials in the model were modeled as elastic-plastic with elastic moduli of 2,900ksi, Poisson's ratios of 0.25, and density of 165 pcf. The deformation response was yield combined with Mohr-Coulomb failure and with a relative contact stiffness of the roof and floor once the coal is removed. The model was calibrated using the subsidence survey and tilt meter measurements of I-79 (Gutierrez, Vallejo, and Lin, 2010) when it was undermined by longwall panels 51 and 52 of the Cumberland Mine. These panels were at depths that varied from 640 ft to 800 ft. To calibrate the model, Rostami, Elsworth, and Watson (2012) varied cohesion, friction angle, and seam thickness at different depths to match the measured subsidence results.

Rostami, Elsworth, and Watson (2012) evaluated several combinations of geologic settings and gate configuration:

- Homogeneous strata and three-entry system.
- Alternating 32.8-ft layers of shale and sandstone and three-entry system.
- Alternating 32.8-ft layers of shale and sandstone and two-entry system.
- Layered as above with bed slippage and three-entry system.
- Layered as above with bed slippage and two-entry system.

For the three-entry cases, the first longwall retreated on the yield pillar side of the longwall. Our discussion focuses on the first or homogeneous case because values used were calibrated against actual Pennsylvania subsidence and on the fourth case as it is closest to simulating actual conditions.

For the homogeneous strata, the distortions assume continuous lateral displacements along the well length. Rostami, Elsworth, and Watson (2012) conjectured that shear movement along beds can concentrate the displacements at selected locations. This concentration may occur between thick rigid beds such as sandstone and softer beds such as shale. They approximated this concentration by multiplying the local shear strain by the bed

thickness resulting in an offset at the bed boundary. To calculate this offset, bed thickness was assumed to be 3.3 ft and 33 ft as they considered these to be reasonable bounds of bed thicknesses in coal bearing strata. Rostami, Elsworth, and Watson (2012) concentrated on the distortions caused by lateral displacement of the well bore during subsidence noting that a well could not survive an offset greater than its diameter if applied over a short distance. The shear results for the homogeneous model for a mine depth of 984 ft and above the gate road are presented in Table 8. The retreat of the second longwall panel counteracts the movement from the first longwall panel.

The horizontal movements above the gate road were also determined for the homogeneous case. These movements are largest at the surface and are largest for the 328-ft-deep seam with horizontal movement of approximately 9.8 in after the first longwall is completed. Horizontal strains above the gate road after the first longwall panel was extracted are greatest at the surface and approach 20 millistrains in extension at the surface and reduce with depth to below 5 millistrains at a depth greater than 50 ft. Vertical strains are below 6 millistrains and are largest near the surface and above and below the pillar. In the model, these strains are tensile at the surface and compressive near the seam.

Rostami, Elsworth, and Watson (2012) estimated the shear strain and horizontal displacement around a three-entry gate system consisting of a single row of yield pillars and a single row of abutment pillars with the strata being comprised of alternating layers of sandstone and shale. Bed contacts between these layers were allowed to delaminate and slip. This condition examined the effects of alternating layers of shale and sandstones with their contrasting stiffnesses and strengths. Each layer was assumed to be 32.8 ft thick with the coal seam placed between two layers of shale. The assumed properties of these layers are shown in Table 9.

Shear strains within the layers generated in this model are approximately one half of those shown in the homogeneous case. Rostami, Elsworth, and Watson (2012) opine that bedding slip dissipates some of the shear stress that would have occurred within the rock layers. Thus, shear movement is concentrated at the bed that separated one layer from the next. Shear displacements of the beds are largest above the panel and reached a maximum of about 8 in for a 33-ft layer above the pillar edges of the first retreated longwall panel. The shear offsets then reduce to around 4 in over the pillars.

COMPARISON OF CONSOL AND PSU RESULTS

The comparison of results from CONSOL and Dr. Rostami shows similar findings. These two studies can only be compared for shear displacement as this is the only area of overlap. CONSOL's seam depth for their analysis is 1,032 ft, which is close to Dr. Rostami's depth of 984ft, so offset is compared at these depths. Here, Dr. Rostami reported magnitude of change while CONSOL reported scaled change. These changes are reported in Table 10.

CONSOL's locations for the maximum offsets are above and below Dr. Rostami's location of 164 ft above the coal and all of CONSOL's stiff beds, except one, are between Dr. Rostami's thicknesses of 3.3 ft and 32.8 ft. Also, all of CONSOL's offsets are between Dr. Rostami's offsets of 0.04 in and 4 in. This suggests Dr. Rostami's results are supportive of CONSOL's research.

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Table 8. Dr. Rostami's Subsidence Model Results.

Scenario	First Mined Panel				Second Mined Panel			
	Peak Strain	Peak Strain Depth (m)	Bed Thickness (m)	Shear Offset (cm)	Peak Strain	Peak Strain Depth (m)	Bed Thickness (m)	Shear Offset (cm)
1	10^{-3}	0	1	0.1	10^{-3}	250 to 300	1	0.1
			10	1			10	1
2 through 5	10^{-4}	50	1	-	10^{-3}	250	1	0.1
		-	10	-			10	1

Table 9. PSU's Assumed Material Properties Used in Models.

Rock Type	Elastic Modulus ($\times 10^6$)	Poisson's Ratio	Friction Angle (Degrees)	Cohesion (psi)
Shale	0.725	0.25	30	3,046
Sandstone	2.900	0.25	30	12,760
The friction angle between layers was assumed to be 20 degrees.				

Table 10. Comparison of PSU and CONSOL Determined Offset.

		Location Above Coal (ft)	Thickness of Stiff Bed (ft)	Offset (inches)
CONSOL		109	7.6	0.47
		135	4.5	0.91
		191	1.6	0.64
		193	9.8	0.60
PSU	Homogeneous	164	3.3	0.04
		164	32.8	4.00
	Layered	328	32.8	4.00

Impact of Industry Research on Protective Pillar Recommendations

Research on the impact of coal mines on oil and gas wells is limited, thus any study will have an impact, and possibly a disproportionate impact, on any developed recommendations. Of the three studies presented here, the Alpha study had the largest impact on BOYD's final recommendations to the PADEP. This study involved an experiment using a producing gas well located in a longwall gate and being affected by two longwalls. The failure of the well, even if mostly plastic-elastic transition yield, did give the opportunity to evaluate gate pillar design and develop procedures to mitigate such occurrences in the future.

The CONSOL and PSU studies, which support each other, point to the importance of horizontal movement and shear on gas and oil wells allowing the establishment of standards to limit this movement and stress at the well.

It is interesting as well that existing and accepted procedures were used to conduct and evaluate these studies. This supports the use of such available tools as ALPS, ARMPS, and numerical

modeling to understand and to mitigate the impact of mining on wells.

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Influence of Roof/Floor Interface on Coal Pillar Performance

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ABSTRACT

Pillar design formulae have existed for decades, and the guidelines for design stability factors are generally accepted by regulatory agencies and broadly used within the coal industry. Nearly all design pillar formulae are based on geometrical parameters of pillars resulting from empirical investigations, with primary focus on the pillar width-to-height (W/H) ratio and associated shaping constants from regression. The most widely used modern pillar design formula in the coal industry, the Mark-Bieniawski equation, also adheres to this principle.

Common practice when applying Mark-Bieniawski equation is to use a coal compressive strength of 900 psi. This reduces the design equation to sole dependence upon geometry. However, recent studies have shown that pillar strength is highly dependent on roof and floor properties, particularly the interfaces between the roof-pillar and pillar-floor. This study utilized numerical modeling using FLAC^{3D} to evaluate the relative effects of varying roof and floor interface and mechanical properties.

The results confirm that pillar strength is significantly influenced by the properties of the coal pillar interfaces with roof and floor. The mechanical properties of the roof and floor material have only a minor impact on pillar strength when compared with interface effects. These results were used to formulate design recommendations for existing pillar design methods.

INTRODUCTION

Coal pillars provide the most elementary yet central support system in underground coal mining. While many mechanical principles are well understood, the mechanical response and behavior of geologic material is governed by an array of material properties and interactions. This results in a large spectrum of possible responses to stress. Therefore, changes in geologic environment and/or associated properties often necessitates modification of initial design methods due to alteration of rock mass behavior and deviation from initially applicable geologic conditions (Peng, 2008). Coal pillar design is conventionally two-fold, comprising of load estimation and comparison with estimated pillar strength. While numerous factors affect both phases, the focus of this study is the estimation of pillar strength.

Traditionally, pillar strength is empirically related to the pillar geometry, more specifically the width-to-height (W/H) ratio, and in situ pillar strength. Using statistical methods, numerous pillar design formulas have been created over the years as a function of this geometric parameter and curve-fitting constants. The Mark-Bieniawski equation has found the widest use in modern U.S. coal operations by offering a modified version of the Bieniawski formula to account for difference in strength and behavior of rectangular pillars (Mark, 1999):

$$S_p = S_1 \left[0.64 + 0.54 \frac{w}{h} - 0.18 \left(\frac{w^2}{Lh} \right) \right]$$

where: S_p = Pillar strength

S_1 = In-situ coal strength

w = Pillar width

h = Extraction height

L = Pillar length

This formulation is the design mechanism used in the Analysis of Retreat Mining Pillar Stability (ARMPS) software to determine pillar strength. ARMPS uses a mechanical approach to calculate expected vertical loading and abutment stresses for a variety of typical extraction environments including retreat mining. A stability factor is calculated as the ratio of pillar strength to this calculated load. A large database was constructed using this systematic approach and logistic regression techniques were employed to derive design standards as a function of stability factors that would theoretically predict success (Figure 1). This approach concluded that an ARMPS Stability Factor meeting or exceeding a value of 1.50 would result in success, categorized by effective resource extraction without significant ground control incident (Mark, 2010). Though the latest ARMPS guidelines contain misclassifications of some case histories, Mark (2010) concluded

that these misclassifications were all pillar squeezes, which are inherently less dangerous than pillar bursts/bumps or collapses.

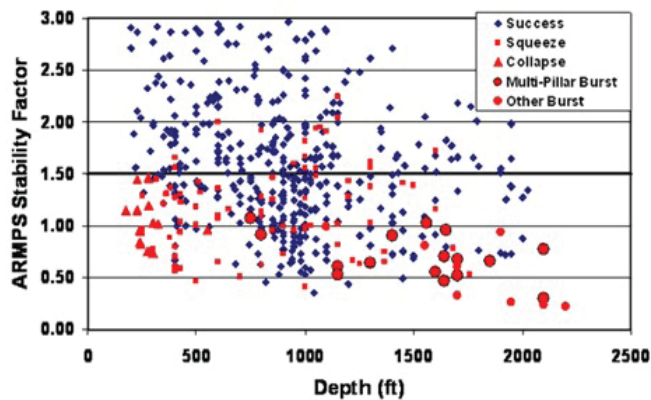


Figure 1. ARMPS 2010 Case History Database (Mark, 2010).

When evaluating the root cause of misclassification, it is essential to evaluate both the calculations of pillar loading and pillar strength. When considering pillar loading, it is important to note that the ARMPS case histories are all single seam stress environments, thereby reducing some of the anticipated loading complexity typically associated with multiple seam environments. Many of the case histories involved some form of retreat mining, necessitating the formulation of a gob loading mechanism. Gob loading and abutment distribution is a complex exercise that must consider localized geologic characteristics of the seam, surrounding strata, and in situ loading. However, given the inherent inability to obtain accurate stress measurements in gob areas, maintaining a consistent mechanical approach is the best option. Combined with a good knowledge of tributary loading and pressure arch theory, a significant portion of misclassifications due to errors in predicted pillar loading have been effectively reduced.

Regarding pillar strength, an initial concern is the accuracy and determination of in situ coal strength. While significant databases of coal strength testing data are available, Mark and Barton (1997) concluded that pillar strength estimation and ultimately design is much more reliable and successful when using a uniform in situ coal strength value of 900 psi. This cast doubt on the validity of laboratory testing to determine in situ coal strength. Furthermore, this relationship indicates a primary dependence on geometry without concern for local geologic environment or rock mass properties.

Iannacchione (1990) suggested that additional mechanisms other than geometry, particularly coal pillar interfaces, can produce a significant deviation from projected pillar behavior. Su and Hasenfus (1996) reaffirmed this finding and concluded that to accurately assess pillar strength and behavior, not only should the coal characteristics be understood, but the surrounding strata properties (strength, discontinuities, composition, etc.) are of even greater importance. This is especially true for large (high W/H ratio) pillars. It is understood that one of the primary drivers of pillar strength is confinement, a widely accepted fundamental concept of mechanical theory. Confinement is influenced by many factors, chiefly the in-seam and near-seam strata composition and available constraint, which results from inherent frictional

properties and shear strengths (Su and Hasenfus, 1997). Studies suggest that varying the available frictional and shear strength of coal pillar interfaces has a significant impact on the development of confining pressures. Lu et al. (2008) reported high confining stress gradients, represented by minimum principal stress contours, for increasing W/H ratio and strains. This is logical because smaller pillars, which derive their strength more from inherent material properties than development of confinement, begin yielding under increasing stress and exhibit strain-softening behavior. Larger pillars may act as strain-hardening, a result largely owed to significant minimum stress generation that maintains an elastic core.

A typical stratigraphic sequence was modeled utilizing the finite-difference code FLAC^{3D} (Itasca, 2009). FLAC^{3D} is a common numerical modeling software used in geotechnical applications in the mining and civil industries. This basis will be compared with two additional cases. The first comparison is made with strong roof and floor as expected with massive sandstone. The second comparison is made with typical roof and weak floor as represented and expected with the presence of fireclay. For all three cases, the interface strength will be varied. While the ARMPS method accurately classifies the majority of case histories, significant deviation in ultimate pillar strength and behavior is expected for substantial differences in geologic environment. These conditions likely account for a significant portion of the misclassifications.

MODEL CHARACTERISTICS

Model Configuration

It has become somewhat standard procedure to model pillar behavior with a one-quarter section due to inherent symmetry of the square or rectangular pillars typically used in underground coal mines. This also offers the advantage of greater result clarity and accuracy through increased zone density without significant increase in the total number of zones. Three pillar widths at a 20-ft entry width for a 6-ft coal seam were selected to represent three distinct W/H ratios: 4 (24-ft pillar width), 7 (42-ft pillar width) and 10 (60-ft pillar width). The bottom of the model was fixed while the sides were fitted with roller boundary conditions. The model was submitted to an initial vertical and horizontal stress field at a ratio of 2:1; this is representative of a typical in situ stress environment in deeper cover coal mines. A vertical velocity was applied at the top of the model at a constant value of -5×10^{-5} ft/step. This allows average pillar strength and deformation to be recorded and the stress-strain curve to be computed via a fish function (Itasca, 2009). The model was divided into several zone groups including a roof, seam, floor, and base (Figure 2). The roof and floor/base each totaled 30 ft in thickness, respectively.

Material Properties

Most material properties were selected to represent average material properties from a range of values. This includes a natural modification for geologic material as a function of zone group and a coal strength representing the traditional 900 psi value. Stiff clay properties as reported by Budhu (2011) were used to derive fireclay strength parameters (Table 1).

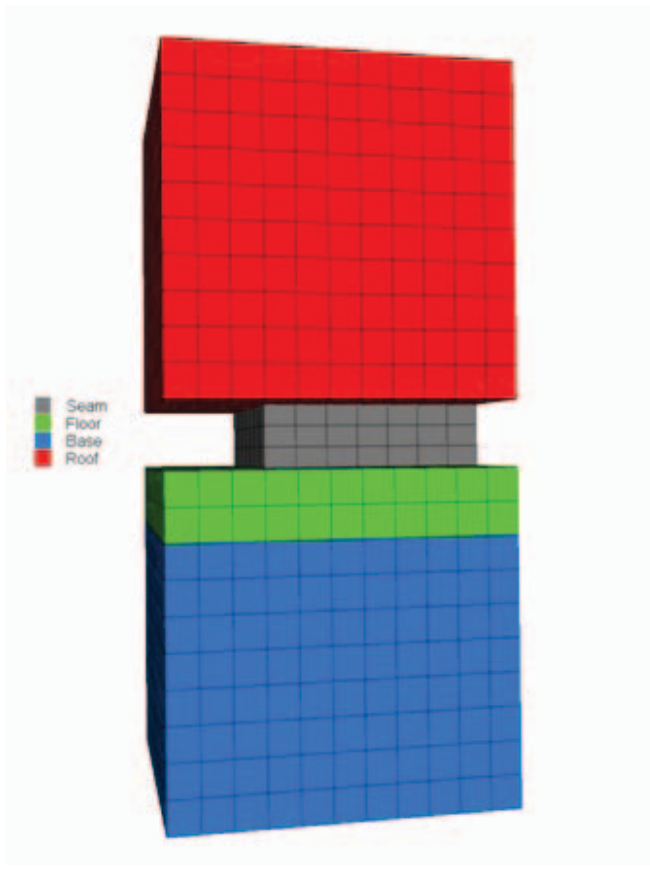


Figure 2. W/H 7 Pillar Model in FLAC3D.

Table 1. Strata Material Properties
(Lu et al., 2008, and Budhu, 2011).

Material	K (KSI)	G (KSI)	σ_t (PSI)	c (PSI)	ϕ (Deg)
Sandstone	1,350	1,140	208	1,650	41
SS Floor	1,600	1,350	208	1,030	41
Shale	722	394	150	1,000	30
Sh Floor	642	296	150	625	30
Coal	134	375	22	235	25
Fireclay	750	160	50	100	20

Constitutive Models

The coal seam was modeled as non-linear Mohr-Coulomb strain-softening material with cohesion (Table 2) and frictional softening (Table 3) as a function of plastic strain. Iannacchione (1990) concluded coal pillar behavior would most accurately be modeled with a non-linear Mohr or Mohr-Coulomb failure criterion to reduce the yield zone and core stress magnitudes. All rock strata were assumed to follow a linear-elastic Mohr-Coulomb model.

Table 2. Coal Seam Cohesion vs. Plastic Strain
(Lu et al., 2008).

Plastic Strain	Cohesion (PSI)
0.000	235
0.100	59
1.000	59

Interface Properties

Interface properties directly affect the resultant pillar strength. Iannacchione (1990) concluded interfaces would best be modeled with non-linear coulomb behavior. This acts to realistically limit the strength of interfaces and initiate slip. One established method of representing this non-linear mathematical response is the selection of a bi-linear function representing coulomb sliding (Lu et. al., 2008). Through the use of a fish function, the interface normal stress was evaluated at all interface zones and replaced with the applicable cohesion and friction angle. Iannacchione (1990) documented a list of generalized interface properties for modeling coal pillars:

1. Interface friction angle should range between 10° and 20°.
2. Cohesion should range between a value of 0 psi and 145 psi.

A bi-linear interface model has been supported by results from direct shear strength tests of existing strata contacts (Peng, Patrick, and Khair, 1983). Following these recommendations and maintaining a bi-linear model, three distinct interface models (Figure 3) were created with initial cohesion values as follows: 75 psi, 150 psi, and 300 psi. The friction angle was initially assumed to be 20° but decreased to 10° at 500 psi normal stress.

When considering the case of fireclay floor, definitive properties are not well documented. However, one of the defining characteristics of fireclay is inherently low material strength that may be further reduced by the presence of water. This often results in floor heave and pillar punching associated with an inherently weak interface between the floor and pillar. Therefore, the fireclay floor was modeled as a linear interface with zero cohesion and a friction angle of 10°.

MODELING RESULTS

All resultant stress-strain curves were statistically fitted to 5th order polynomials in an effort to smooth data and eliminate scatter in the data resulting from the iterative solution method used in FLAC^{3D}. The coefficient of determination, R^2 , for all polynomial regressions had a minimum value of 0.99 for all data sets, indicating strong correlation and low proportion of variability. For reference, the pillar strength, as determined for each W/H ratio according to the Mark-Bieniawski relation, is provided for each data series.

Table 3. Coal Seam Friction Angle vs. Plastic Strain (Itasca, 2009).

Plastic Strain	Friction Angle (Deg)
0.000	35.0
0.005	32.5
0.500	30.0

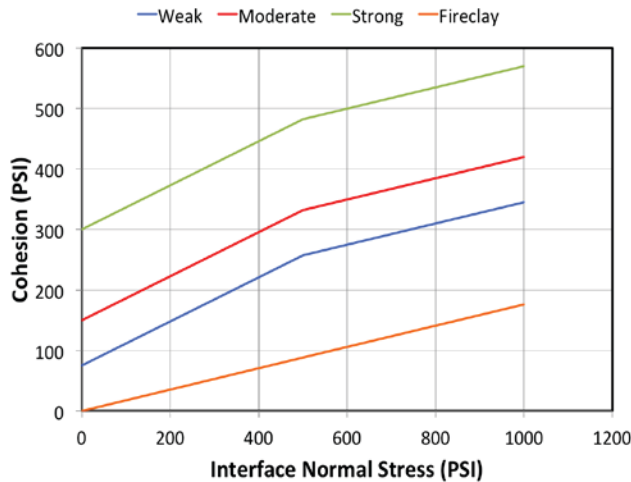


Figure 3. Interface Bi-linear Strength Models.

Pillar Behavior/Strength

As expected, the W/H 4 pillar demonstrated strain-softening behavior (Figures 4, Figure 7, Figure 9, & Figure 10). The W/H 7 pillar appeared to define the transition zone between strain-softening to strain-hardening, primarily as a function of interface strength, with the middle case demonstrating perfectly plastic behavior (Figure 5 & Figure 8). This result largely mirrors previous modeling results (Lu et al., 2008). Overall, the effect of interface strength was more pronounced for the larger W/H ratios as predicted, but in all cases the mechanical response and relative strength is influenced by the coal pillar interface properties. The W/H 10 pillar exhibited strain-hardening behavior for normal geologies at all three interface strengths (Figure 6 & Figure 9). Weak floor with poor interface strength also resulted in the W/H 7 pillar behaving strictly as strain-softening (Figure 11) and the W/H 10 pillar exhibiting nearly perfectly plastic behavior (Figure 12).

When examining the peak pillar strength across different types of strata, the peak strength remained very consistent between the shale roof/floor base and the massive sandstone case. However, when compared to fireclay floor, the peak pillar strength is reduced by an average of approximately 22% (Figure 13). This strength reduction was more pronounced for higher W/H ratios, with the W/H 10 pillar strength reduced by approximately 26%. While this effect has been documented, it is primarily driven by the issues with interface properties presented by fireclay floors, especially with elevated moisture contents. It is believed the lack of sufficient confinement generated by the weak interface (cohesionless) is the primary reason a significant reduction in strength is realized. While some issues with weak floors may remain (e.g. floor heave), if water is not allowed to accumulate, the effect on pillar strength

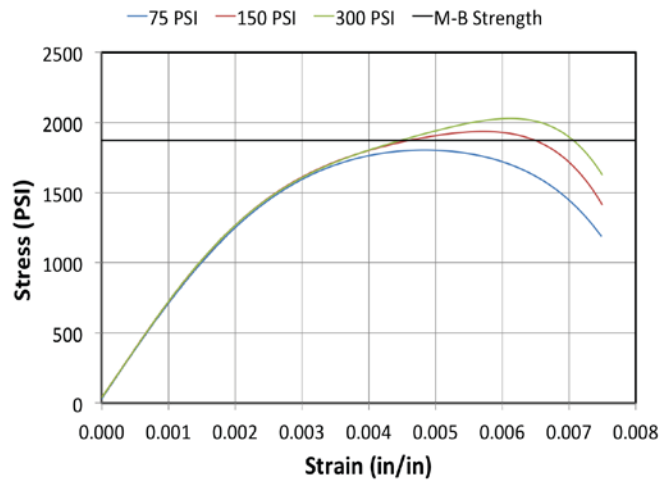


Figure 4. W/H 4 Pillar with Shale Geology at Varying Initial Interface Cohesion.

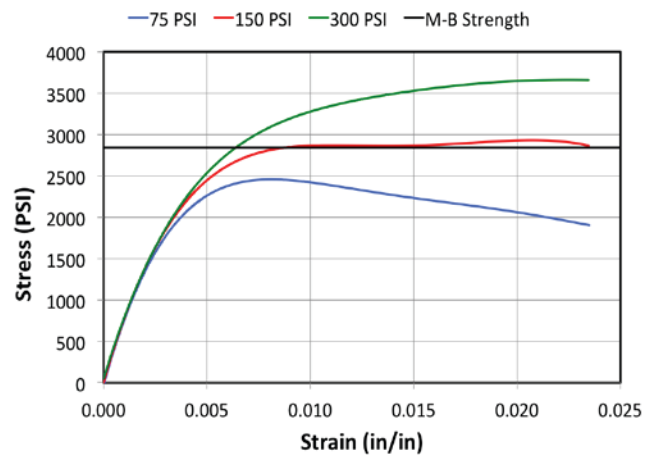


Figure 5. W/H 7 Pillar with Shale Geology at Varying Initial Interface Cohesion.

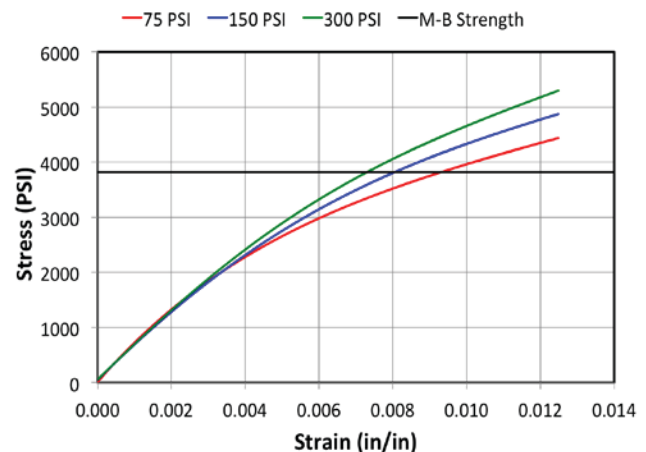


Figure 6. W/H 10 Pillar with Shale Geology at Varying Initial Interface Cohesion.

from weak floor would be significantly reduced if the interface properties remain unaltered.

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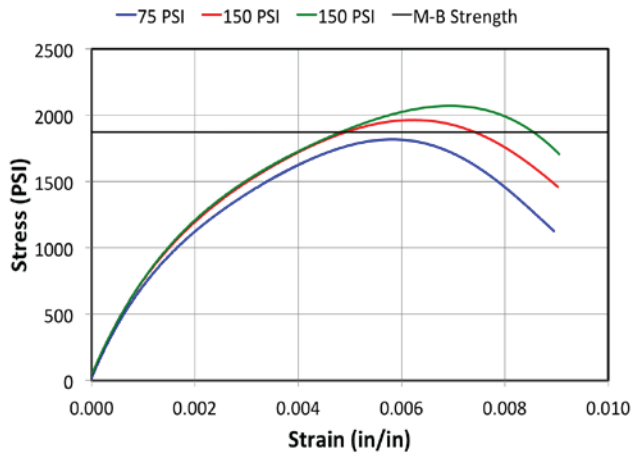


Figure 7. W/H 4 Pillar with Sandstone Geology at Varying Initial Interface Cohesion.

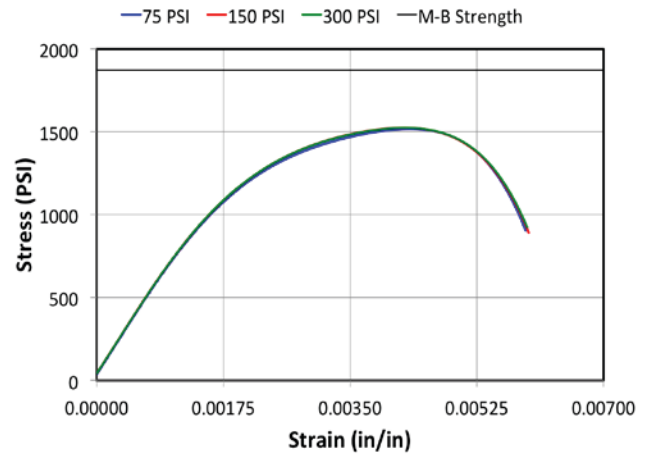


Figure 10. W/H 4 Pillar with Shale Roof and Fireclay Floor at Varying Initial Interface Cohesion.

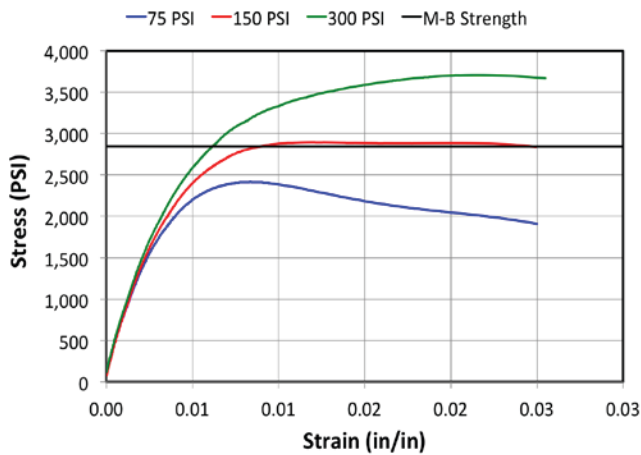


Figure 8. W/H 7 Pillar with Sandstone Geology at Varying Initial Interface Cohesion.

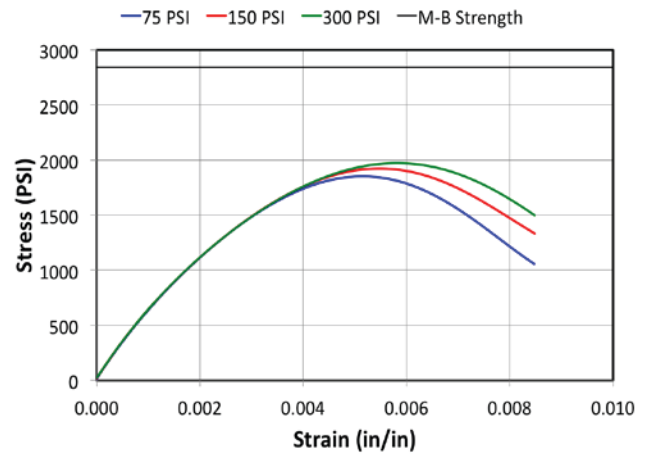


Figure 11. W/H 7 Pillar with Shale Roof and Fireclay Floor at Varying Initial Interface Cohesion.

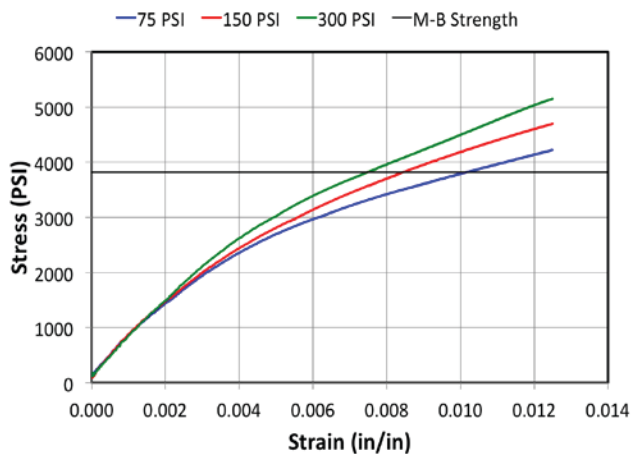


Figure 9. W/H 10 Pillar with Sandstone Geology at Varying Initial Interface Cohesion.

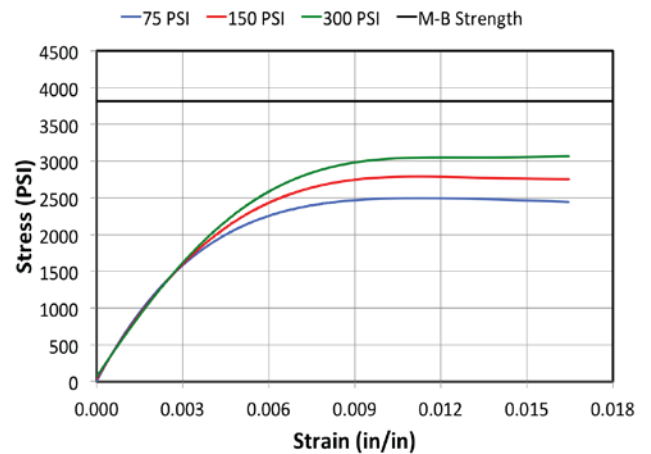


Figure 12. W/H 10 Pillar with Shale Roof and Fireclay Floor at Varying Initial Interface Cohesion.

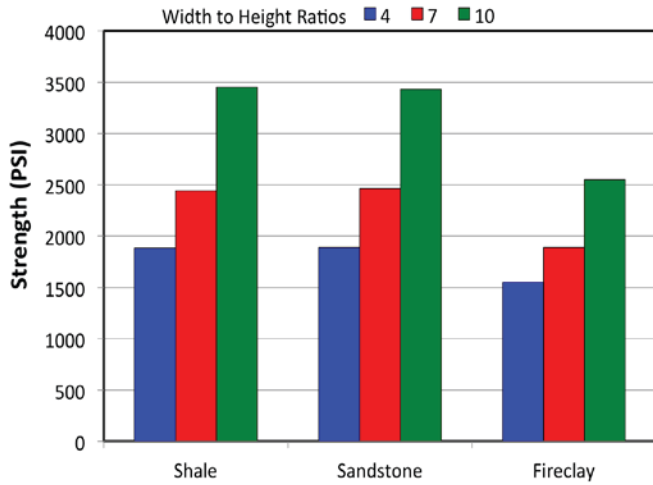


Figure 13. 75 PSI Interface Peak Pillar Strengths.

For a typical coal environment with average interface properties (150 psi), the determination of pillar strength by the Mark-Bieniawski relationship is confirmed. However, with significant deviation in interface strength, the peak pillar strength can be drastically changed. For example, when considering the W/H 7 pillar with shale roof and floor, the peak pillar strength was altered by an average of ± 600 psi.

ARMPS Database (SF vs. CMRR)

The statistical evaluation of stability factor as a function of CMRR (Figure 14) is inherently marginalized for a couple of reasons. First, only case histories with definitive CMRR values (330 in total) were considered. This reduced the total available case history quantity and diversity, a sentiment mirrored by Mark (2010). Furthermore, the case histories that do have specific CMRR values are coincidentally weighted towards higher stability factors, a result that hinders comprehensive statistical analysis.

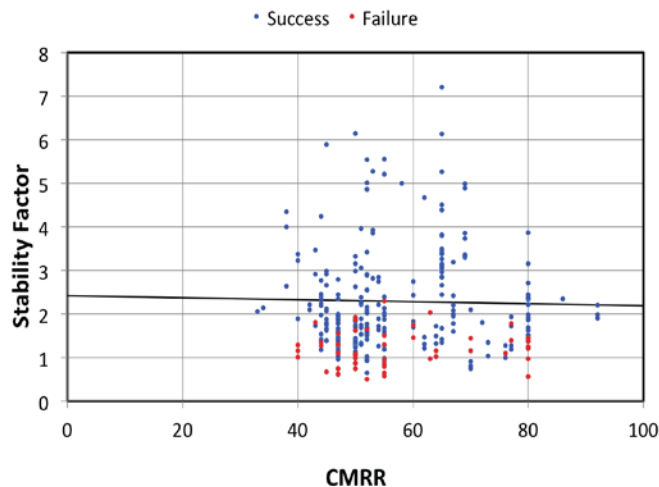


Figure 14. ARMPS Database (CMRR vs. SF).

Subsequently, the results do not appear to be very telling of a relationship between roof strength/quality and stability factor. While CMRR does provide an effective engineering tool to evaluate the quality of coal mine roof, it may not be highly

indicative of pillar strength. Pillar strength is reduced from poor roof quality, but it has a much greater dependence on coal pillar interface, a measure not quantified or evaluated in the case histories. This concept is also supported by the modeling results, which found that interface properties played the most important role in pillar strength and behavior. Only minimal variances in peak strength and mechanical behavior were observed for differences in roof/floor material properties or composition.

Design Recommendations

Most modern U.S. coal pillar strength predictive models utilize the Mark-Bieniawski pillar strength formula, including empirical (ARMPS, Analysis of Multiple Seam Stability (AMSS)) and numerical codes (LaModel (Heasley, 2007)). To enhance the prediction of success or failure, the following recommendations are made:

1. Prior to implementing roof/floor interfaces, coal pillars must be constructed and represented with appropriate material properties. While many constitutive models are available, this study utilized non-linear Mohr-Coulomb failure criterion, with the standard 900 psi strength. While Mark and Barton (1997) do not imply that coal compressive strength is uniform or constant, deducing a correlation between values obtained from laboratory testing and in-mine seam strength is difficult to justify. If adjusting coal strength purely on the basis of mechanical strength, do not deviate more than a few percent from the standard 900 psi compressive strength without strong justification. The recommendations of Su and Hasenfus (1996) could be good guidelines to follow. This leads to practical in situ seam strength range between 840 psi and 960 psi.
2. Quantify roof and floor material properties and make reasonable assessments of interface strength/behavior. This can often be done with observation of previous in-mine pillar behavior. This is especially important in high stress environments and high W/H ratio pillars.
3. Utilize a numerical code (e.g. FLAC^{3D}) to determine a pillar strength that incorporates the site-specific geology. This is especially important when confronted with unique lithological environments such as clay partings, weak floor, or strong massive sandstones. It is important to validate the theoretical results produced by the model.
4. Calculate an effective coal strength that would yield equivalent pillar strength from the Mark-Bieniawski pillar strength formula.

This effective strength value should provide a higher degree of accuracy when determining site-specific pillar strength using existing techniques and standards (ARMPS, AMSS, ALPS, LaModel) because these methods cannot directly account for localized site conditions. It is worth reiterating that these design recommendations should be employed with caution. Large deviations should be justified and validated. The final effective strength should not deviate more than 20%–30% from the commonly accepted 900 psi value (Heasley, 2007). This echoes the results of Su and Hasenfus (1997) who deduced that when considering weak roof and floor, the practical reduction in pillar strength may be upwards of 30%. This concept is not new to the ARMPS software, as initial versions allowed for deviations in design stability factor as a function of roof integrity.

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CONCLUSIONS

While empirical pillar design methods have created highly successful and efficient engineering design tools, the implementation can be improved. The determination of pillar strength by Mark-Bieniawski can be customized to represent site-specific conditions and improve classification. The use of numerical modeling proves that pillar strength can be highly variable, and sole dependence on geometric parameters, (W/H) ratio, may not be indicative of true strength.

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Barrier Pillar Design for Safe and Productive Longwall Mining

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ABSTRACT

Effective barrier pillar design is an essential component for safe and productive underground coal mining. Barrier pillar design methodologies that incorporate sound engineering judgment have been used for practical everyday usage. The next logical step is to utilize numerical modeling techniques that help combine practical experience and empirical observations into barrier pillar design analysis. A case study is presented that describes the barrier pillar design process, but also highlights the importance of considering the global mine response by evaluating: adjacent pillar stabilities, impact of intersection and entry widths, capacity of primary and secondary bolting systems and rib stabilization used for the safe and efficient removal of the longwall face equipment.

INTRODUCTION

Effective barrier pillar design is essential for safe and productive underground coal mining. The design of barrier pillars has spanned nearly 170 years. Serving as the primary abutments for loads generated by production panel mining, barrier pillars provide long-term protection for main, panel, and bleeder entries, adjacent production panel, and mine access and ventilation shafts. Barrier pillars are also utilized to isolate abandoned sections from active workings to simplify ventilation, provide independent escape routes, or retain large quantities of mine water – almost acting like a bulkhead. Finally, barrier pillars must be employed along the outcrop to protect surface features and maintain ventilation courses, and may be left at a property line to separate one mine from another in order to block the advance of an explosion, fire or water (Koehler and Tadolini, 1995).

Estimates indicate that from 5% to as much as 15% of recoverable reserves are lost to oversized barrier pillars in modern underground coal mines. Conversely, undersized barrier pillars can result in decreased safety because excessive load is transferred to adjacent openings which may also result in lost reserves if these areas become too unstable for mining (Koehler et al., 1989). Nowhere is this problem more intense than in the multiple-seam mine setting where seam interaction serves to further the redistribution of damaging high stress.

As with other mine support structures, optimum barrier pillar design is often attained through a process of trial and error. From simple observations of entry conditions to the more involved collection of entry closure and load redistribution data, in situ study of a baseline design can provide the information needed to successfully determine changes in pillar geometry that will result in improved performance. Simplified numerical modeling techniques are also provided to compliment empirical designs and measurements. A case study is presented that utilizes combinations of empirical design, underground observations and evaluations of intrinsic support densities that were successfully used to radically modify longwall panel “take-off” barrier pillars at the end of panel extraction.

Dunn’s Rule

The earliest known barrier pillar design method was proposed by Dunn in 1846. “Dunn’s rule” states that the width of a barrier pillar “should be 5 yards at 180 ft of depth, to be increased by 1 yard for each additional 60 ft of depth.” Dunn’s rule can be expressed mathematically by using the following equation:

$$W = \frac{(D - 180)}{20} + 15$$

where W = barrier pillar width, ft,

and D = depth of mining cover, ft.

For example, if the depth of mining is 800 ft then the barrier pillar would be 46 ft. Research and practice suggests that Dunn’s rule specifies significantly undersized pillar dimensions for a given depth of cover, especially for applications where thick, competent sandstone roof members promote high load transfer distances. In addition, Dunn’s rule does not account for varying seam thickness, nor does it consider coal strength, suggesting that Mr. Dunn developed this formulation from experience from a single seam. Dunn’s rule does not account for side loads applied by impounded water, thus rendering this formula invalid for the design of barrier pillars intended to act as retention dams.

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Not every barrier pillar design method is going to be presented in this report, but the authors thought it was important to cite the earliest known work. Even though experience and time have found the stated concerns, it is still important to recognize the earliest innovators in pillar design mechanics.

Old English Barrier Pillar Law

The 1928 “Coal Miner’s Pocketbook” presents an empirical formula for barrier design referred to in the text as the “mine inspectors’ formula for barrier pillars.” This design equation, also known as the “Old English barrier pillar law,” is cited in several publications by Ash and others (Ash, 1948; Ash, 1954; Ash, et al., 1949). The publications address barrier pillar design in anthracite coal seams. The original mathematical expression for the Old English law is presented below:

$$W = \frac{H \times T}{100} + 5 \times T$$

where H = hydrostatic head, or depth below drainage level, ft,

and T = coal seam thickness, ft.

Using the same depth or H as the example above of 800 ft and a coal seam thickness of 8 ft, the barrier pillar width would be 104 ft. An examination of the Old English barrier pillar law suggests that it is based on studies of barrier pillars utilized as impoundment features. This method should not be used for the design of a barrier pillar whose intended function is other than that of an impoundment dam because if no water pressure exists the hydrostatic head equals 0 ft and the equation specifies the same barrier pillar width (five times the coal seam thickness) for all mining depths. Also, like Dunn’s rule, the Old English formula does not provide for variations in coal strength, which underscores the need to exercise caution when applying this method.

Pennsylvania Mine Inspector’s Formula

In 1930, the State of Pennsylvania enacted legislation, amending an earlier act of 1911 for barrier pillar design that specified the required barrier size for mining coal in any seam. In his report on these new rules, Ashley stated “The Legislature of the Commonwealth of Pennsylvania at its last session passed a new act dealing with barrier pillars, which may have a wide interest in other states.” (Ashley, 1930) “The Pennsylvania mine inspector’s formula,” as it became widely known, states that “the minimum barrier pillar shall not be less than 20 feet, plus four times the thickness of the coal bed, plus ten feet for each 100 feet or fraction thereof of cover at the boundary in question.” The Pennsylvania mine inspector’s formula is expressed mathematically below by the following equation:

$$W = 20 + 4 \times T + 0.1 \times D$$

Where D = depth of mining cover, or height of hydrostatic head if greater than the thickness of overburden, rounded to the nearest 100 ft. If we use an 8 ft seam thickness and 800 ft of cover, the barrier pillar width, W , is 132 ft. An advantage of the mine inspector’s formula over Dunn’s rule and the Old English barrier pillar law is that it accounts for variations in both depth of mining cover and seam thickness. Also, this formula provides for the effects of water pressure so you could apply this approach to the design of water retention structures. As with Dunn’s rule and the Old English law, the mine inspector’s formula does not provide for variations in coal strength; however, this approach has resulted from experience of the Eastern coal fields of the United States, suggesting it is most applicable to coals having similar strengths.

Pressure Arch Method

Introduced in the 1950’s the “pressure arch method” is derived from empirical observations made in the coalfields of Northern England. Considerably more complex than the methods previously discussed, the pressure arch concept proposes that load is transferred across an opening of limited width, or across a series of openings separated by yield pillars, by a “pressure arch” that forms in the strata as shown in Figure 1. Because of pillar yielding, the area under the pressure arch is a zone of low stress, since most of the overburden load is transferred to the adjacent solid coal or barrier pillars. The width of the pressure arch increases proportionately with the width of the workings until the span across the workings becomes greater than the maximum pressure arch width that can be developed. In this situation, one stress abutment lies on the yielded pillars and the other lies on the unmined solid. Severe ground stability problems are likely because the yielded pillars cannot adequately support the additional abutment loads.

Field data indicates that the strata comprising the overburden influence the width of the pressure arch, whereas the minimum width of the maximum pressure arch is primarily a function of the overburden depth. It is from the minimum width of the maximum pressure arch that the minimum barrier pillar width and maximum allowable span can be calculated. The minimum width of the maximum pressure arch is represented by the following equation:

$$A = 3 \times \frac{D}{20} + 20$$

Where A = maximum width of maximum pressure arch, ft. In our continuing example, at a depth of 800 ft, $A = 180$ ft. It should be noted that this equation should not be used for overburden depths of less than 400 ft or greater than 2,800 ft.

Once the minimum width is determined, the proper panel and barrier widths may be calculated. The total width of the developed panel should not be wider than 75% of the minimum width of the maximum pressure arch. In our example, the total width of the panel, including entries and pillars should not exceed 180 times 0.75 or 135 ft. The minimum width of the adjacent barrier pillars must be equal to or greater than the average of the actual panel width and the minimum width of the pressure arch. This can be mathematically represented by the following equation:

$$W = \frac{D}{10} + 45$$

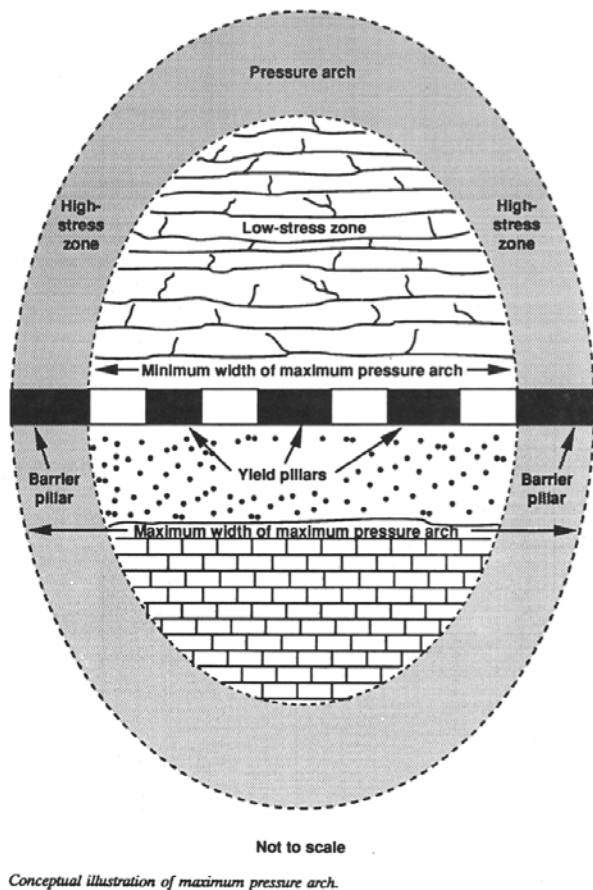


Figure 1. Pressure Arch Method.

$$W = \frac{S + A}{2}$$

Where W = the minimum barrier pillar width, ft,

and S = actual panel width, ft.

Because $S = 130$ and $A = 180$, W , the minimum barrier pillar width is 155 ft. This is just one example of the application of the pressure arch method. Barrier pillars intended to withstand the stress abutments from longwall mining can be, and are often, designed using this approach.

British Coal Rule of Thumb

The "British Coal rule of thumb," was reported by King and Whittaker in 1971, is a design formula developed and used successfully for years by British mine operators. The formula states that the width of a barrier pillar should be one-tenth of the overburden height plus 45 ft (King and Whittaker, 1971). A mathematical representation of the British Coal approach is:

For our example, the depth is 800 ft then the barrier pillar width, W , would be 125 ft. As with the Dunn's rule and Ash and Eaton impoundment formula, the British Coal rule of thumb does not account for variations in coal seam thickness. The beds of the British coalfields, however, rarely exceed 10 ft in thickness, suggesting that this value as an upper bound for valid application. The British Coal approach does not provide for variations in compressive strength, but typically the coal seams of Great Britain possess compressive strengths that are considered low by U.S. standards. Finally, no provision is made in the British formula for the effects of hydraulic head, making this method unsuitable for the design of underground water-retention dams.

Background Summary

The barrier pillar design methods presented here are empirically-based scaling laws developed entirely from in-mine observations and measurements of pillar performance. Additional methods that utilize convergence measurements, coal strengths and ground pressure measurements have all been used and reported in the literature (Holland and Gaddy, 1956; Holland, 1962; Borecki and Belinski, 1964; Holland 1965; Holland, 1973; and Koehler, Jones and DeMarco, 1989). The earlier methods did not benefit from the technological improvements in observational and measurement techniques available to today's ground control engineer, nor were the years of experience in barrier pillar research available to pioneers in this design field. Most of the design techniques do not account for variations in one or more mining parameter, such as coal strength or seam height, which renders their use contingent on a fairly high degree of engineering judgment and experience. It is also unreasonable to assume that any design approach can completely account for all site variables and variations in a specific mine. With these basic design guidelines, the experienced ground control engineer can apply any one of a number of rational design criteria to develop a barrier pillar size and configuration to satisfy the needs of a specific mining operation.

CASE STUDY - DESIGNING LONGWALL PANEL TAKE OFF BARRIER PILLARS

As previously described, barrier pillars are a critical component of longwall panel design, not just the barrier pillars placed between longwall regions to protect adjacent workings and seal off areas for ventilation systems, but also for the ends of the panels where long term mains, belt lines and structure are located. An opportunity to reduce the planned size of a longwall "take-off" barrier pillar was presented while examining general ground conditions and pillar performance. The original "pull-out" entries were designed with regard to an inclination in the coal seam and unfavorable geological conditions that had been previously encountered on adjacent panels. The mine is a multiple-seam longwall operation located in the Alma 2-3-4-6 coal seams. The original "take-off" lines are presented in Figure 2. As shown for the 8LT panel, the barrier pillar is approximately 15 cross-cuts or 1,500 ft from the main entries, which equates to about 375,000 tons of coal. A program was undertaken to analyze the barrier pillar size and select

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a “take-off” entry for safe and rapid extraction of the longwall face equipment.

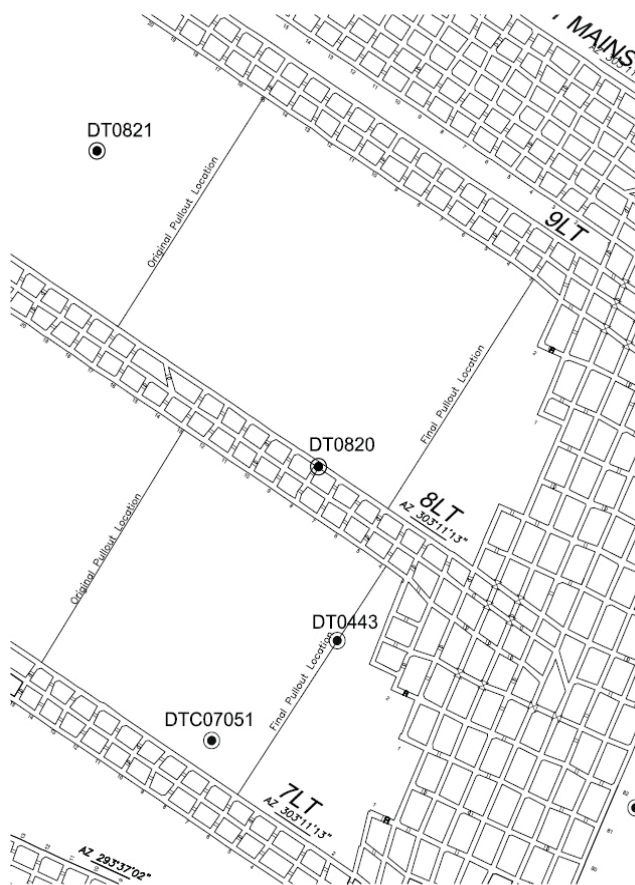


Figure 2. Original take-off lines for 8LT and 7LT longwall panels.

Underground Observations

A team was formed to examine the existing chain pillars, roof, and rib conditions. Borehole data was obtained and changes in geology were mapped and analyzed. The examination revealed that the entry in cross-cut #3 could be used as a possible extraction point. This zone was free of any adverse geological features, and with additional measures taken to reinforce the roof area and provide rib control technologies; it could serve as a potential location for the equipment extraction point. The depth of cover varied by almost 600 ft, ranging from 1,100 to 500 ft and was the deepest at the take off entries of the first site of the 8LT panel, shown in Figure 3.

Barrier Pillar Analysis

As discussed, there are several barrier pillar design methodologies that incorporate sound engineering principals and are easy to apply. A method that doesn't consider coal seam thickness (6 ft) but only overburden depth would be appropriate and there are no provisions required for a hydraulic head. We elected to apply the British Coal Rule of Thumb. British Coal approach is:

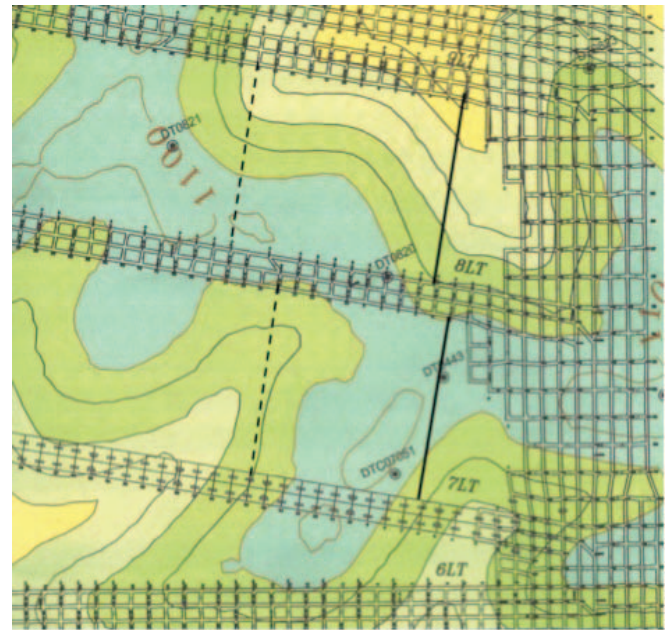


Figure 3. Overburden depth map of longwall areas and the new take-off lines for 8LT and 7LT panels.

$$W = \frac{D}{10} + 45$$

In the case study, the depth was 600 ft at the smallest part of the barrier pillar on the right side of the panel toward the 9LT entries. This resulted in a recommended barrier pillar size of 105 ft. If the stop line was at cross-cut #3, the distance or width of the barrier pillar would be 110 ft, which was wider than recommended. On the second panel (7LT) on the 8LT entries side, the barrier pillar becomes approximately 80 ft thick if the stop line is at cross cut #4 but “thickens” to about 130 ft at a distance of approximately 350 ft from the tailgate to the headgate. This is a large block coal that will protect the entries during the longwall equipment extraction process. If we apply the British Coal Rule of Thumb, the result is, $W = (1000/10) + 45$ or 145 ft. The barrier size determined from the British Coal Rule of Thumb indicated that the barrier pillar is 65 ft undersized. What appeared encouraging is that the pillars immediately behind this area will accept a large portion of the load and the entries are not necessary to keep open during the longwall extraction. Additional bolts and standing support could be used to bridge the load from panel onto the large pillars. The behavior of the barrier pillar on the first panel could be observed and monitored closely, and then the final design could be made and implemented well in advance of the longwall panel approaching. Figure 3 also shows the new “take-off” lines for the 8LT and 7LT panels and the overburden contours throughout the area.

Additional Data Analysis and Recommendations

While the barrier pillar can play a major role on the overall mine stability during the longwall equipment extraction, there are several

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other areas that need to be examined, analyzed and acted upon to have a safe and efficient extraction. The existing pillar designs, intersection spans, roof bolt density, supplementary support, and rib stabilization were all analyzed and presented to provide a completed picture.

Existing Pillar Design

While a considerable amount of time was spent assessing the barrier pillar design, the existing adjacent pillars developed for the mains and gate roads are also an important piece of the overall stability and performance of the barrier pillar system. An analysis of the pillar stability was reviewed using the Analysis of Longwall Pillar Stability (ALPS) software program (Mark, 1992). The ALPS method, which is embodied in the software program, consists of three basic steps:

- Estimate the loading that will be applied to the pillars during all phases of longwall mining,
- Estimate the load-bearing capacity of the longwall pillar system, and,
- Calculating “stability factors” (SF) by comparing the load and the load-bearing capacity.

For this case, the stability factors of the pillars were examined to ensure that excessive sloughage, the result of pillar loading, or instability would not apply additional loads to the intrinsic bolting system and onto the designed barrier pillar.

The generated gate roads were sized utilizing an 80 x 100 ft pillar (centers) and an entry width of 19 ft – again, the worst-case scenario for the longwall entries examined. The depth of cover varies by almost 600 ft, ranging from 500 to about 1,100 ft, particularly toward the 9LT development gates. The tailgate loading indicated that the calculated stability factor varies from 1.8 to below 0.5 when the depth is at its maximum or 1,200 ft. The overburden is interlaced with competent sandstone members that are relatively thick, providing arching and less load on some pillars but also higher on others. The pillars will also be adjacent to the barrier pillar that will help to ensure the pillar and entry stability. The key was to control the immediate roof to prevent it being driven to excessive deformations. The components of control will be increased bolt densities, standing supports, rib supports and surface control (mesh and mats).

Intersection Spans

An analysis of the intersection spans throughout the immediate area of the “take-off” entry revealed that the spans varied but were well within the design guidelines established for the barrier pillar design program. The suggested method for determining intersection span stability is:

$$\text{Span} = 20 + 0.26 (\text{CMRR})$$

The CMRR is the Coal Mine Roof Rating, established for the immediate roof area is 45. The CMRR is a function of the compressive strength of the intact rock, the spacing and persistence of discontinuities like bedding planes and slickensides, the cohesion and roughness of the discontinuities, and the presence of ground water and the moisture sensitivity of the rock (Molinda and Mark, 1994). An average acceptable span would be 31.7 ft.

The average span in areas approached this limit but was within the chosen guidelines.

Roof Bolt Density

The entries throughout the areas were evaluated for the actual bolt spacing's and capacities. Tension rebar (TR) bolts, 3/4-inch diameter, 5 and 8 ft long, were used throughout the entries. The largest variable encountered was the number of bolts used per row; 4, 5, and 6 bolts were used depending on the entry (travelway, belt, return). A 3/4-inch-diameter TR bolt has a capacity of 16.6 tons, and logically the more bolts you install, the more capacity you achieve. Based on an entry width of 19.5 ft, intersection span of 32 ft and a CMRR value of 45, a total capacity of 9.8 tons is recommended (Mark, Molinda and Dolinar 2001). When only a 5-ft-long bolt was used and installed 4 or 5 bolts per row, the recommended capacity fell slightly below this target at Analysis of Roof Bolt System (ARBS) values of 6.8 and 8.5. This prompted the initiation of a secondary bolting plan that required that two, Resin Assisted Tension Bolts (Double Lok), 7/8-inch diameter, 8 ft long or Cable bolts, .6 diameter (29 ton capacity), 10 ft long, be installed between the existing bolt rows throughout the entire take-off areas. That raised the Analysis of Roof Bolt Systems (ARBS) values above 19.0 for the entire zone. Figure 4 shows the supported zone after the additional supports were installed.



Figure 4. The supported longwall entry after additional supports were installed.

Rib Stabilization

The pillars, both inby and outby the “take-off” line, would be experiencing heavy loading due to the forward longwall abutments and an increased roof span at the designated “take-off” entry. The program detailed a rib support plan that included welded or geogrid mesh and roof bolts with additional surface control Spider Plates, that would provide confinement to the outer skin of the pillar and minimize sloughed material from falling into the access entries. The program was designed not only for rib stabilization but also to minimize tripping hazards and equipment barriers during the longwall equipment removal. Figure 5 shows an example of the rib support systems used throughout the recovery zone.



Figure 5. Example of the additional rib support systems used throughout the take-off areas.

Longwall Face Equipment Recovery

As anticipated, the gateroad pillars began to accept additional loading as the longwall panel was extracted. The side-abutment loads caused an increased yield zone on the outer edges which resulted in rib sloughage and increased the effective roof span. The additional supports installed to accept these larger loading conditions and maintain the failed rib material performed as designed. Additional surface control was also provided on the “critical” longwall face extraction entry that included woven mesh and additional surface control Spider Plates. Figure 6 shows the supports utilized in that critical entry. When the longwall face entered the take-off location, the equipment was successfully moved in 5 days and production was restarted. The barrier pillar performance was evaluated using observational techniques coupled with roof-to-floor closure measurements. Outby the barrier pillar, only minor loading was realized on the adjacent pillars. The project was deemed a complete success in that longwall operation was completed safely and an additional 375,000 tons of coal was extracted which greatly offset the cost of additional supports and labor.

NUMERICAL MODELING METHOD

As more confidence is established with numerical modeling techniques, it seems valid to combine these results with the existing empirical and observational techniques currently being used to evaluate barrier pillar stability. Comparative stress analysis was used to investigate the structural stability of the barrier pillars using the LaModel. LaModel was originally developed in 1993. It is a boundary element program that simulates a laminated overburden as a stack of frictionless plates (Heasley, 1998). Within the seam, the different coal and gob areas are represented by elements with various stress-strain behaviors. Using the stiffness behavior of the seam elements and the prescribed flexure of the overburden, the LaModel program calculates the displacements and loads throughout the modeled area of the seam.

A calibration method has been developed for the LaModel program that duplicates the abutment loading, coal strength, gob



Figure 6. The completed roof and rib support systems.

loading, and pillar strength used in ALPS and Analysis of Retreat Mining Pillar Stability (ARMPS). During the calibration, it is critical to note that these parameters are strongly interrelated, and because of the model geo-mechanics, the parameters need to be calibrated in the order of rock mass stiffness, gob stiffness, and coal strength. With this sequence of parameter calibration, the calibrated value of the subsequent parameters is determined by the chosen value of the previous parameters, and changing the value of any of the proceeding parameters will require re-calibration of the subsequent parameters.

Utilizing the mine layout and overburden, vertical stresses were determined for the developmental stage and after the longwall retreat to the projected take off location. The model input parameters contained lamination thickness and final gob modulus values suggested in the program based on the suggestions of Heasley et al., 2010. During the calibration of the lamination thickness, the highest overburden depth value of 1,100 was used. Because the overburden depth ranges from 1,100 to 500 ft in the area, the same analysis was performed using calibrated lamination thicknesses based on both 800 and 500 ft overburden depths. The analysis indicated that there were no significant changes in the results. Table 1 summarizes the numerical model input parameters.

The final dimensions of the barrier pillar are not uniform as can be seen as previously shown in Figure 2. This necessitated dividing the barrier pillar into 4 sections for analysis. The sections were numbered 1 to 4 from the headgate (9 LT) to the tailgate entry (8 LT). Obviously, the barrier pillar is the smallest where the 9 LT gate road entries were developed. The final width dimension designed for this side of the barrier pillar utilizing the empirical and observational techniques was 110 ft and then thickened slightly to 130 ft in the defined section. The thickest dimension was in section 3 where the barrier pillar was approximately 400 ft long perpendicular to the retreat longwall mining face. The section that will control the barrier pillar stability is the thinnest.

During the development stage, the vertical stresses developed on the gate road pillars was determined to be about 2,000 psi and the stress safety factors of the pillars are well above 2.0. In the high vertical stress regime, the vertical stress increases between 4,000 to

Table 1. Summary of the LaModel input parameters.

Overburden Depth	Variable, topographic stress analysis utilized
Mining Height	6 ft
Lamination Thickness	416 ft (calibrated for 1,100 ft overburden depth)
Final Gob Modulus 1	591,835 psi (panel width 1,000 ft)
Coal Modulus of Elasticity	300,000 psi
Rock Modulus of Elasticity	3,000,000 psi
In-situ Coal Strength	900 psi
Overburden Stress Gradient	1.1 psi/ft
Element Size	10 ft
Elastic perfectly plastic pillar elements used for the gateroad pillars, main pillars and barrier pillars	

6,500 psi and the stress safety factor of the pillars is between 2.8 to 4.0 on section 4. If the barrier pillar is narrowed 20 additional feet to a head gate dimension of 90 ft, the barrier pillar safety factor in section 4 falls between 1.4 and 2.1. Values below 2 would require additional scrutiny and be a signal that advises caution. Extracted barrier pillar coal is an added bonus, but would not be considered if the stability of the area compromised the safety of personnel, increased time of equipment removal, or imposed difficulties that necessitated leaving equipment in place. Figure 7 shows the final stress safety factors if the longwall retreats to the fictional take-off location of 90 ft. It should also be noted that it provides more flexibility to move equipment with a cross cut, but the scope of this paper is not intended to write a book on operations.

SUMMARY

Effective barrier pillar design is an essential component for safe and productive underground coal mining. Barrier pillar design methodologies date as far back as 1846 when John Dunn proposed a barrier pillar design methodology based on his observations and practical experience, realizing how critical these mine design elements were for safe and productive coal extraction. Several other empirical design methods have since followed that incorporate sound engineering judgment based on experience, instrumentation measurements and “calibration” of observed pillar response. This paper suggests that these methods, coupled with numerically modeling, can provide an understanding of the selected design and permit “what ifs” without increasing the calculated risk. Barrier pillar coal extraction provides additional tons of mined coal, but the risk needs to be evaluated and understood.

The case study also emphasizes that the complete stability of the selected location needs to be considered. The existing pillar designs, intersection spans, roof bolt density, supplementary support, and rib stabilization should be evaluated and measures taken to ensure stability. Support densities were increased and secondary supports were added to stabilize and minimize pillar sloughage. The project culminated in an additional 375,000 tons of coal being removed from the barrier pillar and a longwall equipment move (coal-to-coal) of 5 days. There are no ground control designs that can be safely incorporated or be successful

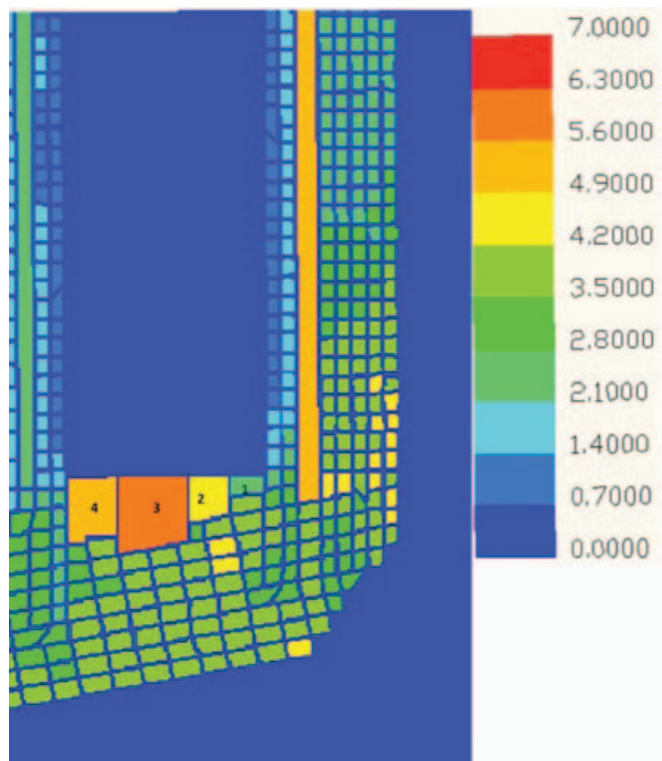


Figure 7. Final stress safety factors after face retreated to a 90 ft thick barrier pillar.

without the expertise and professional approach of a well-trained operational team.

ACKNOWLEDGEMENTS

The authors would like to thank the Management and Operational Staffs at Arch Coal for supporting this barrier pillar program and Dr. Ihsan B. Tulu, West Virginia University for assisting with the numerical modeling analysis.

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The Potential of Using Coal Washing Plant Waste As a Backfill in Room and Pillar Mines

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ABSTRACT

Backfilling using coal waste is not a novel means for waste disposal throughout the world; yet, backfilling using a high density (paste) backfill is relatively new, except in Germany and Poland. One of the main benefits of a high density backfill, as opposed to a low density (slurry) backfill, is that, if placed at a low permeability and low moisture content, the risk of water table contamination can be relatively low. In addition, if paste backfill can be placed cost effectively, environmental benefits are huge because the surface impoundment footprint can be greatly reduced or even eliminated. This is especially true in room-and-pillar applications. In its current state, backfilling is considerably more costly than surface disposal. This is mainly because paste, as opposed to slurry, significantly increases backfill placement costs; however, this could be offset by increasing the achievable extraction rate. Overall, such a system could potentially improve the public's image of coal mining in the US for a myriad of reasons. This paper discusses a proposed backfill system for a typical room-and-pillar coal mine in the Illinois Basin of the US and clearly shows the technical advantages, although costs are still higher with fill. The cost differential will likely reduce over time, in favor of the backfill option, as costs of traditional surface disposal increase, due mainly to ever-restrictive disposal regulations.

INTRODUCTION

Backfilling is the process whereby waste material (generally metallurgical tailings or prepared aggregates) is hydraulically placed back underground into open mine workings. Backfill is typically used for the following mining applications:

- A working platform
- An artificial roof
- A stope filler
- An artificial sidewall

This has many potential advantages for typical underground mining applications, such as:

- Reduced tailings disposal on the surface (i.e. impoundments/tailings dams)
- Reduced surface subsidence

- Increased mine extraction
- Reduced rockburst (seismic) damage
- Improved support for mining excavations and/or working surfaces
- Increased worker safety
- Improved control of stresses

Typical sources of backfill material include:

- Waste metallurgical tailings that can be used in total or as classified material
- Underground waste rock
- Sand or quarried rock
- Industrial waste (such as fly ash)

Most coal mines in the US use conventional surface impoundments to dispose of washing plant waste. Recently the US Environmental Protection Agency (EPA) began implementing more stringent rules for this disposal, which will increase conventional disposal costs. In addition, the environmental lobby continues to oppose the granting of new or modified impoundment permits. These largely political actions make the disposal of waste products underground an increasingly viable option in the future.

One of the overlying issues with underground disposal is the potential contamination of the water table in the long-term, mainly from water runoff through the placed backfill (Earthjustice, 2009). To mitigate this, placing backfill with a low permeability coefficient (as yet to be determined) may be the solution; however, this can only be achieved by placing backfill at a high density, low porosity, and possibly with a cement addition. Fortunately, the effective placement of high density backfill has already been achieved. Longwall coal mining with high density backfill has been common in Germany and Poland for several decades. There, its main purposes are to reduce surface subsidence and help control stresses. Likewise, Spearing et al. (2011) established that high density backfill could be made from coal wash plant waste and transported underground as required. From their study the following conclusions were drawn:

- It is technically feasible to pump high density coal waste underground as opposed to the current practice of using low density slurry with all the inherent disadvantages mainly

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associated with free water. Admixtures to reduce pump pressures and reduce the beach angle are essential.

- The high density backfill option would be significantly more costly than traditional impoundment disposal; however, there is considerable potential to reduce the high density backfill costs significantly.
- Any candidate backfill material needs to be tested to obtain the rheology in pipe loops. Slump tests can be used as an initial indicator, but slump tests alone are not recommended.

BACKFILL USAGE AND AVAILABILITY IN ROOM AND PILLAR COAL MINES

Beside the aforementioned advantages, high density backfill could also eliminate the need for explosion-proof seals and reduce ventilation losses in room-and-pillar coal mines. The keys would be placing the backfill material at low moisture content and low permeability so that the risk of subsequent water table contamination is low.

Typically, about 40-50% of run-of-mine coal is removed as waste in a coal washing plant by weight. This is the available material for backfilling. Depending on the specific mine location; this material availability could be augmented using other suitable locally available waste products, such as thermal power plant waste (typically top and bottom ash). Thus, backfill could be placed in the following configurations:

- About 45-55% backfill in each panel
- About 90-100% backfill in every second panel

The increase in backfill percentage placement over the reject coal waste percentage is due to the bulking of material, assumed to be 30% after it is mined.

ROOM AND PILLAR EXTRACTION

In the Illinois Basin, most of the coal is underlain by a weak layer of strata in the immediate floor termed “fireclay” or “claystone”. This tends to cause pillar punching, where the floor is unable to sustain pillar loads and, in effect, pillars sink (punch) into the floor. Additionally, this can lead to increased rib instability and entry closure in the short-term and increased surface subsidence in the long-term (Gadde, 2009).

Traditionally, the most basic closed form solution of a pillar’s stability can be achieved through the pillar Factor of Safety (FOS) calculation, where pillar stress is estimated using the Tributary Area Theory and pillar strength is estimated using an empirically based formula (e.g. Obert-Duvall/Wang; Holland-Gaddy; Holland; Salamon-Munro; or Bieniawski). In coal mines with rigid floors, FOS approximations are sufficient for pillar stability analysis; however, where the mine floor is exceedingly weak, as in the Illinois Basin, FOS calculations may not be as appropriate because they do not adequately account for weak strata below (or above) pillars.

Various approaches have been used in the past to handle weak floors, the most current and successful to date being the Vesic-Gadde formula (Gadde, 2012), which was adapted from the Vesic formula (1975). This formula calculates the Floor Safety Factor (FSF), which is used in place of the FOS.

COMPARISON OF FOS & FSF

As part of a research project at Southern Illinois University Carbondale, three mines in the Illinois Basin were assessed to determine their actual operating FOS and FSF based upon geology and stress conditions. Mine A in the Illinois Coal Basin uses 40x40-foot in-panel pillars and 20-foot rooms. Using Tributary Area Theory formula, the LaModel computer modeling program (Heasley, 1998), and the Vesic-Gadde formula (Gadde, 2009), FSF and FOS were determined as shown in Table 1. It is obvious that the effect of weak floor is dramatic when designing pillars under such conditions. As the FSF suggests, Mine A is operating at around 50% lower stability than indicated by a traditional FOS calculation based on Tributary Area Theory and the Bieniawski formula.

Table 1. FSF and FOS for Mine A.

Parameter	40’x40’ pillars	35’x35’ pillars	30’x30’ pillars	25’x25’ pillars
FSF	2.51	2.20	1.88	1.55
FOS	5.45	4.50	3.55	2.71
FSF/FOS	0.46	0.49	0.53	0.57

FSF values greater than 1.5 indicate that the pillar design will be safe in the long term (Gadde, 2012). This figure is, however, only acceptable if the immediate floor is not adversely affected by water in the long term, which tends to happen over time (abandoned mines typically become flooded). Exposure to moisture further weakens clay floors implying that surface subsidence is likely to occur over time. The placement of high density backfill could therefore aid in reducing these long-term effects for obvious reasons.

Regarding short-term stability, an FSF of 1.2 is currently desired. While the “short-term” definition may be somewhat subjective, it is generally considered great enough to allow for a single panel to be finished before floor instability becomes an issue (Gadde, 2012). Assuming backfilling starts on each panel immediately upon completion, minimum pillar sizes could be used (such that the FSF equals 1.2) without sacrificing short-term panel closure and surface subsidence. However, there appears to be two examples of Illinois Basin mines that indicate that the “minimum” safe FSF value may actually need to be higher. In both cases, pillar sizes yielding FSFs greater than 1.2 were associated with near-immediate pillar sloughing. If this pillar sloughing was due to tensile failure caused by floor displacement, the 1.2 FSF minimum needs to be reconsidered. These mines are currently being modeled and analyzed, but information is scant and they are sealed and probably flooded.

BACKFILL DISTRIBUTION

The key factor associated with distribution complexity and cost is the beach angle that can be achieved with backfill as this directly influences the number of discharge points. The beach angle is the angle at which backfill material naturally comes to rest after placement. A coal mine in Australia has produced a high density

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backfill material for a longwall application with a beach angle of between 4° and 6° (Tarrant et al, 2012).

The following analysis assumes annual mining of two typical room-and-pillar panels each with eleven entries, 60-foot pillar centers, and 10,000-foot length. The extraction ratio would be 55.6% and each panel would produce about 847,000 run-of-mine tons. Assuming 60% washing plant recovery, the two panels would yield about one million tons of clean coal in one year.

The piping quantity and cost can be estimated based on the backfill beach angle. Examples of potential distribution systems are shown in Figures 1 and 2 with required piping shown as blue lines. For each system, a beach angle of 5° was assumed at the 6 ft seam height. Empirically, this yields an effective distribution area of 140-foot diameter for each spigot. These effective fill areas are shown as blue circles.

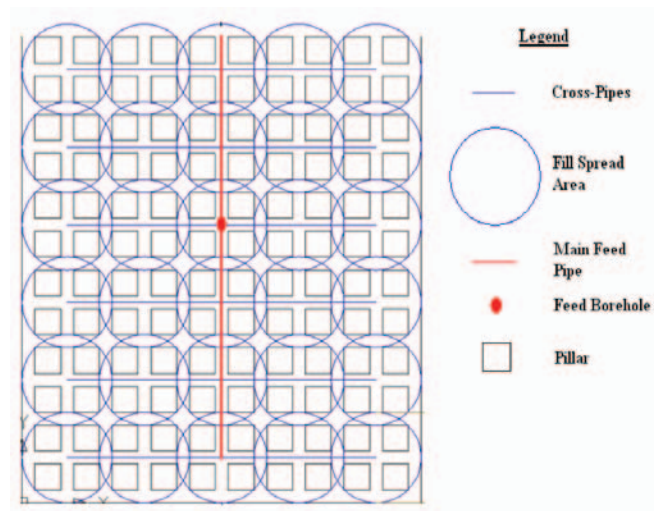


Figure 1. Herring-bone pipe schematic.

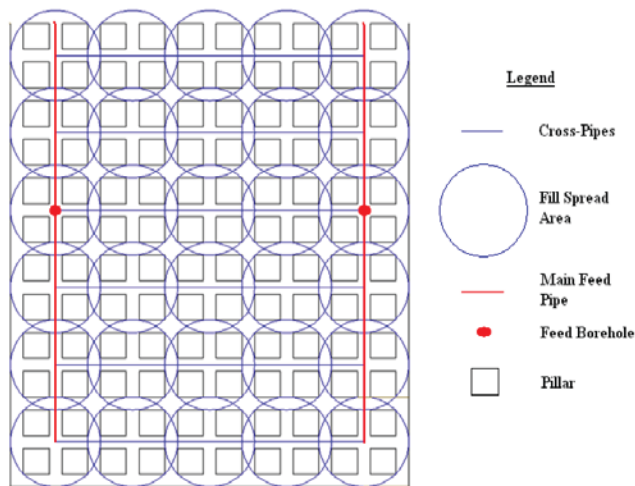


Figure 2. Ladder-type pipe schematic.

Ideally, backfill material is pumped overland to boreholes above each panel to be filled. To minimize the risk of blockage

anywhere in the system, it is recommended that two boreholes be used to provide redundancy. In the herring-bone schematic shown in Figure 1, boreholes would connect to a single main feed pipe running the entire length of the panel in the center of the panel. In the ladder-type schematic shown in Figure 2, boreholes would connect to two main feed pipes, one on either edge of the panel.

Tables 2 to 7 show cost relationships with the following important variables regarding fill distribution to mined-out panels: piping schematic, pipe diameter, fill ratio, and beach angle. The costs include all the piping except for the vertical column assumed to be in a borehole above the mined out panel. Each piping schematic (herring-bone and ladder-type) has its own minimum required pipe length. In addition to this minimum pipe length, each beach angle has its own length requirement due to the differing extents of fill spread associated with each beach angle. The lower the beach angle, the less amount of pipe length required. A 75% steel and 25% plastic combination was assumed for distribution network cost models. Clearly plastic piping is easier to handle and install so in practice, pressure permitting it's usage would be maximized. Schedule 40 A120 carbon steel and Schedule 80 PVC piping were chosen because of their high-pressure capacities. Four different pipe diameters were chosen and then matched with each beach angle's required pipe length to determine the total cost of piping per panel. Tables 2 and 3 show steel piping costs for both designs schematics, as well as maximum working pressures for which each pipe diameter is rated. Tables 4 and 5 show the plastic (PVC) piping costs for each design schematic; with the maximum working pressure for each pipe diameter. Tables 6 and 7 show the total cost for the 75:25 piping mixture for herring-bone and ladder-type schematics, respectively. Total material cost figures and pipe properties for both pipe types and their diameters were obtained from InfoMine's 2012 Coal Cost Guide (Leinart, 2012).

Backfill rate is dependent mainly on pipe diameter, and the following flow rates could be estimated from each typical pipe nominal diameter based as an example on spiral waste (Spearing et al., 2011). Clearly, specific rheology tests are needed for all candidate fill mixes before accurate design of the distribution system can be undertaken. Table 8 estimates flow rates in different pipe diameters based on rheology trials conducted by Spearing et al. (2011).

Assuming each panel would be filled about 45% (by volume), the time required to fill two panels mined each year as a function of flow rate is also given in Table 8. This assumes 20 hours of filling per day and a maximum of 300 days of backfilling per year. It is obvious that only 5" or 6" diameter pipe could be used to fill both panels in 300 available backfill days.

The cheaper of the two delivery options is, in every case, the herring-bone system, all other variables (including beach angle, flow rate, pipe diameter) being equal. The increased cost of the ladder-type schematic is due to its second length-wise main feed pipe. Because both options use the same amount of cross-panel piping, if the cost of the second main feed pipe used in the ladder-type schematic can be tolerated, the long-term benefits of having two feeder pipes may outweigh the initial cost. The primary long-term benefit of the ladder-type schematic is that the two main feed pipes ensure that the ladder-type schematic can still deliver fill to mined-out workings if a blockage occurs in one of the main feeders. The herring-bone system does not facilitate continued

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Table 2. Approximate steel piping costs for herring-bone fill of two panels per year.

Herring-Bone	Schedule 40 A120 Steel	2"d, 400 psi	4"d, 320 psi	5"d 290 psi	6"d, 280 psi
50% Fill Ratio		\$4.90/ft	\$12.93/ft	\$14.61/ft	\$20.75/ft
Beach Angle	Pipe Length (ft)	Cost (\$)	Cost (\$)	Cost (\$)	Cost (\$)
2°	35741	\$131,349	\$346,600	\$391,634	\$556,222
3°	53926	\$198,177	\$522,945	\$590,891	\$839,219
4°	72086	\$264,916	\$699,055	\$789,883	\$1,121,840
6°	108494	\$398,714	\$1,052,116	\$1,188,818	\$1,688,431
8°	145140	\$533,390	\$1,407,497	\$1,590,373	\$2,258,744
12°	219585	\$806,976	\$2,129,429	\$2,406,106	\$3,417,297

Table 3. Approximate steel piping costs for ladder-type fill of two panels per year.

Ladder-Type	Schedule 40 A120 Steel	2"d, 400 psi	4"d, 320 psi	5"d 290 psi	6"d, 280 psi
50% Fill Ratio		\$4.90/ft	\$12.93/ft	\$14.61/ft	\$20.75/ft
Beach Angle	Pipe Length (ft)	Cost (\$)	Cost (\$)	Cost (\$)	Cost (\$)
2°	55398	\$203,586	\$537,217	\$607,018	\$862,124
3°	73697	\$270,836	\$714,674	\$807,532	\$1,146,906
4°	91914	\$337,786	\$891,341	\$1,007,153	\$1,430,419
6°	128379	\$471,794	\$1,244,959	\$1,406,717	\$1,997,904
8°	165055	\$606,576	\$1,600,619	\$1,808,588	\$2,568,665
12°	239529	\$880,269	\$2,322,831	\$2,624,638	\$3,727,668

Table 4. Approximate PVC piping costs for herring-bone fill of two panels per year.

Herring-Bone	Schedule 80 PVC	2"d, 500 psi	4"d, 510 psi	5"d 460 psi	6"d, 430 psi
50% Fill Ratio		\$4.45/ft	\$13.14/ft	\$20.56/ft	\$25.34/ft
Beach Angle	Pipe Length (ft)	Cost (\$)	Cost (\$)	Cost (\$)	Cost (\$)
2°	35741	\$39,762	\$117,410	\$183,710	\$226,420
3°	53926	\$59,992	\$177,146	\$277,178	\$341,620
4°	72086	\$80,196	\$236,803	\$370,523	\$456,665
6°	108494	\$120,699	\$356,401	\$557,657	\$687,307
8°	145140	\$161,468	\$476,785	\$746,020	\$919,463
12°	219585	\$244,289	\$721,338	\$1,128,669	\$1,391,073

Table 5. Approximate PVC piping costs for ladder-type fill of two panels per year.

Ladder-Type	Schedule 80 PVC	2"d, 400 psi	4"d, 320 psi	5"d 290 psi	6"d, 280 psi
50% Fill Ratio		\$4.45/ft	\$13.14/ft	\$20.56/ft	\$25.34/ft
Beach Angle	Pipe Length (ft)	Cost (\$)	Cost (\$)	Cost (\$)	Cost (\$)
2°	55398	\$61,630	\$181,981	\$284,743	\$350,943
3°	73697	\$81,988	\$242,094	\$378,801	\$466,869
4°	91914	\$102,255	\$301,939	\$472,440	\$582,278
6°	128379	\$142,822	\$421,726	\$659,870	\$813,283
8°	165055	\$183,623	\$542,205	\$848,381	\$1,045,622
12°	239529	\$266,476	\$786,852	\$1,231,178	\$1,517,415

delivery in such an event. On the other hand, the herring-bone design allows for more immediate filling of the center of the panel,

which in essence means that the effective width of the panel is more immediately reduced. The ladder-type system would fill near

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Table 6. Total cost of 75:25 steel-to-plastic piping for herring-bone fill of two panels per year.

Herring-Bone 50% Fill Ratio	75:25 steel-to-PVC	2" diameter	4" diameter	5" diameter	6" diameter
Beach Angle	Pipe Length (ft)	Cost (\$)	Cost (\$)	Cost (\$)	Cost (\$)
2°	35741	\$171,111	\$464,010	\$575,343	\$782,642
3°	53926	\$258,169	\$700,091	\$868,069	\$1,180,839
4°	72086	\$345,112	\$935,858	\$1,160,406	\$1,578,505
6°	108494	\$519,413	\$1,408,517	\$1,746,475	\$2,375,737
8°	145140	\$694,858	\$1,884,282	\$2,336,393	\$3,178,206
12°	219585	\$1,051,265	\$2,850,766	\$3,534,775	\$4,808,370

Table 7. Total cost of 75:25 steel-to-plastic piping for ladder-type fill of two panels per year.

Ladder-type 50% Fill Ratio	75:25 steel-to-PVC	2" diameter	4" diameter	5" diameter	6" diameter
Beach Angle	Pipe Length (ft)	Cost (\$)	Cost (\$)	Cost (\$)	Cost (\$)
2°	35741	\$265,216	\$719,198	\$891,762	\$1,213,067
3°	53926	\$352,823	\$956,768	\$1,186,334	\$1,613,775
4°	72086	\$440,041	\$1,193,280	\$1,479,593	\$2,012,698
6°	108494	\$614,616	\$1,666,685	\$2,066,587	\$2,811,187
8°	145140	\$790,200	\$2,142,823	\$2,656,969	\$3,614,287
12°	219585	\$1,146,744	\$3,109,684	\$3,855,816	\$5,245,083

Table 8. Flow rate and backfill time as a function of pipe diameter.

Nominal pipe diameter	2" diameter	3" diameter	4" diameter	5" diameter	6" diameter
Flow Rate (yd ³ /hr)	21	45	80	125	180
Backfill Time @ 45% fill (days)	1,638	764	430	275	191

the panel edges first, which would in effect increase the effective barrier pillar width, but would more slowly reduce the effective width of the panel.

COSTS

To illustrate the effect of improved recovery due to backfilling, simple Net Present Values (NPVs) were calculated. The general assumption is that improvements in recovery would manifest only in increased mine life, because coal production would remain constant. Two scenarios were compared – one using current non-backfilled operations versus one using backfilling that results in a 2.5% incremental increase in total mine life due to increased extraction. These NPVs were then used to calculate a total breakeven cost per ton of clean coal produced for each incremental increase in recovery seeking to find the point for backfilling to be cost neutral over life of the mine.

Assumptions made were:

- Conventional mine life (i.e. no backfill) of 20 years
- Total clean coal tonnage of 1 million/year

- 2 panels mined per year
- Cost of clean coal (including capital costs) = \$36/ton
- Sale price of clean coal = \$50/ton
- Cost of blast-proof seals per year = \$250,000
- Surface disposal capital cost of \$900,000 (Spearing et al., 2011)
- 5 degree beach angle with a 5 inch nominal bore pipe
- Annual surface disposal operating cost of \$2,490,000 (Spearing et al., 2011)
- Rate of return = 15% (assumed high rate of return based on industry practice)

Table 9 and Figure 3 illustrate the breakeven cost per clean ton of backfill for various options. Table 9 shows three different costing scenarios. The first two columns assume no cost benefit to using backfill (i.e. surface disposal and panel sealing costs remain constant). The next two columns remove the cost of blast-proof seals from total cash flow. The last two columns remove both blast-proof seals and surface disposal costs from total cash flow. It should be noted that some limited surface waste storage would still be needed on a temporary basis even if backfilling is employed.

Table 9. The cost of backfill as a function of increased recovery.

Assumes no cost benefits		Includes elimination of blast-proof seals		Includes elimination of blast-proof seals and surface disposal	
Recovery Increase (%)	Breakeven Cost of Fill (\$/ton)	Recovery Increase (%)	Breakeven Cost of Fill (\$/ton)	Recovery Increase (%)	Breakeven Cost of Fill (\$/ton)
0	0.00	0	0.00	0	0.00
2.5	0.06	2.5	0.31	2.5	2.94
5	0.12	5	0.37	5	3.00
7.5	0.17	7.5	0.42	7.5	3.05
10	0.22	10	0.47	10	3.10
12.5	0.26	12.5	0.51	12.5	3.14
15	0.31	15	0.56	15	3.19
17.5	0.34	17.5	0.59	17.5	3.22
20	0.38	20	0.63	20	3.26
22.5	0.41	22.5	0.66	22.5	3.29
25	0.44	25	0.69	25	3.32

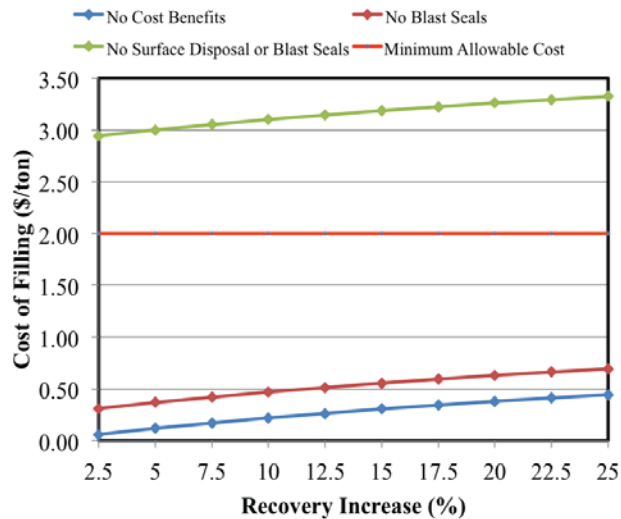


Figure 3. Progression of cost of filling vs. recovery increase for different backfilling scenarios.

Typical costs of paste backfill per ton for hard rock mines vary between \$2 and \$6 (Spearing *et al.*, 2010). In essence, this means that for a paste backfill regimen to be economically feasible, the breakeven cost must be at least \$2 per ton. From Table 9, it can be seen that improved recovery alone is incapable of justifying a paste backfill regimen. Even if blast-proof seals were eliminated, the breakeven cost per ton of fill does not meet the paste cost range based on industry experience.

Another potential productivity gain could be realized if continuous miners could mine complete crosscuts in one pass. Assuming a maximum allowable mining depth of 40 feet, continuous miners could hole through in one advance if pillar dimensions were reduced below 40 feet. In some cases, the potential exists for pillar dimensions to be reduced to 30x30 feet. Should this become possible, two continuous miner advances could produce two pillars and one complete room (30'+30'+20'=80').

The productivity gain would be a result of decreases in the amount of time needed for continuous miners to change places. The logistics and magnitude of this benefit are not considered in this paper.

THEORETICAL EXAMPLE

Referring to Table 1, the following example compares a conventional mine with 40x40-foot pillars (FSF of 2.17, FOS of 5.63) to a mine using 25x25-foot pillars and backfill (with a short-term FSF of 1.34). Mine panel depth has been set at 250 feet and mining height at 6 feet. All of the same assumptions used in the previous costing discussion carry over to this example, and the breakeven cost for each of the three increased extraction scenarios is calculated. Backfill cost estimates are given in Table 10. The decrease in pillar size yielded an extraction increase of 13.58%.

Table 10. Backfill operating cost estimate per clean ton mined.

Cost Element	Cost Estimate (\$)
Labor (4)	200,000
Maintenance	100,000
Admixture	1,000,000
Cement (1.5%)	600,000
Piping	1,200,000
Power	200,000
TOTAL ANNUAL OPERATING COST	\$3,300,000
COST/CLEAN COAL TON	\$3.30

The total annual excavated volume of this scenario is calculated to be 39 million cubic feet. Assuming 60% recovery of run-of-mine coal and in-situ coal weight density of 80 lbs/ft³, the annual clean coal tonnage would be roughly 1 million, leaving roughly 600,000 tons of coal waste. If the weight density of paste backfill

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is considered to be 80 lbs/cu ft, the total volume of waste that could be returned to the mine workings as backfill would be roughly 45% of the initial excavated volume (17.5 million cubic feet). The resulting backfill tonnage is thus calculated to be roughly 600,000.

Spearing et al. (2011) estimated paste fill capital costs to be about \$7.0 million. For simplicity assuming a ten year useful plant life, the capital cost per clean ton mined for the paste plant would be \$0.70. Table 10 provides a breakdown of estimated paste fill operating costs per clean ton mined.

Total estimated capital (\$0.70/ton) and operating (\$3.30/ton) costs per clean ton mined in this example are therefore:

\$4.00/CLEAN TON MINED

Ignoring any potential productivity benefits, the current breakeven paste fill cost at 13.58% extraction increase is \$3.14 per clean ton mined as shown in Table 11, which is less than the actual estimated cost of \$4.00 per clean ton mined.

Table 11. Breakeven costs for decreasing pillar size from 40x40-feet to 25x25-feet.

Hypothetical Mine	
Current NPV	\$87,630,641
Current Mine Life	20 years
Extraction Increase (%)	13.58
New Mine Life	22.716 years
No Cost Benefits	
New NPV	\$89,423,987
NPV Difference	\$1,793,346
Breakeven Cost	\$0.28
No Blast-Proof Seals	
New NPV	\$91,020,844
NPV Difference	\$3,390,203
Breakeven Cost	\$0.53
No Blast-Proof Seals or Surface Disposal Costs	
New NPV	\$107,825,539
NPV Difference	\$20,194,898
Breakeven Cost	\$3.14

CONCLUSIONS

Based on research to date and ignoring potential productivity increases, paste backfilling would increase current operating costs by \$0.86 per clean ton mined. Whilst this is unacceptable currently, it indicates that further research is needed to specifically investigate any productivity increase potential, and to update costs based on tightening surface disposal regulations and the adverse public perception concerning surface tailing disposal sites. In the medium term, it is likely that backfilling will become a cost effective option.

Areas that require further study are as follows:

- Permeability of fill material required to protect the water table from post-filling water egress.
- The “safe” short-term pillar FOS and FSF (for Illinois Basin specifically).
- The risk of failure due to blockage of the distribution system and development of a more reliable system and cost estimate.

ACKNOWLEDGEMENTS

This work was made possible by funding from the Illinois Department of Commerce and Economic Opportunity through the Illinois Clean Coal Institute. The authors greatly appreciate the cooperation of three Illinois Basin coal mines that participated in the study.

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Probabilistic Approach to Coal Pillar Design

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ABSTRACT

The classical approach to engineering design involves the purely deterministic calculation of the factor of safety, the ratio of strength to stress. The factor of safety is an easy calculation to perform in many circumstances, and it is very easy to interpret the results. The simplicity of the method is both a strength and a weakness. The results of the deterministic factor of safety calculation are simple to determine and interpret, but they have no ability to account for the uncertainty present in the inputs or outputs.

Uncertainty is prevalent in geotechnical engineering applications. This uncertainty can stem from spatial variability, poor sampling, a dynamically changing environment, etc. Probability analyses allow input parameters to take the form of a distribution rather than a single value, making them uniquely equipped to handle the uncertainty inherent in geotechnical engineering design considerations. A stochastic approach also has the benefit of providing results as a probability of failure, which is more meaningful than a factor of safety.

Though stochastic modeling is becoming more common in many geotechnical applications, it is still not widely used for coal pillar design. The objectives of this study are to justify the concept of using a stochastic approach to coal pillar design and to compare the results of a simple stochastic analysis to those of standard industry practice.

A simple synthetic study was performed to compare the results of a probabilistic analysis of coal pillar design to the results obtained from running ARMPS, which is published by NIOSH. Normal distributions were assigned to each input parameter that is required to determine the factor of safety of a single coal pillar. The probability of failure resulting from the synthetic study was found to be more conservative than the ARMPS stability factor.

A probabilistic approach to coal pillar design is discussed in comparison to the traditional deterministic approach. The greatest strengths of the probabilistic approach were determined to be the effective handling of uncertainty and the increased meaning of the inputs and outputs inherent to a probabilistic approach.

INTRODUCTION

The method of engineering design is the process of choosing the appropriate components to make a functional structure that is sufficiently safe. Engineering design typically involves an iterative, purely deterministic approach where a multitude of inputs are adjusted until the optimal design is reached. This deterministic approach to assess safety and durability generally results in a factor of safety (FS) value, as calculated by

$$FS = \frac{\text{STRENGTH}}{\text{STRESS}}$$

The factor of safety is a very easy calculation to perform in many circumstances, and it is very easy to interpret the results. If the strength of a component or a structure is greater than the stress expected to be applied to the component or structure, then the factor of safety will be greater than 1 and the component or structure is not expected to fail. However, if the stress exerted on the component or structure exceeds the strength, then the factor of safety will be less than 1 and the component or structure is expected to fail.

For the deterministic approach to be used, it must be assumed that the exact inputs are known. If the exact inputs are known, then the exact solution can be determined. The deterministic approach is a very simple approach to engineering design. The greatest strength of a deterministic factor of safety calculation is also its greatest weakness: it is extremely simple. Because of its simplicity, the deterministic approach to factor of safety calculations is completely lacking in uncertainty quantification.

Uncertainty quantification is an important practice in all engineering practices. This is especially true when regarding geotechnical engineering. A high level of uncertainty is involved with engineering geotechnical structures. A common method of skirting the issue of uncertainty quantification is to take a conservative approach to design. The conservative approach involves underestimation of the strengths and/or overestimation of the stresses in an attempt to guarantee a safe design. Using a

conservative approach for engineering design does not guarantee safety, can lead to overdesign of components and structures, and removes meaning from the term factor of safety by introducing intentionally arbitrary inputs (Ashley, 1981).

This conservative approach has led to design standards that require factors of safety for elevators to be as high as 11. Elevators must be overdesigned to some degree because of the likelihood of misuse and the dire consequences of failure, but many engineering structures do not require the overdesign that is typically present. Intentionally designing to a factor of safety much greater than 1 indirectly admits the fault in the deterministic approach while bypassing more meaningful and less arbitrary alternatives (Lundquist and Heins, 1972).

PROBABILISTIC APPROACH

A probabilistic approach to engineering design can be a superior alternative to a deterministic approach. Instead of considering one value for each input in an engineering design calculation, a probabilistic approach allows for a distribution of values for each input parameter. A distribution is used rather than a single value to represent the uncertainty or variability in the data. With a distribution of inputs, the output will also take the form of a distribution. This distribution of outputs leads to the greatest strength of the probabilistic analysis, which is a meaningful result.

Using a probabilistic approach for calculating factors of safety can easily be visualized in a few different ways. Figure 1 shows possible distributions of stress and strength for an example probabilistic analysis. The probability of failure in the example component or structure is the area confined in the overlap where the stress exceeds the strength. Figure 2 shows a second possible visualization of results from a probabilistic analysis. With the factor of safety output plotted as a single distribution, the probability of failure is simply the area under the curve to the left of a factor of safety equal to 1. This is a simplified version of possible results from stochastic analyses, but a more complex analysis will likely take the same general form.

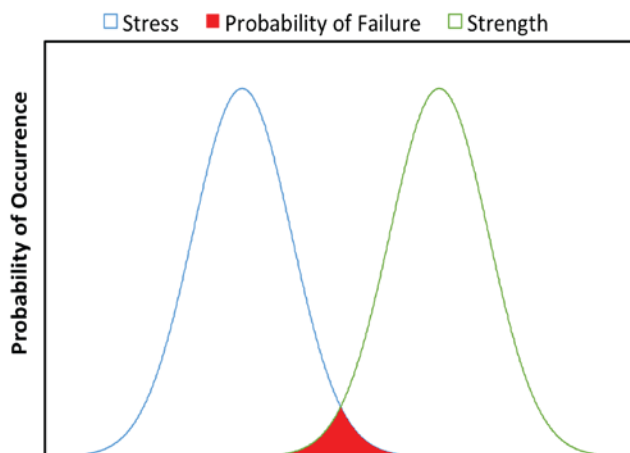


Figure 1. Example probability distributions of stress and strength.

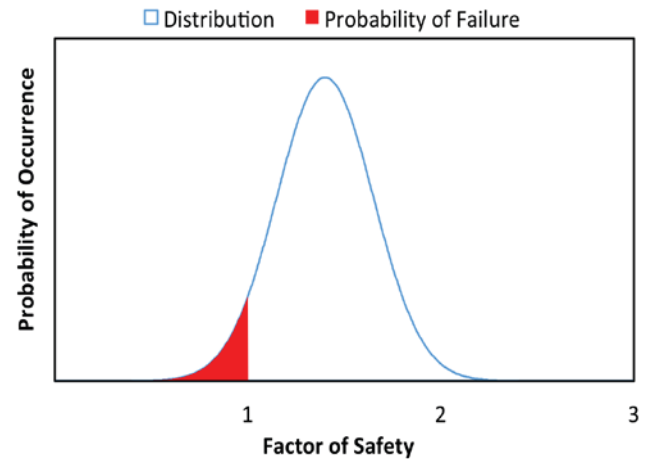


Figure 2. Example output distribution of factor of safety.

The probabilistic approach has some inherent advantages over the more widely used deterministic approach. Two of these advantages are listed below:

- Input parameters represented by single values suggest knowledge of the exact value. Input parameters represented by distributions show uncertainty or variability in the data which makes them a more realistic representation.
- *Probability of failure* is more meaningful than *safety factor* in many instances. Designing to an acceptable probability of failure is a better engineering practice than setting an arbitrary factor of safety threshold.

These are two of the more important advantages of the probabilistic method over the deterministic method, but there are additional, less obvious advantages. There is a smaller chance of user error when a probabilistic analysis is used. This is inherent to the method, because the user is not forced to determine a single value for the input parameters. Fundamentally accounting for any uncertainty in the calculation allows the user to be more certain of inputs. Furthermore, the process can be quicker and easier to implement. After performing a deterministic design calculation, a sensitivity analysis is typically required to deal with uncertainty. A probabilistic analysis removes the need for a sensitivity analysis because the uncertainty is handled inherently.

Probabilistic analyses for engineering design have been used in many industries to determine probability of failure (Townsend, 1993). The natural variability of the ground makes geotechnical design a unique setting where a probabilistic approach should be used more widely. A transition toward using the probabilistic approach for engineering design of geologic structures is taking place for rock wedge failure (Park and West, 2000), slope stability (Zhou et al., 2002), and room-and-pillar failure in oil shale mining (Reinsalu, 2000) and underground uranium mines (Coates, 1974), among others. This transition to using a probabilistic approach for geotechnical engineering design has been slow, controversial, and mostly neglected by the room-and-pillar coal mining industry (Baecher, 1982).

COAL PILLAR DESIGN

Like many other structural design problems, coal pillar design can be expressed as a factor of safety. Estimating stress on a coal pillar can be relatively straightforward, but the method for determining coal pillar strength is not as well established. There are many equations for coal pillar strength, and any of them can be appropriate in a given set of conditions.

The stress supported by a pillar in a room-and-pillar mine can be assumed to be a function of the tributary area and the pillar area, shown in Figure 3, as well as the overburden stress. It is assumed that each pillar supports the volume of overburden in the column above the tributary area of that pillar. For square pillars with a consistent, rectangular pattern, the equation for pillar stress, σ_p , becomes

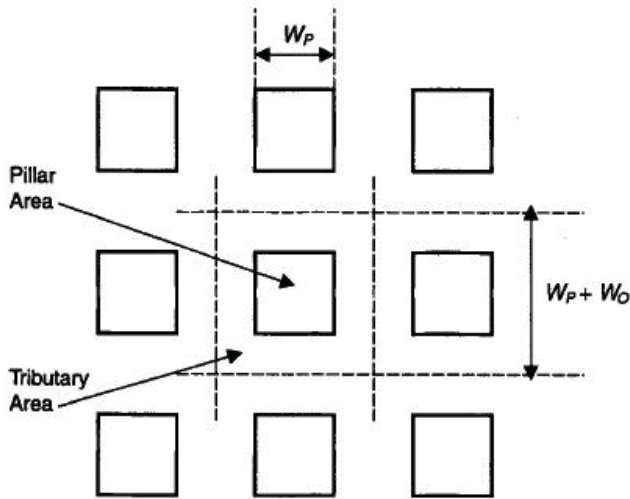


Figure 3. Tributary area in a room-and-pillar mine (Zipf, 2001).

$$\sigma_p = \sigma_z \left(\frac{W_p + W_o}{W_p} \right)^2$$

$$\sigma_z = \gamma z$$

Where σ_z , the vertical stress, is the product of γ , the unit weight of the overburden, and z , the overburden thickness. As shown in Figure 3, W_p is the pillar width and W_o is the width of the opening.

Determination of coal pillar strength is much less straightforward. Pillar strength is commonly estimated using the Bieniawski equation (Bieniawski, 1992a):

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

Where S_i is the compressive strength of coal, and W/H is the coal pillar width to height ratio. Like most equations for coal pillar strength, the Bieniawski equation is empirical. This equation was chosen because its ubiquitous use for calculating the stress of square coal pillars.

The Analysis of Retreat Mining Pillar Stability (ARMPS) program created by the National Institute of Occupational Safety and Health (NIOSH) is often used in addition to or instead of a factor of safety calculation. The ARMPS program reports a "stability factor," which is similar to a deterministic safety factor. The stability factor is the ratio of the load bearing capacity of the pillars within the active mining zone (strength) to the total load applied to the pillars within the active mining zone (stress). The active mining zone is defined to include all of the pillars within five times the square root of the depth of cover of the excavation front. Designing with a stability factor of at least 1.5 is recommended when the depth of cover is greater than 1,000ft (Mark and Chase, 1997).

PROBABILISTIC COAL PILLAR STUDY

Two mine geometries were assumed to test the probabilistic procedure involved with underground coal pillar design. Both geometries were six-entry panels with 20-ft entries. One scenario had 40 ft x 40 ft pillars, the other had 60 ft x 60 ft pillars. The results for these designs were calculated for multiple depths.

A mean value was assigned to each required input parameter, and a reasonable standard deviation for each value was estimated. Each input parameter was assumed to be normally distributed. Except for the pillar dimensions and the depth of cover, the distribution of all parameters remained the same for both scenarios.

The compressive strength of coal is notoriously difficult to test in a laboratory. Coal samples degrade quickly once removed from confinement, and coal exhibits a rather extreme size effect. The in situ strength of coal is generally accepted to be 900 psi, but this value has been known to vary. In situ coal strength has been determined to fall between 780 and 1070 psi (Mark and Barton, 1997). Therefore, the mean value for in situ coal strength was assumed to be 900 psi with a standard deviation of 72.5 psi. This range results in the typical range of in situ coal strength reported by Mark and Barton to be approximately ± 2 standard deviations.

The room-and-pillar layout was assumed to consist of 40-ft square pillars on 60-ft centers for the first scenario and 60-ft square pillars on 80-ft centers for the second scenario. Both of these geometries result in an entry and crosscut width of 20 ft. The pillar width was given these mean values with a standard deviation of 0.5 ft. The entry width was assumed to be the center-to-center spacing minus the pillar width to keep the mine geometry consistent. The mining height was given an arbitrary mean of 6 ft and standard deviation of 0.25 ft.

All of the input parameters required for this simple study are listed in Table 1. The only parameter changed between the first and second runs is the pillar size. Depth was varied to compare the probabilities of failure from the probabilistic study to the stability factors from ARMPS. The stability factors were calculated for the mine geometries under development loading only.

Table 1. List of statistical parameters for each input value.
All distributions were assumed to be normal.

Parameter	Mean	St. Dev.
Depth	Varied	20 ft
Overburden Unit Weight	165 pcf	6 pcf
Entry Width	20 ft	0.5 ft
Pillar Height	6 ft	0.25 ft
Strength	900 psi	72.5 psi
Pillar Width		
Run 1	40 ft	0.5 ft
Run 2	60 ft	0.5 ft

Monte Carlo simulations were performed for each of the two runs to determine distributions for the likely factors of safety resulting from these synthetic data sets with the stress and strength equations discussed above. Random numbers were generated by MATLAB (MATLAB, 2012) for each of the input parameters, with the exception of entry width, which is a function of pillar width, and the factor of safety was determined. Each Monte Carlo simulation consisted of one million iterations for each of the two runs.

RESULTS

The results from the Monte Carlo simulations were compared to the stability factors calculated by ARMPS and the deterministic safety factors for a variety of depths. The deterministic factor of safety results were found by inputting only the distribution means into the stress and strength equations. For the first scenario, the depth of cover was varied from 400ft to 1,400ft. The depth of cover was varied from 400 ft to 2,400 ft for the second scenario.

Plotted in Figure 4 are the probability of failure from the stochastic analysis, the safety factor from the deterministic analysis, and the stability factor from ARMPS for varying depths of cover for 40 ft x 40 ft pillars. There is relatively good agreement between all three pillar design methods. The minimum stability factor recommended for a “satisfactory” pillar system is 1.5 for depths of cover greater than 1,000 ft (Mark and Chase, 1997). The depth of cover corresponding to a stability factor of 1.5 is approximately 1,100 ft. At this same depth, the safety factor resulting from the deterministic method is approximately 1.

The probability of failure at 1,100 ft is approximately 0.6, which means 60% of pillars subjected to this loading scheme are expected to fail. This probability of failure is more conservative than the other results. The distribution of safety factors is normally distributed and centered close to 1, as shown in Figure 5. The largest safety factor calculated was approximately 1.6 and the smallest value was approximately 0.6. This range is due to the standard deviations assigned to the input parameters.

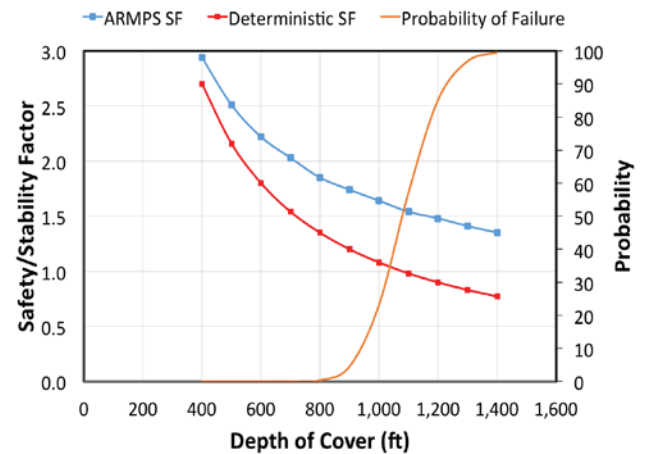


Figure 4. ARMPS stability factor, deterministic safety factor, and the probability of failure for the first scenario (40 ft x 40 ft pillars) plotted versus depth of cover.

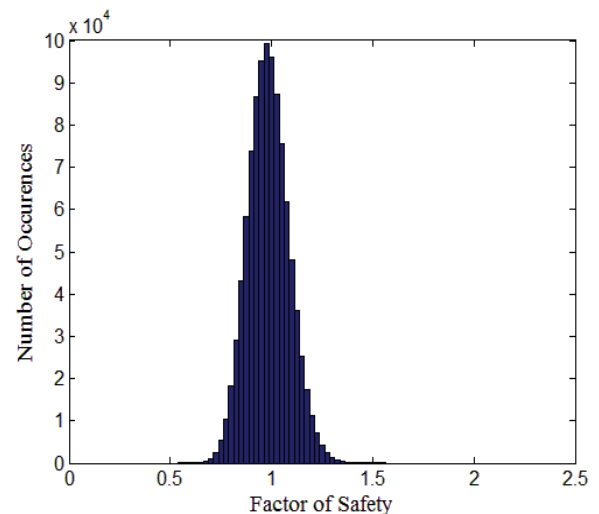


Figure 5. Factor of histogram for the second mine geometry (40 ft x 40 ft pillars) at a depth of 1,100 ft.

The second Monte Carlo analysis was performed assuming 60 ft x 60 ft pillars. The depths for this geometry were varied from 400 ft to 2,400 ft. The resulting ARMPS stability factors, deterministic safety factors, and probabilities of failure are plotted versus depth in Figure 6.

The synthetic study resulted in more conservative values than ARMPS for the second scenario. The ARMPS stability factor approaches 1.5 for a depth of 2,400 ft. At 2,400 ft deep, the deterministic safety factor is approximately 0.8 and the probability of failure is 0.992.

DISCUSSION

The results from the synthetic study and the results from the ARMPS varied somewhat. The difference between the results of the two methods increases with increasing depth of cover. This is most likely because the tributary area method of pillar stress determination was used for the synthetic study. The tributary area

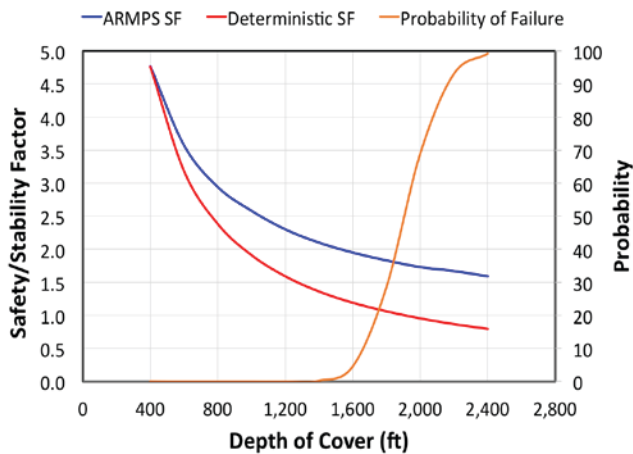


Figure 6. ARMPS stability factor, deterministic safety factor, and the probability of failure for the second scenario (60 ft x 60 ft pillars) plotted versus depth of cover.

method leads to a conservative estimate of the stress on a pillar by as much as 40% (Bieniawski, 1992b).

Before the probabilistic approach to coal pillar design could become widely applied, standards would need to be created. Geotechnical engineering involves too much variation to adhere to the six sigma standard. An acceptable probability of failure threshold would have to be decided upon.

For example, if a probability of failure of 30% were deemed an appropriate design threshold, this would correspond to a stability factor of approximately 1.6 for the first scenario and 1.8 for the second scenario. A more realistic approach for estimating coal pillar stress should be used that would result in a more reasonable value that could be consistent with industry-wide standards.

The scope of the probabilistic approach could be increased by performing a similar calculation for an entire panel rather than just one pillar. This could be done with a Gaussian field representing the spatial distribution of coal strengths, entry height, etc. While the Monte Carlo for the simple, single-pillar example has a rather quick computation time, a panel-sized probabilistic method could take a significant amount of time to perform.

CONCLUSIONS

Uncertainty quantification is necessary in all engineering practices. This is especially true when regarding geotechnical engineering. Probabilistic analyses for engineering design are becoming more prevalent in many fields where a high degree of uncertainty is present but remain mostly absent in coal pillar design.

Performing a probabilistic study rather than a deterministic one is a relatively easy way to quantify the uncertainty in an engineering design. A sensitivity analysis requires actively varying the inputs a slight degree to determine how the output is affected. This process can be much more time consuming, more user-intensive, and less comprehensive than a probabilistic analysis like the one performed here.

A probabilistic approach to coal pillar design has potential to be a viable alternative to traditional deterministic methods. Accounting for variability or uncertainty in the inputs and outputs of engineering design calculations provides more meaningful results. Designing to arbitrary deterministic thresholds indirectly admits the fault in the deterministic approach.

The simple synthetic study performed involved comparing the results of a simple factor of safety determination to the stability factor calculated by ARMPS. Both the probability of failure and a deterministic factor of safety were determined using the Bieniawski equation for coal pillar strength and the tributary area assumption for coal pillar stress.

For 40-ft square pillars in a six-entry panel under development loading, ARMPS resulted in a stability factor of 1.5 at approximately 1,100 ft of cover. This same scenario in the synthetic study resulted in a probability of failure of approximately 60%. The results of the synthetic study agreed somewhat with the output from ARMPS, but were more conservative than ARMPS. The conservative nature of the synthetic study was found to increase with depth.

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Validation of Entry Stability Factors Determined By the Strength Reduction Method Against Empirical Approaches

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ABSTRACT

The design of support for underground excavations is a complex process in which the interaction between supports and the rock mass must be evaluated. The strength reduction method (SRM) is useful because it provides a stability factor of the supported excavation that can easily be compared between support alternatives. The method is based on numerical model analysis where the stability factor is determined by reducing the rock mass strength until collapse is indicated in the model. This paper presents the results of validation studies that were conducted using the FLAC3D stress analysis software to compare SRM-calculated stability factors to the empirically based Coal Mine Roof Rating (CMRR) and the Analysis of Roof Bolt Systems (ARBS). The objective was to determine whether the SRM, with its basis in mechanics, would predict similar outcomes as the fundamentally different ARBS and CMRR methods with their basis in empirical observations. A total of 450 different scenarios were evaluated in which the geology, horizontal stress, depth of cover, entry width, support spacing, and support length were varied. The results showed that the SRM-calculated stability factors were able to satisfactorily capture both the “weak bed” and “strong bed” adjustments in the CMRR. The SRM results were found to be well correlated to the ARBS stability factors for the 450 cases considered. Further analysis showed that the SRM accurately captures the effect of bolt length, depth of cover, and entry width on the overall stability of an entry.

INTRODUCTION

During the design of coal mine entry support systems, the design engineer needs to analyze the suitability of the proposed support system for the expected geological environment and loading conditions. The complex interaction between the surrounding rock and the support needs to be taken into consideration when designing support systems (Brown and Bray, 1982; Hoek *et al*, 2000). Numerical models are well suited to conducting rock-support interaction studies during the design stage. However, numerical model outputs do not necessarily provide a direct assessment of the overall stability of an excavation, making it difficult to determine the relative merits of one support system over another. Recently, numerical modeling procedures were developed to estimate the “stability factor” of supported coal mine entries

(Esterhuizen, 2012) using the strength reduction method (SRM) (Zienkiewicz *et al*, 1975). The method makes use of numerical models in which the rock strength is reduced until collapse is indicated. The stability factor is calculated as the inverse of the strength reduction factor. The calculated stability factors can be used to as an indicator of the overall stability of an entry, allowing for direct comparison of support alternatives.

During the development of the SRM modeling procedures, five case histories were selected to calibrate the models and to test the method. The models provided reasonable estimates of stability factors for these selected case studies (Esterhuizen, 2012). However, it is necessary to test the validity of the approach against a broader range of empirical evidence.

In this paper, the stability factors calculated by the SRM are compared to the results of the empirically based Coal Mine Roof Rating (CMRR), (Molinda and Mark, 1994 and 1996) and the Analysis of Roof Bolt Systems (ARBS) (Mark, 2000; Mark *et al*, 2001). The objective was to determine whether the SRM, with its basis in mechanics, would predict similar outcomes as the fundamentally different ARBS and CMRR methods, with their basis in empirical observations.

THE CMRR AND ARBS EMPIRICAL METHODS

The CMRR rates the strength of bedded strata in the bolted horizon on a scale of 0 to 100, with higher values indicating stronger roof (Molinda and Mark, 1996). The CMRR can be determined from drill core (Mark *et al*, 2002b) or by underground inspection of roof exposures. The CMRR is calculated by first determining the ratings of the individual lithological units in the roof and then calculating the weighted average of the ratings within the bolted horizon. The CMRR can be adjusted upwards for the presence of a “strong bed” within the bolted horizon, or it can be adjusted downwards if the bolted horizon is overlain by a “weak bed.” Adjustments can also be made to account for moisture sensitivity of the rock. The method of rating the roof and making adjustments is based on observations and experience with roof performance in operating coal mines and has been successfully used to characterize the roof in coal mines around the world.

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The ARBS method was developed to assist in the design of coal mine entry support through statistical analysis of more than 150 real-world case histories of support performance in operating coal mines (Mark, 2000; Mark *et al*, 2001). The database of case histories was developed by inspecting rock conditions and support systems at 37 operating coal mines in the United States (Molinda *et al*, 2000). At each site, the roof strata were described by the CMRR, the details of the actual supports installed were recorded, and information was collected on the rate of roof falls in the area. The roof fall rate was used to evaluate the performance of the support systems. Areas in which the roof fall rate exceeded 2 falls per 3,000 m of entry development were considered to be “unsuccessful.” The database was statistically evaluated using multiple regression and logistic regression techniques.

As part of the analyses, a parameter was developed that represents the “support intensity” by incorporating the support length, capacity, spacing, and entry width into a single parameter. The relationship between the support intensity, CMRR, and roof fall rate was used to determine the discriminant equation that best separates between “successful cases” and “unsuccessful cases” (Mark, 2000). The discriminant line can be used to determine the required support intensity for any given CMRR value. A “stability factor” of 1.2 is suggested for support design, which implies that the support intensity should be 20% greater than indicated by the discriminant equation.

For the analyses presented here, ARBS stability factors of the support systems were calculated by first determining the support intensity that would result in a stability factor of 1.0 for the given CMRR and geometric information. The stability factor of the entry was then calculated as the ratio of the actual support intensity over the support intensity that is required for a stability factor of 1.0.

THE NUMERICAL MODEL BASED STRENGTH REDUCTION METHOD

The strength reduction method (SRM) calculates a stability factor of an excavation by gradually reducing the rock strength in a numerical model until failure is indicated by the inability of the model to reach a state of equilibrium. The excavation stability factor is expressed as the inverse of the strength reduction factor. For example, if collapse occurs when the strength is reduced by a factor of 0.8, the stability factor would be 1.25.

The technique is applied by first conducting a stability analysis using average rock strength properties. Depending on the outcome, the analysis is repeated using either a decreased or increased rock mass strength until the point of failure is satisfactorily bracketed. Strength adjustments are achieved by simultaneously reducing or increasing the cohesion, tensile strength, and the coefficient of friction of the rock matrix and the bedding planes. The cohesion and tensile strength values were reduced or increased without restrictions. The friction coefficient was only allowed to range within 10% of the average value to prevent unrealistically low or high frictional properties from being defined. The strength of the support system is unchanged.

The focus of the study was on large-scale roof collapse, and it was assumed that smaller roof falls would be accommodated by surface supports, such as roof screen. Failure was, therefore, defined as the collapse of the roof of the entry, up to or above the

supported horizon. In the models, collapse usually occurs when the roof of the entry is moving downwards at a constant velocity, and equilibrium is not reached after an extended number of solution cycles.

When applying the SRM to underground coal mine excavations, it was found that floor heave was sometimes the mode of ultimate instability. This occurred because the floor rocks can be weaker than the roof rocks. All analyses were, therefore, conducted by only reducing the rock strength above the roofline of the excavation while leaving the coal and floor rocks unadjusted.

NUMERICAL MODELING PROCEDURES

All the numerical models were evaluated using the FLAC3D finite difference software (Itasca, 2011). The SRM was implemented by using the built-in FISH programming language available in FLAC3D.

Model Layout

The models presented here were set up to simulate cross sections through coal mine entries in various geological settings encountered in the United States. The model boundaries were located at least three entry widths away from the edges of the entry and extended about 20 m above and 10 m below the entries. The outer boundaries of the models were fixed against normal displacement, while allowing displacement to take place along the boundary surface (roller boundaries). The models were typically 1.2 m thick in the direction perpendicular to the section, which represents the typical slice of rock supported by a single row of bolts. The element (zone) sizes in the vicinity of the entry were between 0.25 and 0.35 m. The element width was increased up to about 1.2 m at the excavation remote locations where rock failure was not expected to occur. Explicit interfaces were modeled between the different rock units. The interface elements follow the Coulomb Criterion and allow bedding opening and closing to be simulated. Rock bolt supports were modeled across the entry roof, located in the center of the slice. The boundary conditions caused the model to effectively simulate a long entry excavation with repeating rows of supports. Figure 1 shows a typical model in which grouted bolts were evaluated. The shaded elements indicate yielding of the rock by shear failure or sliding along the bedding planes. The image represents the rock yield condition at the point of collapse of the entry during an SRM analysis.

Constitutive Model

The strain-softening ubiquitous joint constitutive model in FLAC3D was used to simulate the bedded coal measure rocks. The modeling approach closely resembles the “synthetic rock mass” concept (Pierce *et al*, 2007) in which the rock matrix and the discontinuities are modeled separately. This approach is necessary to capture both rock matrix failure and bedding-related slip in the sedimentary coal measure rocks. A significant modification to the standard ubiquitous joint model was the inclusion of anisotropic rock matrix strength to simulate the lower strength of bedded strata when loaded parallel to the bedding planes. Matrix anisotropy was incorporated through the built-in programming capabilities of FLAC3D. The models were run in the small-strain mode to alleviate the formation of unrealistic kink-bands in the ubiquitous joint materials.

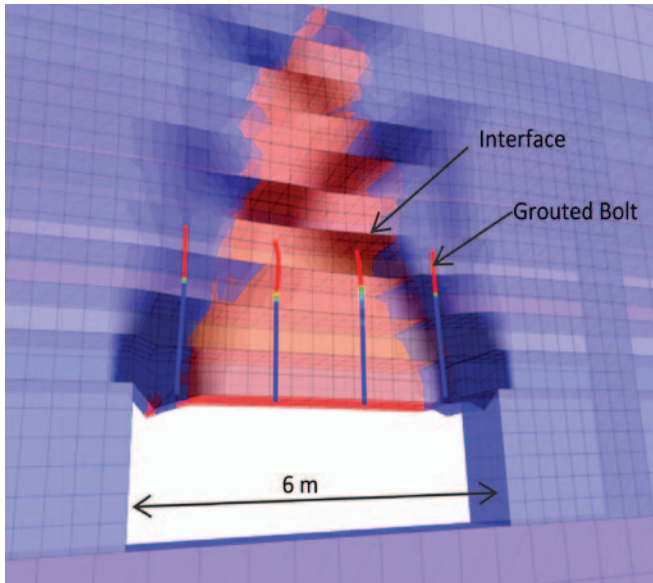


Figure 1. Example of a numerical model of an entry supported by 1.8-m grouted bolts in weak bedded shale rocks at the point of collapse. Dark grey shading indicates rock damage (plastic strain). The red shaded roof is collapsing. Bolts are indicated as blue lines with potential grout slip shown in red.

Model Loading

The vertical stress within the models was initialized to represent the depth of cover being considered in each model. Various horizontal stress scenarios were modeled. The horizontal stress was assumed to have a depth-dependent component, as well as a tectonic component, according to Mark and Gadde (2008) and Dolinar (2003). The tectonic component is closely related to the modulus of elasticity of each rock layer.

Modeling Supports

Support units were modeled using the built-in finite element structural components available in FLAC3D. All the supports modeled were assumed to be fully grouted rock bolts with a small pretension of 10 kN associated with the thrust of the roof bolting machine. Bolts were assumed to be No. 6 grade 75, which was modeled as 19 mm in diameter with an ultimate tensile strength of 190 kN. The grout bond strength was set at 200 kN/m to represent conservative bond strengths achieved in coal measure rocks (Zipf 2006). The grout initial stiffness was set at 670 MN/m/m based on typical field trial results (Mark *et al.*, 2002a). The support units were installed in the models after about 30% of the elastic roof deformation has taken place, representing bolts that are installed near the face of an advancing entry, before full deformation has occurred.

Rock Properties

Standardized procedures have been developed (Esterhuizen *et al.*, 2013) that allow design engineers to develop estimates of rock strength properties from CMRR ratings. The rating-derived properties can be supplemented by site-specific laboratory test data, if available. For the models presented here, it was assumed

that the CMRR rating of each rock unit was known, allowing the standardized procedure to be applied to create model inputs.

SCENARIOS EVALUATED

Numerical models were created to simulate coal mine entries in five different geological scenarios representing a variety of conditions that can be encountered in US coal mines. Three horizontal stress scenarios were considered and three depths of cover. Several support and entry width scenarios were considered. Each support scenario was evaluated for 45 different combinations of CMRR, horizontal stress and depth of cover. Stability factors were determined for each model. The ARBS-predicted stability factor was also calculated for each scenario.

Geological Scenarios

The five geological scenarios were created by combining four lithological units in different ways to produce a variety of roof conditions that are likely to be encountered in coal mines. The four lithological units and their CMRR based unit ratings are presented in Table 1.

The geological scenarios were designed to provide CMRR values ranging from about 35 to 45, representing roof conditions where the support system is expected to act in a beam-building mode (Mark 2000). Higher CMRR values are associated with bolts operating in the suspension mode, which falls outside the scope of this paper.

The roof strata were assumed to consist of 60-cm-thick units, separated by distinct interfaces. The interfaces were assumed to have similar shear resistance properties as the bedding planes within the underlying rock unit. Table 2 summarizes the five different geological scenarios and the CMRR calculated for a supported interval of 1.8 m.

Depth of Cover

Models were evaluated at 100-m, 200-m and 300-m depth of cover. The cover depth essentially affects only the magnitude of the vertical stress in the models. The vertical stress magnitude is used in the determination of the horizontal stress.

Horizontal Stress

Three horizontal stress scenarios were selected to represent unfavorable, moderate, and favorable stress conditions that are likely to be encountered in US coal mines. The first two scenarios represent high horizontal stress conditions that exist in the Eastern US coal regions, while the third scenario is more typical of the Western US coal regions. Table 3 summarizes the stress scenarios modeled. The minor horizontal stress was set at 0.6 times the vertical stress in all the scenarios.

Support Scenarios

The following support variables were evaluated:

- Support lengths of 0.9, 1.2, 1.8 and 2.4 m
- Spacing between supports in a row of 1.2 and 1.7 m
- Entry widths of 5, 6 and 7 m

Table 1. Description of lithological units used in the models.

Description of Layer	Rock Type	Rock Strength (MPa)	Bedding Intensity Rating	Bedding Strength Rating	Unit Rating
Very Weak	Shale	20	9	10	29.0
Weak	Shale	30	12	13	37.0
Moderate	Shale	40	15	16	44.0
Strong	Sandstone	60	18	20	54.0

Table 2. Description of roof rocks and CMRR values of geological scenarios evaluated.

Scenario	Roof composition	CMRR
1 Weak roof	Entire roof is weak shale	37.0
2 Moderate roof	Entire roof is uniform shale of moderate strength	44.0
3 Increasing strength	Weak shale 1.2-m thick overlain by moderate shale	38.3
4 Weak bed	Moderate shale roof with single very weak shale unit at 1.8-2.4 m	40.0
5 Strong bed	Weak shale with a single moderate shale unit at 1.2-1.8 m	38.3

Table 3. Description of horizontal stress conditions evaluated.

Stress Condition	Description
1 Unfavorable	The entry is oriented perpendicular to the major horizontal stress which is 2.0 times the vertical stress
2 Moderate	The entry is oriented parallel to the major horizontal stress, which is 2.0 times the vertical stress
3 Favorable	The entry is oriented perpendicular to the major horizontal stress which is equal to the vertical stress

The spacing between rows of supports was held constant at 1.2 m in all the analyses. Each of the support scenarios was evaluated for the 45 combinations of geology, depth of cover, and horizontal stress.

COMPARISON OF THE SRM TO CMRR TRENDS

The results of the SRM analyses were evaluated to determine whether they satisfactorily capture the increasing stability of coal mine entries with increasing CMRR. A further assessment was made to see if the “strong bed” effect and the “weak bed” surcharge effect were also captured by the SRM models.

Increasing Stability with Increasing CMRR

The results for a case where a 6-m-wide entry is located at 200 m depth of cover, supported by 1.8-m fully grouted bolts spaced

1.2 m apart are presented in Figure 2. The results for the three stress scenarios are shown separately, as well as the averaged results. The averaged results show a strong correlation with the CMRR values. The stress scenario can have a significant effect on the SRM stability factors. This is not unexpected because experience has shown that the horizontal stress magnitude has a significant effect on entry stability (Mark, 2000; Mark and Mucho, 1994). The impact of the increasing CMRR on stability factors is reduced when the stress is unfavorable compared to the favorable stress case.

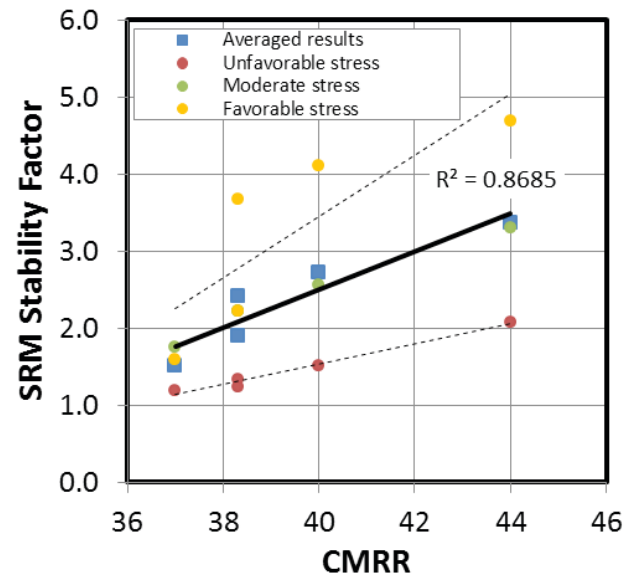


Figure 2. Increasing trend of SRM-calculated stability factors with increasing CMRR values. Results are for a 6-m-wide entry at 200 m depth supported by 1.8-m fully grouted bolts.

The results also demonstrate that the effectiveness of the support system can vary for a given CMRR, depending on the details of the geologic layering. For example, the CMRR of 38.3 applies for both the “increasing” and the “strong bed” geological scenario. However, in favorable stress conditions, the SRM stability factors differ considerably, mainly because the bolts are overlain by stronger strata in the “increasing” geological scenario, which inhibits roof collapse. This kind of detail is not captured in the CMRR rating of the roof.

Weak Bed Surcharge and Strong Bed Effects

The ability of the SRM models to capture the effects of a weak bed and a strong bed within the bolted roof is demonstrated

by considering the same set of results as in the previous section. The weak bed surcharge effect is captured when comparing the results of geological scenario 2 to scenario 4, where a weak bed is introduced in the moderate strength shale. The strong bed effect is captured when comparing geological scenario 1 to scenario 5, where a stronger bed is introduced into the weak shale. The results presented in Figure 3 show that the SRM-calculated stability factors follow the same trends as the CMRR adjustments. The stress condition can again be seen to have an important effect on the results. For example, in the unfavorable stress condition, the strong bed only has a minor impact on the SRM-calculated stability factors. Examination of the model results shows that in this case the stress is able to fail the stronger bed, negating most of the advantage of being able to anchor in stronger rock.

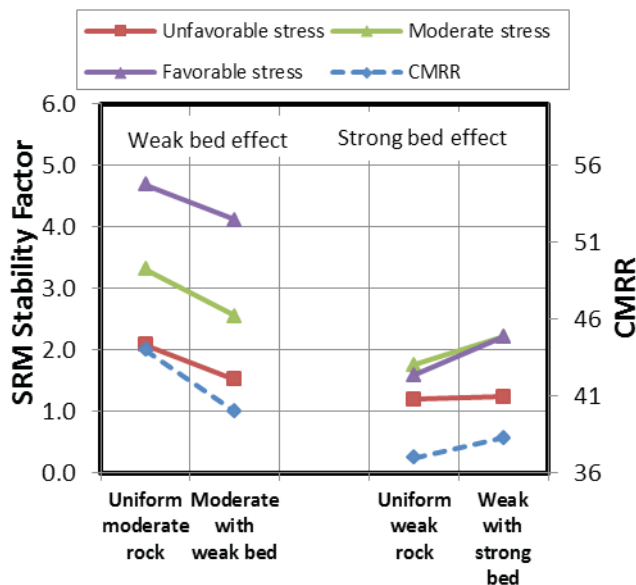


Figure 3. Changes in stability caused by weak bed and strong bed adjustments indicated by SRM and CMRR results.

This evaluation has shown that the SRM stability factors follow similar trends as the empirically based CMRR ratings. The SRM is also able to identify the advantage of a strong bed in the bolted horizon and also the weakening effect of surcharge from overlying weak layers.

COMPARISON OF THE SRM TO ARBS RESULTS

The stability factors calculated by the SRM were compared to ARBS-predicted stability factors for the various combinations of geology, stress, depth of cover, support lengths, and support spacing described in the Scenarios Evaluated section of this paper. A total of 450 cases were analyzed.

Since the ARBS discriminant line captures the “average” effect of a variety of horizontal stress scenarios, the SRM stability factors for the three stress scenarios were also averaged for each case. This resulted in 150 averaged cases from the 450 original cases. The results are presented in Figure 4. The good correlation between the results for this wide ranging set of conditions demonstrates the similarity of the outcomes of these two fundamentally different

approaches—the ARBS being an empirical method and the SRM being a mechanics-based approach. The figure shows that the SRM stability factors are generally higher than the ARBS stability factors. This is expected and is discussed in greater detail in the Discussion section of this paper.

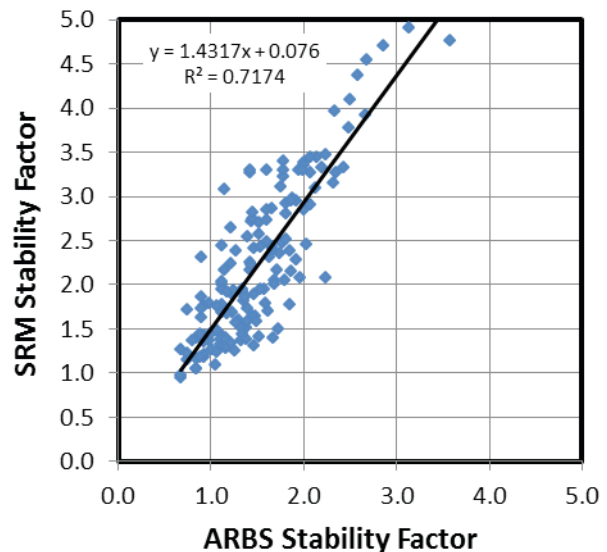


Figure 4. Correlation between SRM-calculated stability factors and ARBS-calculated stability factors for 150 cases. Each SRM case represents the average of three horizontal stress scenarios.

IMPACT OF INDIVIDUAL PARAMETER VARIATION

Further evaluations were conducted to investigate how variation in individual parameters affect the SRM-calculated stability factors and how this compares to the roof fall analysis results conducted during the development of the ARBS method.

The Effect of Depth of Cover

The depth of cover was identified as one of the factors impacting entry stability and was incorporated in the ARBS method for selecting bolt length and support intensity. The depth effect is also clearly captured by the SRM, as shown in Figure 5, which shows the stress-averaged stability factor results for various CMRR values for the 6-m-wide entry, supported with 1.8-m grouted bolts at depths of 100 m, 200 m and 300 m. The figure also shows the ARBS-calculated stability factors, indicating similar trends with increasing depth. The SRM stability factors are generally higher than the ARBS stability factors, as discussed previously. However, the correlation between the two sets of stability factors for these selected cases is excellent, as shown in Figure 6.

The Effect of Bolt Length

Bolt length is one of the important factors affecting entry stability and is used as an input variable in the calculation of the support intensity and the ARBS stability factor. The effect of increasing bolt length on both ARBS- and SRM-calculated stability factors is shown in Figure 7 for depths of cover of 100 m, 200 m

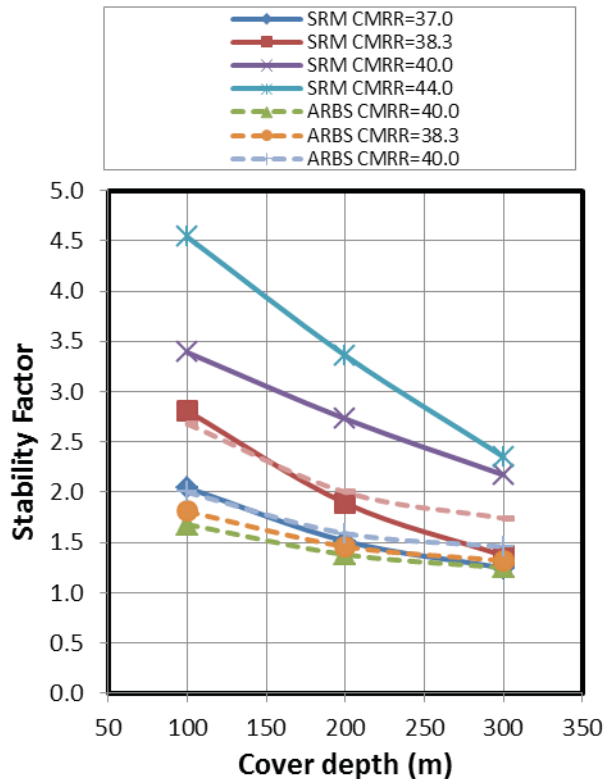


Figure 5. The effect of depth on SRM-calculated stability factors for various CMRR values. Results are for a 6-m-wide entry supported by 1.8-m fully grouted bolts. Each of the SRM results represents the average of three horizontal stress scenarios.

and 300 m. The SRM stability factors were calculated by averaging the results for each bolt length over the five geologies and three stress conditions. The ARBS stability factors are the average for the five geologies. The SRM stability factor values are lower than the ARBS values, but they follow very similar trends, indicating that bolt length is adequately accounted for in the SRM method.

The Effect of Entry Width

Entry width is a well-known factor affecting the stability of coal mine entries. When poor roof conditions are encountered, one of the first actions that can be taken is to reduce the entry width. The entry width is a parameter in the ARBS calculation of the support intensity and is carried over into the ARBS stability factor calculation. Figure 8 compares the trends in ARBS- and SRM-calculated stability factors as the entry width is increased from 5 m to 6 m and finally to 7 m. Each data point was calculated in a similar manner as in the previous section. The results again show that the SRM achieves similar trends in results as the ARBS method. A deviation from the general trend can be seen in the SRM results for the 300-m depth, which indicates that there is no advantage in reducing the entry width from 6 m to 5 m at this depth. This result requires further investigation and verification.

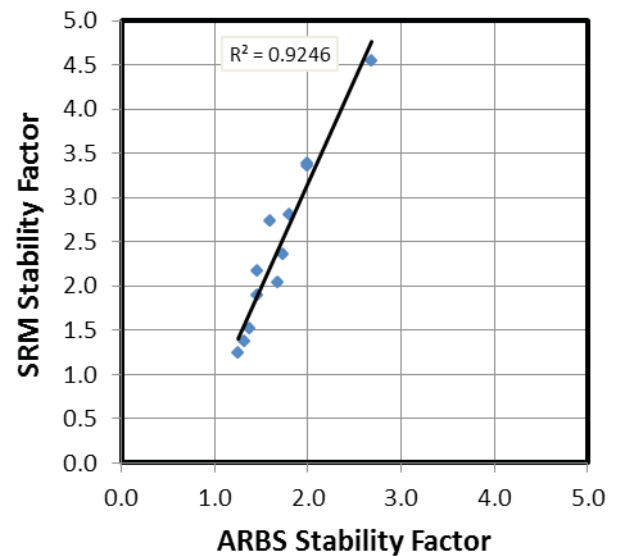


Figure 6. Correlation between stability factors calculated by the ARBS and SRM for an entry supported by 1.8-m-long fully grouted bolts. Each of the SRM results represents the average of three horizontal stress scenarios. These results are a subset of the results shown in figure 4.

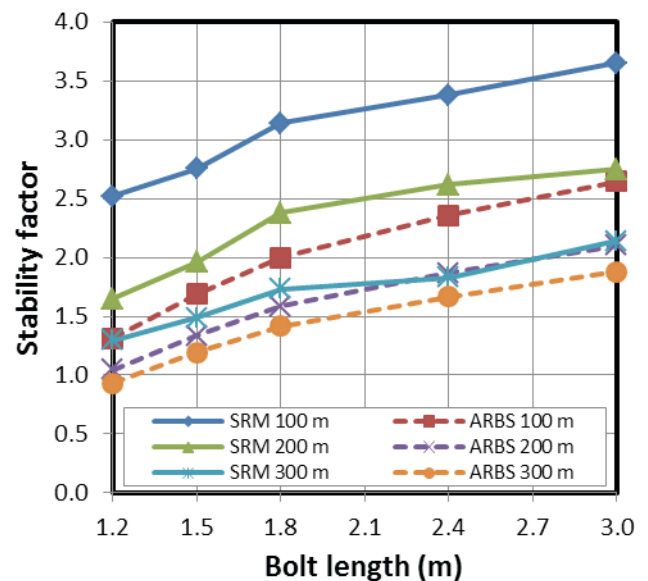


Figure 7. The effect of bolt length on averaged entry stability factors calculated by the ARBS and the SRM methods.

Horizontal Stress Effects

The most obvious parameter causing deviation of the SRM results from ARBS results is the horizontal stress. The horizontal stress may also be one of the major factors that caused the scatter of results found during the analysis of the roof fall database that was used to develop ARBS (Molinda *et al*, 2000). Horizontal stress was not explicitly determined for each case in the development of

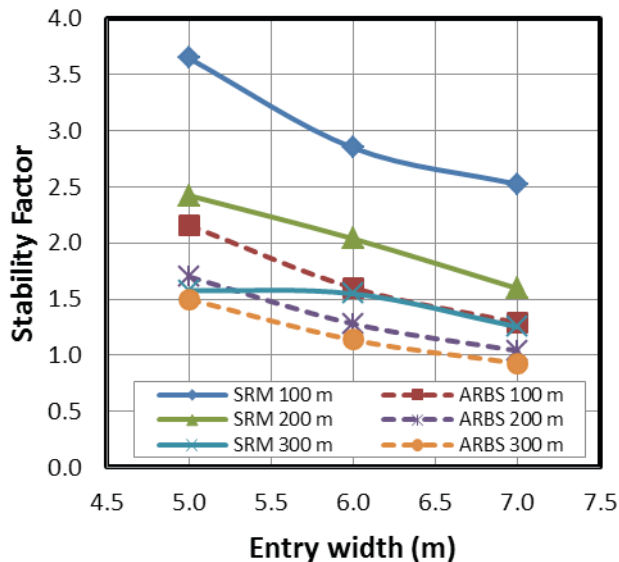


Figure 8. The effect of entry width on averaged entry stability factors calculated by the ARBS and SRM methods.

ARBS but was assumed to be accounted for in the depth of cover. Horizontal stress is seldom measured in operating coal mines.

A review of the SRM results shows that the average SRM stability factor of all entries modeled in the unfavorable stress condition is 1.51. This increases to 2.09 for the moderate stress condition and to 3.07 for the favorable stress condition. The importance of horizontal stress on entry stability has been known for many years (Mark, 2000; Mark and Mucho, 1994) and appears to be well represented in the SRM results.

DISCUSSION

The study of ARBS- and SRM-calculated stability factors for 450 different scenarios shows that good correlation is achieved between the two sets of results. The SRM stability factors are generally higher than the ARBS stability factors. This is not surprising since an ARBS stability factor of 1.0 represents the line that best separates unsatisfactory support performance from satisfactory performance (Mark, 2000), where unsatisfactory support performance is defined as roof fall rates of greater than 2 falls per 3,000 m of entry development. The recommended ARBS stability factor of 1.2 attempts to achieve failure rates lower than 2 falls per 3,000 m.

On the other hand, the SRM stability factor of 1.0 represents a 50% failure probability, as in the classical interpretation of safety factors (Harr, 1987; Hoek, 1991). This represents a very high failure rate when compared to the failure rate associated with an ARBS stability factor of 1.0. The equivalence of the two stability factors can be approximated by the correlation equation shown in Figure 4. Using this equation, an ARBS stability factor of 1.0 is equivalent to an SRM stability factor of 1.5. The recommended ARBS stability factor of 1.2 translates into a SRM stability factor of 1.79. Further work is required to determine whether an SRM

stability factor of 1.79 would be appropriate for general design of coal mine entry support.

CONCLUSIONS

This study has shown that the strength reduction method, with its basis in failure mechanics, produces very similar predictions of entry stability as the empirically developed “Analysis of Roof Bolt Systems” approach, in spite of the fact that the two approaches are fundamentally different. The SRM is also able to capture geological impacts on entry stability, such as the “strong bed” adjustment and the “weak bed” surcharge, that are built into the empirically developed coal mine roof rating.

A study of 450 different combinations of geology, entry width, bolt length, horizontal stress, and depth of cover showed that the SRM is able to produce satisfactory interpretations of expected entry stability when compared to the ARBS-calculated results. The effect of individual parameters, such as bolt length, entry width, or depth of cover, all follow similar trends as identified in the ARBS study.

The success of the SRM in matching empirically developed methods opens the door for greater use of numerical models in support design. The models can be used with greater confidence to conduct comparative analyses of support alternatives, investigate likely causes of support system failure, and figure out how support can be improved. Design questions that cannot easily be determined by empirical observation can be investigated through modeling. The greater use of numerical models will also underscore the importance of an adequate knowledge of the rock mass properties, loading conditions, and support components.

DISCLAIMER

The findings and conclusions in this paper are those of the authors and do not necessarily represent the views of the National Institute for Occupational Safety and Health. Mention of any company name, product, or software does not constitute endorsement by NIOSH.

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A Holistic Examination of the Geotechnical Design of Longwall Shields and Associated Mining Risks

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ABSTRACT

This paper examines the design of longwall shields, focusing on those aspects that are critical to either their successful application or contribution to failure during longwall extraction.

In many instances, longwall shields are assessed and designed along lines similar to that of roadway ground support systems, namely according to typical or normal geotechnical conditions. However it will be contended that for longwall shields, this is inappropriate because unlike roadway ground support systems, longwall shields cannot be supplemented with additional or secondary support in localised areas of adverse geotechnical conditions. In other words, the longwall shield needs to be designed according to what are judged to be worst case or adverse geotechnical conditions—the outcome being that a well-designed shield will be significantly over-rated for the majority of its working life. Therefore, it will be argued that the additional capital cost of such shield design can be readily justified as a prudent risk-based outcome that is essential to minimising future business risks due to strata instability on the longwall face.

Shield geometry will be discussed, including such relevant factors as leg angle, inclination of the top caving shield, canopy ratio, operating height range, and tip to face distance. These are all well-established longwall shield design considerations and, most importantly, areas whereby inadequate design can render a longwall shield as highly ineffective. The issue of tip to face distance will be considered in detail, in particular, the extent by which it is an important geotechnical design consideration, the conclusion being reached that, in fact, it is not.

The critical importance of maximising set to yield ratio within practical operating limits will be justified, the wisdom of this having been subject to questioning in more recent technical literature.

Overall, the aim is to provide the industry with a set of suggested guidelines for future use when designing longwall shields and, hopefully, to initiate discussion on a subject that, unlike roadway ground support design, is not well covered in its entirety in the published technical literature.

INTRODUCTION

Few in industry would deny that the shields on most longwall faces work well for the majority of the time. It is the exception rather than the rule that the longwall is unstable with large cavities hindering production and requiring both remediation and recovery. That is not to say that some longwall faces are not more prone to strata instability than others. Certainly, when major roof cavities do form, the operational difficulties and costs involved in the recovery are substantial.

The various geotechnical drivers for strata instability on the longwall face are not addressed in detail. For example,

- depth of cover, panel width, and general face loading conditions
- extraction height and the propensity for face spall
- the contribution of the chain pillar to face stability for sub-critical panels
- periodic weighting of the near-seam overburden
- the influence of chain pillar cut-throughs in causing surges in face loading
- the competence of the immediate roof strata
- traversing major geological structures, such as faults and dykes
- the alignment of joints with respect to the longwall face

These have all been covered in detail in other publications (e.g., Frith and McKavanagh, 2000; Frith 2005) and are well established based on both research studies by various groups and the investigation of numerous longwall roof falls by the author.

On the assumption that all of these geotechnical parameters are used and considered as part of a longwall feasibility process, and a longwall panel layout is established, once mining commences, the mine operator has only two controls available to prevent instability on the longwall face: the longwall shields and face operations. Good longwall operational practice (e.g., keeping the face straight, setting of shields, and maintenance) has been the subject of significant industry interest via the regular column “Wills on Walls” in the *Australian Longwalls* journal, whereby one of the industries authorities on the subject, Nick Wills, provides an informative and, at times, amusing commentary on the subject.

GENERAL CONCEPTS

However the author contends that unless longwall shields are well designed in the first place, best-practice operational controls may not be sufficient to prevent face instability. For example, if the shield legs are leaking significantly or a minor roof cavity (which can always occur) has resulted in the excavation height being greater than the operating range of the shields, the potential for instability on the face may be far greater than what can be achieved by operational controls, such as “keeping going”.

Therefore, the focus is to consider the design of the longwall shields in a more holistic manner in order to help mining companies make informed choices when procuring face equipment, thus maximising the likelihood that operational controls will be effective during mining.

CURRENT STATE OF THE ART

Establishing the current state of the art is difficult because the amount of published literature or industry guidelines on the complete subject of longwall shield design is essentially nil. Many papers discuss longwall instability case histories (Frith 2005) or report research studies on the *in situ* performance of shields (e.g. Truman 2005, Gale 2009) and Barczak 2000 details a number of the mechanical performance criteria involved and associated shield testing regimes. However none (that the author could find) actually address the issue of how one would go about designing a longwall shield in a known set of geological and geotechnical circumstances. Given the critical importance of the longwall shields to the economic performance of a longwall face, this is a surprising state of affairs and one that perhaps the industry might wish to address. This is an initial contribution to that process.

What appears to occur with the shield design process is that:

- (a) The mining company undertakes a feasibility study and decides that it will invest upwards of AU\$1 billion in a longwall mine.
- (b) The longwall OEMs (Original Equipment Manufacturer) are provided with a series of design parameters (e.g., shield width, yield load rating, set:yield ratio, and operating height range).
- (c) The individual OEMs then each prepare a face/shield design to allow the mining company to make an informed decision as to which OEM will be contracted to provide the equipment.

From the authors own experience of having been involved in these processes in the past, common focus areas are (i) fatigue design and testing of the shields, (ii) building the shields to the design specification, and (iii) commercial terms. Certainly, there would be many examples of highly rated, structurally well-designed longwall shields in the industry that have performed far below expectation in terms of longwall face stability. Accepting that the geotechnical environment is a significant driver of face instability, the question that has to be asked is: should other elements of shield design that are critical to its supporting effectiveness also be evaluated as part of the procurement process?

As a starting point, it is hypothesised that longwall shields offer the highest level of roof control when they meet the following six design requirements:

- (i) They are able to be set to the roof along all or most of their canopy length.
- (ii) The leg loads are applied to the roof as close to the face as possible, albeit within practical mining considerations.
- (iii) The tip to face distance is as small as possible, noting that the tip to face distance is defined by the closest position to the face that the canopy is first in contact with the roof, not simply by the canopy tip itself.
- (iv) The yield rating of the shield is as high as possible, the higher the better in terms of roof control on the longwall face.
- (v) The set:yield ratio is as high as possible, this leading to improved levels of direct or active reinforcement to the immediate roof of the longwall face.
- (vi) The leg hydraulics are not impaired by fluid leaks that would allow leg loads and stiffness to drop during the mining cycle.

However, it is immediately obvious that some of these shield design requirements lead to compromises with other longwall design aspects. For example,

- The location of the shield legs, with respect to the longwall face, is limited by the presence of the AFC and a forward walkway, both having width requirements if they are to meet other longwall design outcomes (e.g. AFC conveying capacity and front walkway ergonomics).
- Even if the canopy tip could be set against the roof at the face, risks associated with the shearers drums colliding with the canopy preclude that as a practical proposition.
- Yield loads are limited by such issues as capital cost and shield weight, in terms of transporting the shields underground using rubber-tyred vehicles.

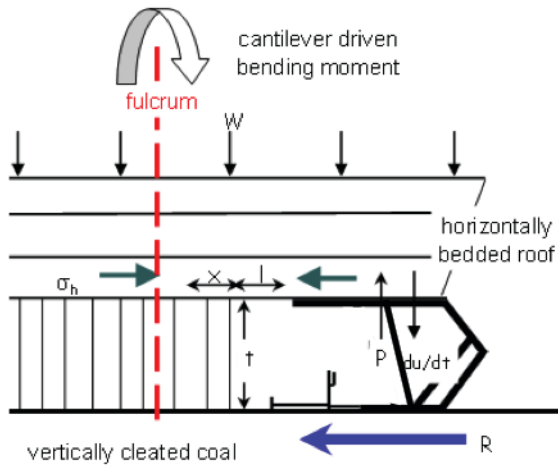
Recognising that such compromises exist, the design of longwall shields should seek to reach the best overall balance so that none of the required design outcomes are unduly poor. The final question then becomes how likely they are to perform within the geotechnical environment of the longwall face under consideration.

The shield design aspects considered (in no priority order) are as follows:

- (i) shield width
- (ii) tip to face distance
- (iii) canopy ratio
- (iv) planned working height range in relation to the maximum operating height
- (v) inclination of the top caving shield within the planned operating height range
- (vi) floor bearing pressures
- (vii) yield load rating
- (viii) set:yield ratio
- (ix) flipper arrangement
- (x) inclination of legs towards the face

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Each will now be considered individually by reference to the conceptual mechanistic model of the longwall face and shield that was first published by Frith 2005 and is reproduced herein as Figure 1.



W - abutment loading
 t - extraction height
 l - tip to face distance prior to spall
 x - increase in tip to face distance due to spall
 σ_h - horizontal stress due to strata bending
 du/dt - rate of powered support/roof convergence
 P - powered support loading
 R - rate of face retreat

Figure 1. Simplified mechanistic model of a longwall face.

Shield Width

Shield widths have increased during the past 20 years from 1.5m to 1.75m, and, currently, 2m wide shields are becoming more common. This evolution has been driven largely by operational efficiency considerations, such as reducing the face production constraint due to shield Lower-Advance-Reset (LAR) cycle times, reducing the number of shields and AFC pans to be moved during face relocations (the move to wider longwall faces has simply exacerbated this requirement), improved shield stability in thick seam longwall extraction and less components on the longwall face requiring maintenance or that can fail.

It is also true to say that, as individual shields get wider, higher load capacities can be achieved (without increasing leg pressures markedly) as increased leg diameters that are directly proportional to the increased shield width, act to increase the shield load rating by the leg diameter squared. The load rating of the current 2m-wide Moranbah North longwall shields (875 tonnes/m) could not have been achieved within a 1.75m-wide shield without significantly increasing leg pressures.

The major downside of increasing shield width (and shield loading as a direct consequence) is that the shields become heavier and potentially more difficult to move around the mine. The 2m wide, 1750 tonne rated Moranbah North shields weigh in the order

of 60 tonnes each so that purpose-built LHDs (Load Haul Dump) needed to be designed and developed to allow their transport within the mine.

Tip to Face Distance

Tip to face distance is undoubtedly the simplest concept in relation to roof stability on the longwall face, being directly analogous to roadway width. It represents the unsupported span between the face and the tip of the shields, the stability or instability of which during mining is a critical outcome. Few longwall operators would disagree that maintaining the tip to face distance as small as possible has a positive impact on roof stability on the longwall face.

The actual tip to face distance in place at any point within the mining process will be governed by a number of independent factors:

- The minimum tip to face distance as determined by the face equipment and method of working (see "l" in Figure 1), which is commonly found to be in the order of 500 mm
- An increased tip to face distance due to face spall effects (see "x" in Figure 1)
- An increased tip to face distance caused by the tip of the shields not being set to the roof — This can have a number of different causes such as the cutting of steps in the roof (Figure 2) and whether two or four leg shields are being used (Figure 3).



Figure 2. Illustration of the impact of 'steps' in the roof on effective tip to face distance.

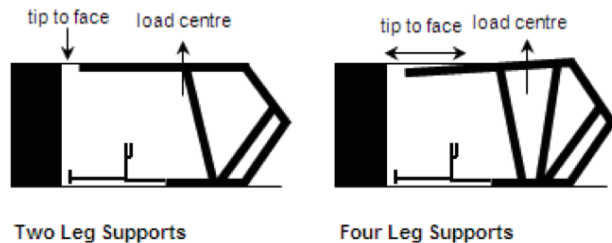


Figure 3. Illustration of different setting characteristics of two and four leg shields on tip to face distance and load centre.

It is typical for the tip to face distance to be analysed and optimised by reference to the position of the shields and face before the cut is taken. A theoretical tip to face of approximately 500mm is consistent with current well-designed longwall shields operating in the Australian coal industry and cannot be reduced further, primarily due to shearer collision concerns. In other words,

operational rather than geotechnical considerations are the limiting factors in this regard.

In reality, the tip to face distance during longwall extraction will only fleetingly, if ever, be as low as 500mm. The influence of face spall can be very significant at times (e.g., under peak face loading conditions or in proximity to a major geological structure), and the author has observed face spall of several metres in conjunction with major instability on the face, rendering any arguments about whether the tip to face distance should be 450mm or 500mm for roof control reasons as essentially meaningless.

Even though the method of face working can be changed to reduce the tip to face figure under extreme face loading conditions by an amount determined by the web depth (e.g., double chocking when working Uni Di IFS, which can only be used when significant face spall is occurring anyway), tip to face distances of 1m to 2m with the shields behind the AFC waiting for the AFC to be advanced (so that double-chocking can again take place) are an inevitable outcome during mining under high face loading conditions, particularly in thick seam environments.

The designed tip to face distance as is shown on OEM face cross-sections is no more than a practical minimum value rather than what is likely to be the case during mining. Accepting that the tip to face distance prior to the cut should be minimised, attempts to reduce it should not be done at the expense of front walkway utility, AFC width, or canopy ratio (see next section) due to the potentially overriding influence of face spall. It is assessed to be of little overall value to compromise the overall face geometry to minimise tip to face before the cut, only to eventually realise that it makes little or no difference to the maximum values that inevitably occur during mining when the potential for roof instability is logically also at a maximum.

Canopy Ratio

Canopy Ratio is the ratio between the canopy length ahead of the legs divided by the canopy length behind the legs (A/B in Figure 4). The importance of the canopy ratio is to provide a balanced canopy, whereby its centre of mass is reasonably close to the location of the legs. This ensures that the canopy tip does not have a significant tendency to drop away from the roof, which will either tend to reduce tip loading of the shield if it stays in contact with the roof or increase the tip to face distance if it does not. Neither of these outcomes is beneficial in terms of roof control.

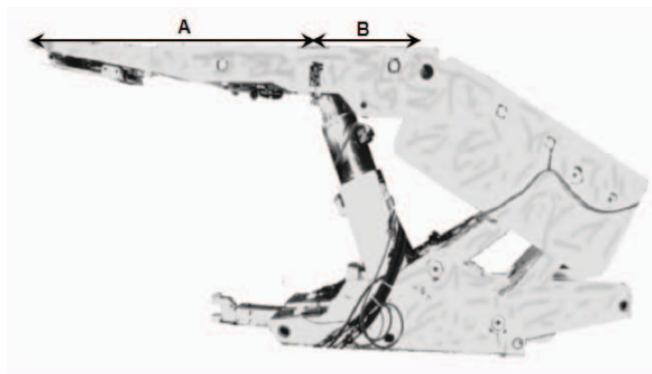


Figure 4. Shield side profile showing the definition of canopy ratio (A/B).

While the author is not aware of any design values for canopy ratio that are backed by fundamental shield design research, values commonly used are between 2 and 3, with values closer to 2 being preferred. Authors such as Gale 2009, Mitchell 2009 and Strawson 2011 generally concur with this, with Barczak and Tadolini 2007 inferring that the value should be < 2 if low tip loads are to be avoided. Obviously, a better canopy balance could be achieved by making the length of the canopy behind the legs (B) longer, although this has the potential to add weight and cost to the shields. It also tends to reduce the shield Support Load Density or SLD (in tonnes/m²), which, despite being a commonly used parameter, is not a particularly useful means of assessing shield design. This is because maximising SLD (by reducing the distance B behind the legs) will almost certainly lead to the canopy becoming less balanced via an increased canopy ratio, which is counter-productive in terms of roof control.

The other obvious outcome when considering canopy ratio is that lengthening the canopy to reduce tip to face distance for strata control reasons is also potentially counter-productive because it will tend to increase the canopy ratio, thus making the canopy less balanced.

Certainly a combination of reducing tip to face distance by making the canopy longer ahead of the legs (A) and also increasing the shield SLD by reducing the canopy length behind the legs (B) is a recipe for disaster in terms of canopy ratio and shield effectiveness.

Planned Operating Height Range in Relation to the Maximum Operating Height

It is important that when a shield is working at the upper end of its designated operating height range, there is still an amount of leg travel to allow the shield to be correctly set to the roof should either the cut inadvertently be taken too high or a small roof cavity forms ahead of the shields. Having the ability to set the shield to the roof for a distance above the designated planned maximum working height is a prudent shield design consideration because once the shield cannot be correctly set to the roof, the threat of major roof instability occurring increases significantly.

Again there are no industry accepted design guidelines in this regard, the amount of extra operating height above the planned maximum working height being a risk mitigation measure to be decided upon by the mine itself. However, as a general rule, the author is of the view that two criteria should be applied:

- (i) the higher the general likelihood of roof instability occurring on the longwall face for geotechnical reasons, the greater the additional operating height, over and above the planned maximum working height, should be.
- (ii) the higher the maximum planned working height, the greater the additional operating height should be - this relates to the argument that the severity of major roof falls on the face increases in line with the face operating height.

Building increased operating height into the shields results in additional capital cost and will also make the shields slightly heavier. However, when this is compared against the consequences of production or reserve losses associated with (a) unplanned roof falls on the face or (b) having to reduce the face working height

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to ensure that a sufficient amount of leg travel is available to accommodate small cavities and/or cut roof horizon variations, such additional costs and shield weight are surely prudent compromises to make.

Inclination of the Top Caving Shield within the Planned Operating Height Range

It is a fundamental requirement of shield design that the inclination of the top caving shield is never less than about 30 degrees to the horizontal. The reason is that broken goaf material should always slide off the top caving shield so that when the shield is being reset to the roof, the available leg load is applied to the roof above the canopy (where it assists roof stability), rather than through the top caving shield where it has no useful purpose, other than to raise broken goaf material during the shield setting process. These two scenarios are illustrated schematically in Figures 5(a) and 5(b).

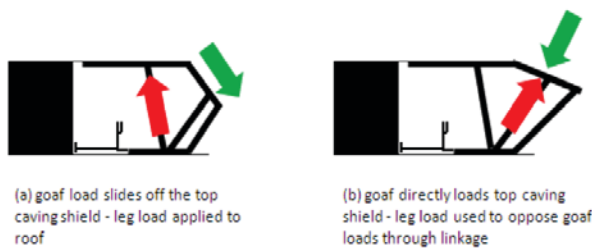


Figure 5. Schematic illustrations of how the inclination of the top caving shield influences leg load distribution.

Frith and Stewart 1994 undertook shear tests between steel and rock and found that the limiting angle of friction was on the order of 26 to 27°. Therefore, a top caving shield inclination of no less than 30° would have a very high likelihood of the resulting broken goaf material sliding off and not directly loading the shield.

While not published, the author is aware of several examples in Australia and overseas of longwall shields being worked very close to the bottom of their operating range with top caving shield inclination angles of substantially less than 20°. Face roof control difficulties have commonly occurred under this scenario.

In this regard it is also interesting to consider an example of field monitoring of linkage loads as a function of associated leg loads (Barczak 2006). Figure 6 is reproduced from Barczak 2006 and it shows that the linkage was going into tension in the same manner and at a not dissimilar magnitude to the compressive loads being generated in the shield legs. The author is not familiar with the specific circumstances of the field monitoring site, other than to state that if the linkage is loading up in this manner as a direct result of leg loading, it can only logically be concluded that the roof above the shield is being loaded to a far lower level than should be the case. This judged to be as would perhaps be expected if the inclination of the top caving shield were too low and goaf material was directly loading the shield via the top caving shield.

Maintaining the inclination of the top caving shield at greater than 30° throughout the operating height range increases in difficulty as the operating height range is increased. The higher

the operating range, the longer the top caving shield needs to be in order to provide enough shield reach, therefore, the lower its

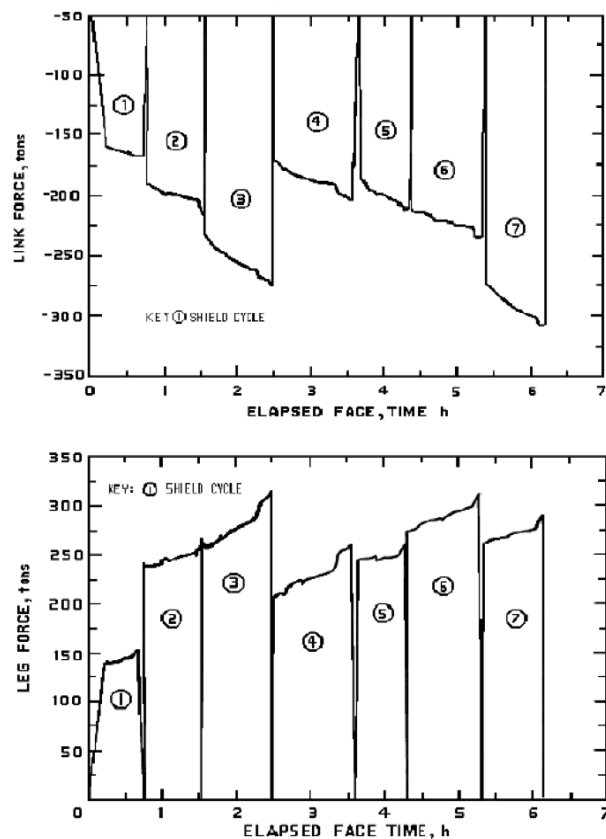


Figure 6. Link force and leg force data for selected shield loading cycles (Barczak 2006).

inclination at the bottom of the operating height range will be (all other factors being equal).

The conclusion is that while providing for a large operating height range may have theoretical benefits, particularly when the seam thickness in a given mine varies significantly, there are potential major roof control downsides when operating such longwall shields at the bottom of their operating range due to the top caving shield having a very shallow inclination angle. The author has observed this on several occasions as being a major controlling factor in ongoing strata control difficulties on the longwall face.

Floor Bearing Pressures

While there are no publications discussing this issue in detail with respect to longwall shields, there are known examples (albeit not published) of major longwall collapses occurring either fully or partly due to the presence of very soft floor beneath the shields, causing a dramatic reduction in shield loading to the roof. It is a fundamental design requirement that any form of standing support (including longwall shields) can be set against both the roof and floor without either undergoing compressive failure.

Whether a longwall shield will fail the floor beneath and effectively “sink” into it, is essentially a bearing failure type

mechanism. Therefore, it is appropriate to consider it in this manner.

Based on information available to the author, the peak vertical stress acting through the shield pontoons and into the floor can be as high as 8 MPa, this being applied at or very close to the toe of the base.

The bearing capacity of any strata unit beneath the shields is a function of (a) the load being applied, (b) the width of the footing (in this case the shield base), and (c) the strength and thickness of the floor material. There are many different methods of analysis that could be used in this regard. Seedsman (2012) provides the following equation for use when considering the stability of the floor beneath a coal pillar:

$$\text{Bearing Capacity} = \text{UCS} / 2 (4.14 + W / 2 / h) \quad (1)$$

where:

W = pillar width

h = unit thickness with an Unconfined Compressive Strength (UCS)

Using this equation with a peak applied stress of say 8 MPa, a UCS can be determined that would represent the limiting equilibrium condition for the first 1m of floor beneath the shields, in terms of whether a bearing failure would occur or not. On the assumption that a pontoon is in the order of 700mm wide (this representing the minimum footing dimension expected for a 1.75m shield), it is found that for the first 1m of floor beneath the base, the limiting equilibrium UCS is 3.6 MPa. For values lower than this floor failure is likely, for values above this, floor failure becomes less likely.

The above equation can be used to consider floor stability for different sections of strata beneath the shields, different maximum floor pressures, and different pontoon widths. However, the analysis presented indicates that the typical UCS of the first 1m of floor needs to be below 4 MPa before the issue of floor failure compromising shield effectiveness becomes a significant possibility.

It is noted that this analysis has not considered the extent by which the shields may tend to “plough” the floor material ahead of the pontoons as they are advanced. This relates to such considerations as the effectiveness of base lifting rams to lift the toe of the base clear of the floor and also whether the shields are being advanced up-dip or down-dip, the specifics of which are outside the scope of this report.

Yield Load Rating

The yield load rating of longwall shields, in terms of their supporting effectiveness, is simply covered by the statement “the bigger the better.” The justification for this statement is that the load rating design of a longwall shield cannot and should not be likened to that of roadway ground support, whereby support levels can be modified during mining according to the actual conditions encountered. With a longwall face, the only available roof support is that which is contained within the shield. Therefore, it should be

specified based on worst-case face loading and strata conditions. The problem from a design perspective is that those worst case loading and strata conditions are almost certainly undefined. Hence, it is not possible to then design the shields on that basis.

As a result, the shield needs to carry the highest practical level of available load so that, on those infrequent occasions when it is required, it is available for use. If longwall shields are designed according to typical face loading conditions and potentially down-rated for either cost or weight savings, the impact upon face stability and, hence, production levels could be many times more significant than any capital or shield weight benefits.

Does this then mean that all mines should simply default to the 2m wide, 1750 tonne Moranbah North shields described by Martin *et al* (2012)? The answer is “of course not” because these shields were clearly designed and procured based on both a history of major roof control difficulties at the mine and concern over the increasing cover depths. In other words, there were a number of geotechnical reasons as to why these shields were necessary in the first place.

The point being made is that the design of the yield load rating of longwall shields should not be based on a detailed technical assessment of typical or normal geotechnical conditions (by whatever means), but a risk-based assessment of both the general background conditions (to provide overall context as to the stability of the longwall face in general terms) in combination with the potential for locally adverse strata conditions where the roof fall potential is significantly elevated. How this may be done is not considered further in this paper.

Set:Yield Ratio

The history of shield development over the past 50 to 60 years points to the concept that, as well as the yield load rating being “the bigger the better,” the set:yield ratio should be “the higher the better.” The justification for this is that the longwall shield is essentially a reinforcing device for the immediate roof of the longwall face and, in the same way that roof bolts and tendons reinforce the roof of a mine roadway, it acts to clamp together what is typically a layered strata sequence immediately above the shield.

It is argued that the initial cause of guttering type failures of the immediate roof ahead of the shields is bending of the immediate roof strata around the face (see Figure 1). While the shield is rarely ever able to prevent such bending, it can help to minimise it. In fact Barczak 2006 makes the statement that field measurements have demonstrated that increased shield setting forces resulted in less face convergence, which, based on the longwall face model shown in Figure 1, would inevitably result in a more stable roof environment ahead of the shields. Therefore it is concluded that it makes no sense to simply allow such bending of the roof to compress the shield and so develop load in the legs. It is far better to apply such load actively back to the roof and limit the overall level of shield closure, hence roof bending about the face. This then contributes positively to roof stability ahead of the shields. It may also be the case that, by setting the shields to the highest possible or practical level and so reducing roof to floor closure, this acts to reduce face spall, which is again a positive aspect in terms of roof stability ahead of the shields.

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Trueman *et al* (2008) made the suggestion that, under certain face loading conditions, it may be preferable to set the shields to only 60% rather than say 90% of their yield loading. The following quote is taken from that paper:

Where periodic weighting is high enough to result in periodic shield overload, it may be better for set pressures to be nearer 60% of yield than 90% (with shields of support density of 100 tonnes per square meter or greater before the cut). This relates to the effect of time. If the support is set to 60% of yield then it will take much longer to the first yield event and for the same cycle time, there will be fewer yields. Fewer yields will result in less convergence and subsequent roof degradation and it will be easier to mine through the periods of support overload. If a shield periodically has an inadequate capacity for the conditions, the authors have seen no evidence that very high setting loads will stabilise the roof. The belief that very high set pressures are beneficial may have arisen when support capacities were less and set pressures close to the yield value were necessary for the set pressure to be adequate.

The underlying logic behind this statement seems to be that leg yield events allow increased rates of roof convergence, in which case the statement potentially makes sense. However, in all of the shield convergence monitoring studies across the coal industry reported by Frith and Stewart (1994), not once was the rate of shield closure seen to increase due to the legs going into yield. The rate may have remained constant after going into yield (which makes sense), but it never accelerated. Therefore, the idea that reducing the set pressures (assuming it could be done on an as-needs basis), thus allowing greater shield convergence to simply reduce the number of subsequent yield cycles, may not be well-founded, and it goes against the entire history of the development of longwall shields.

Fortunately, Trueman *et al* (2011) make the following statements. Operational controls can nevertheless be effective in minimising roof control issues in the presence of high level periodic weighting leading to support overload. Specific attention to achieving the highest set pressure practicable without compromising the attitude of the support canopy can reduce the extent of cavities and associated delays in many instances.

The author fully agrees with the latter statement because it appears that there is no obvious dissention to the universal concept of maximising the set:yield ratio of shields in order to increase their supporting effectiveness.

Flipper Arrangement

The flipper at the tip of a shield serves two distinct purposes:

- (i) It allows some measure of direct support to the upper coal face, which has both roof stability and also safety benefits.
- (ii) In the event that significant amounts of face spall occur, it can be used to offer confinement to the exposed roof ahead of the tips.

Both are legitimate functions, although, as discussed by Payne (2008), in order to function as a roof support ahead of the tips, the flipper needs to be double articulated, albeit where clearances allow. Otherwise, the situation shown in Figure 7 will occur,

whereby the flipper can only act to control the face and not the roof when the face has spalled significantly.



Figure 7. Inability of single articulated flippers to secure the face or roof due to previous face spall (Payne 2008).

The available load capacity of a flipper arrangement is generally small; however, in terms of acting to control a buckling/guttering type failure mechanism in the tip to face area, roadway roof support knowledge shows that relatively small vertical loads can prevent buckling under much higher levels of horizontal stress (Colwell and Frith 2010). Therefore, despite a low load rating, a flipper that can be positively set to the roof offers potential roof control advantages.

However, the limitations of a flipper are two-fold:

- (i) Like a shield, at some point, it needs to be retracted from the roof, during which time, if it were preventing roof buckling, guttering failure could then occur resulting in a small roof cavity, which immediately then acts to reduce over shield supporting effectiveness.
- (ii) Its reach is limited ahead of the tip. Therefore, as the amount of face spall increases, its overall effectiveness in terms of roof control decreases.

Overall, a double-articulated flipper is a useful addition to a longwall shield, particularly in high seams with friable roof. However, it only assists roof control and will not usually prevent large scale roof collapses that are driven by excessive face loading and associated face spall. It is certainly not a substitute for a well-designed and proportioned longwall panel layout and highly rated and well-designed longwall shield.

Inclination of Legs Towards the Face

The inclination of the shield legs has been a general shield design consideration since Peng (1990) first published the concept of an “active horizontal force” assisting roof stability ahead of the shield tips (see Figure 8).

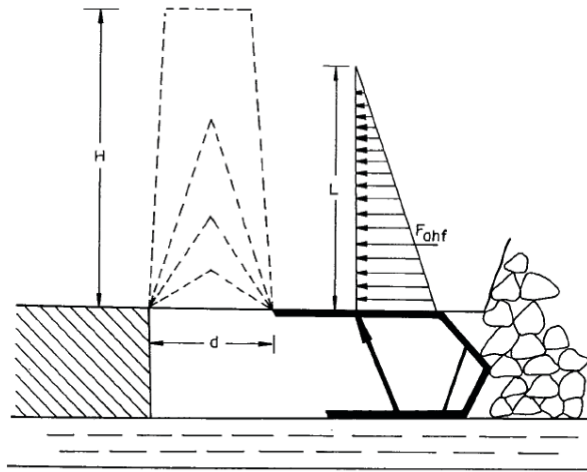


Figure 8. Active horizontal force concept (Peng 1990).

The author takes the view that the active horizontal force concept has little or no practical impact on shield effectiveness as a result of the following:

- (i) The active horizontal force is limited by friction between the canopy and roof (which is why Frith and Stewart (1994) undertook shear tests between steel and rock as mentioned earlier in the paper).
- (ii) Accepting that roof stability on a longwall face decreases as the face height increases, an increasing face height results in the legs becoming more rather than less vertical. Therefore, any active horizontal force decreases as a direct consequence. In other words, when it would be most required, it is least able to be generated.
- (iii) The model shown in Figure 1 has the immediate roof ahead of the shields initially failing due to horizontal stress-driven buckling. The horizontal stress being generated due to bending of the roof strata. Therefore, adding active horizontal force into this area due to the design of the shields can hardly be beneficial to roof stability in the tip to face distance.

As shown in Figure 3, the primary reason why 2 leg shields result in far better roof control than 4 leg shields (which was the context behind the development of the active horizontal force concept) is that they are more effective at (a) setting the canopy tip to the roof and (b) keeping the total leg load as close to the face as possible.

The primary concern with leg inclination is assessed to ensure that it does not result in a significant reduction in vertical force being applied to the roof. On the basis that the vertical force applied is equivalent to the leg load multiplied by the cosine of the leg inclination angle (to the vertical), a leg inclination of 20° only

results in a 6% reduction in the vertical component as compared to the absolute leg force. When it is remembered that the maximum leg inclination occurs with the shield at the lower end of its operating range, such a reduction is judged to be insignificant in terms of the overall roof instability threat.

As a general shield design concept, leg inclination should be maintained at no more than 20° to the vertical throughout the defined operating height range.

CONCLUSIONS

The objective has been to demonstrate that over and above geotechnical and panel layout design considerations, the effectiveness of a longwall shield is subject to a series of compromises that need to be well thought through as part of a shield design process.

It is hoped that following the publication of this paper, tip to face distance as illustrated on an OEM cross-section of the shield and face will never again be considered as a strata control concern. It solely relates to shearer collision concerns that change as a function of extraction height. This is not to say that tip to face distance is not a relevant strata control consideration, simply that it should be evaluated based on face loading and face stability issues, as well as how the shield sets to the roof. These subject areas are far more informative and relevant than simply an OEM cross-section of the shield and face.

The two most important aspects relate to canopy ratio and the inclination of the top caving shield during mining. Both are judged to be fundamental aspects of shield design that should never be compromised as part of improving the SLD of the shield or extending its upper operating height. Having a balanced canopy with all or most of the available leg loading acting through it is fundamental to the effectiveness of a longwall shield.

On a positive note, with all of the relevant shield design considerations defined and a good appreciation of the various geotechnical drivers of face line instability, there is no reason as to why a well-proportioned longwall shield cannot be developed that provides the maximum possible reinforcing effectiveness within the known geometrical face constraints. This then gives the longwall operators the best possible chance of managing the mining conditions through good operational practices. Given the critical role of operational controls, in the context of longwall shield design, this is the best possible outcome that can be achieved.

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Calibrating the LaModel Program for Shallow Cover Multiple-seam Mines

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ABSTRACT

The LaModel program (Heasley, 1998) is a displacement discontinuity, boundary element model (BEM) for analyzing stresses and displacements in single and multiple-seam coal mines. After the Crandall Canyon mine collapse, the need for better design guidelines for LaModel was observed and a new calibration method for deep cover pillar retreat was developed (Heasley et al., 2010). However, the scope of this initial calibration was limited to single-seam mines deeper than 750 ft. This limited application of the initial calibration method lead directly to the research presented in this paper, which attempts to define appropriate LaModel calibration guidelines for pillar design where shallow cover and/or multiple-seam mining scenarios are encountered. As part of this research, a database of shallow cover case histories has been developed, and this database is used to help validate the shallow cover calibration method. By adding the shallow cover database to the previous deep cover database, the LaModel calibration method has now been extended to the complete range of mining depths, as well as to multiple-seam cases. With this shallow cover calibration, the utility of the LaModel program continues to increase.

INTRODUCTION

The LaModel program (Heasley, 1998) is a displacement discontinuity, boundary element model (BEM) for analyzing stresses and displacements in single and multiple-seam coal mines. It utilizes a laminated overburden model that has been found to be fairly accurate for calculating stresses and displacements associated with horizontally bedded sedimentary rocks, as found in U.S. coal mining districts. Since its original development, the LaModel program has been continually upgraded over time with new features and modernized with the evolution of operating systems and programming languages (Heasley and Agioutantis, 2001; Heasley et al., 2003; Hardy and Heasley, 2006; Heasley et al., 2010; Sears and Heasley, 2009). Also, LaModel has always been provided free of charge and, originally, with little guidance for appropriate/accurate pillar design (Heasley, 2011).

Typically, when numerical analysis is to be used at a site-specific level, one needs to calibrate the model to the site-specific conditions. Once a model has been calibrated, future designs can be related back to the initial calibrated model. In contrast to

numerical modeling, empirical design relies on large databases of past observations in an attempt to create design criteria that minimize the likelihood of failure within the parameters of the given database. Both design approaches facilitate the design of an engineered system without fully understanding the mechanisms at work. The concept of developing pillar design criteria based on the statistical analysis of a database is not new (Salamon and Munro, 1967) and has been used for several decades with relatively good results. The most widely used empirical pillar design programs in the United States, ALPS and ARMPs (Mark and Chase, 1997; Mark, 2010), have essentially become design standards.

While both numerical modeling techniques and empirical design attempt to attain the result of suitable and stable pillar design, a difference of site-specific design vs. regional design has existed between the two. This research at shallow cover mines, along with the previous deep cover calibration (Heasley et al., 2010), creates a crossroads between the two design methods by using numerical modeling backed by a standardized calibration procedure in conjunction with a database of case histories.

BACKGROUND AND NEED

The fatal coal bump accidents at the Crandall Canyon Mine in August 2007 rapidly brought the insidious problem of coal bumps and pillar design back into the public spotlight. Moreover, what the public did not necessarily see was the underlying problems with inadequate pillar design at the mine, which partially stemmed from inaccurate calibration due to a lack of design guidelines to use with the LaModel program. Because of this need for better design guidelines, a new calibration method for deep cover pillar retreat was developed and implemented into the LaModel 3.0 program (Heasley et al., 2010). This calibrated method was demonstrated to have very good results with 47 deep cover case histories, where a stability factor (SF) of 1.40 or above showed a 10% chance of failure.

However, the scope of the validation of the calibration method was limited to single-seam mines deeper than 750 ft; therefore, the results were not necessarily appropriate for shallow cover and/or multiple-seam situations. However, during the development of the deep cover calibration method for LaModel 3.0, there was nothing

fundamental in the derivation that limited the method to just deep cover mines.

This limited validation of the calibration method led directly to the research presented in this paper, which will attempt to define appropriate LaModel calibration guidelines for pillar design where shallow cover and/or multiple-seam mining scenarios are encountered. While it is true that over time coal mining is being conducted at deeper depths, it is also true that more multiple-seam interaction scenarios are being encountered as mining matures in the United States. With time and the depletion of reserves, the depth of the remaining reserves and the number of horizons being mined simultaneously, or above or below historic workings, is ever increasing.

DATABASE OVERVIEW

In order to help evaluate and verify a “standardized” calibration method, a database of shallow cover (< 750 ft deep) retreat mining cases was developed. The database consisted of 38 pillar retreat case studies from 12 different mines. Eight of these mines are located in southern West Virginia, two are in central West Virginia, and two are in the Northern Appalachian coal fields. The overburden depths ranged from 195 ft to 700 ft with an average of 439 ft. The mining height at the case study sites ranged from 4 ft to 13 ft with an average of 7.8 ft. The number of entries in the sections ranged from 4 to 17 with an average of 7.9 entries. (This number might appear high because of the number of cases histories where pillars were split, which resulted in twice the original number of entries for these cases.) Pillar widths ranged from 30-ft to 85-ft centers, and crosscut spacing ranged from 50 ft to 110 ft centers with the average pillar size being 56 ft by 81 ft center-to-center. (Again, pillar widths might seem low due to the number of cases where pillars were split resulting in pillars that were half their original size.) The panel widths ranged from 215 ft to 570 ft with an average of 405 ft. Twenty-three of the case studies included loading from an active gob, while 15 of the panels were development loading only. Of the active retreat cases, nine of these had additional loading from one side gob, but none of them had loading from two side gobs. Eighteen of the case study sites were considered failures, and 20 were considered successful.

CALIBRATIONS AND LAMODEL ANALYSIS

The 38 case histories in the database were analyzed using the LaModel3.0 program. The majority (25) of these case histories allowed for using mine maps to create input grids based on the actual mine plan and creating overburden grids based on the actual overburden. Thirteen case histories required using an “idealized” analysis due to the fact that they were historic massive pillar collapses taken from the literature where no detailed mine map or overburden data was available. In these “idealized” cases, perfectly rectangular pillars and an average constant overburden depth were used in the LaModel analysis.

Once the mine grids and overburden grids were completed, the input parameters needed to be calibrated. Initially, a small subset of the database (10 case histories) was analyzed using three different trial calibration approaches. The first approach was to exactly follow the deep cover calibration procedure (Heasley et al., 2010). The second approach involved changing the rock mass modulus from 3,000,000 to 5,000,000 psi, thereby lowering the lamination

thickness and increasing the multiple-seam impact while keeping the same abutment extent. The final approach was to use a thinner lamination thickness (the default of 50 ft) to increase the multiple-seam impact while letting the abutment extent decrease. Figure 1 shows the effect on the abutment stress for an average model with calibrated rock mass moduli and for the default 50 ft lamination thickness. The model presented is created from an average of the parameters in the database for seam height, depth, pillar dimensions, etc.

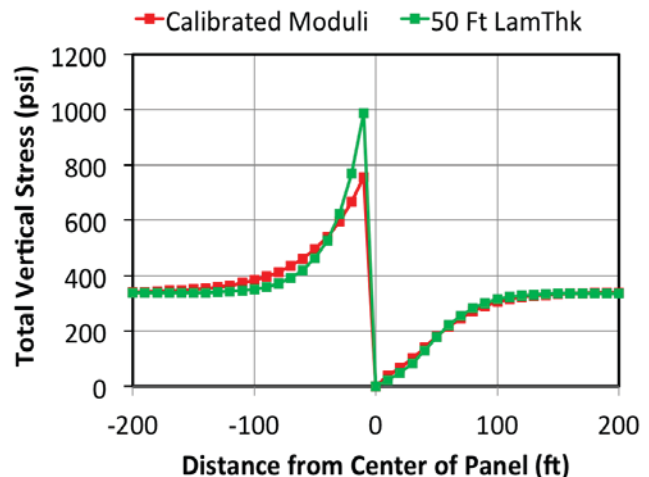


Figure 1. The abutment load for the average case by calibration method.

The deep cover calibration procedure produced maximum multiple-seam stresses in the 200 psi range. Increasing the rock mass modulus and decreasing the lamination thickness produced multiple-seam stress in the 400 psi range, while fixing the lamination thickness at 50 ft resulted in multiple-seam stresses in the 1400 psi range. We believed that the 200 to 400 psi multiple-seam stress value was very low, because in many situations, it appeared that the multiple-seam interactions were causing the observed failures. Therefore, it was decided to continue with the third approach and to hold the lamination thickness constant at 50 ft for the subsequent analysis of the database. Other than fixing the lamination thickness, the procedure used to calibrate the gob material and generate the coal pillar strength for the shallow cover database followed the same procedure as the deep cover database. Figure 2 shows the multiple seam stress calculated by using the deep cover calibration, changing the modulus to 6,000,000 psi, changing the modulus to 10,000,000 psi, and using the default 50 ft lamination thickness. The result for the average model shows that decreasing the stiffness of the rock mass increases the multiple seam stress from just a few psi for the deep cover calibration procedure to nearly 40 psi for the default lamination thickness from the front abutment load applied from an overlying seam.

DATABASE PARAMETERS

The most important variable in the database is the outcome: whether the shallow room-and-pillar section was successful or unsuccessful (Mark et al., 2007). The determination of success or failure was made during mine visits and in conversations with the mine staff, and it followed the same guidelines as used for

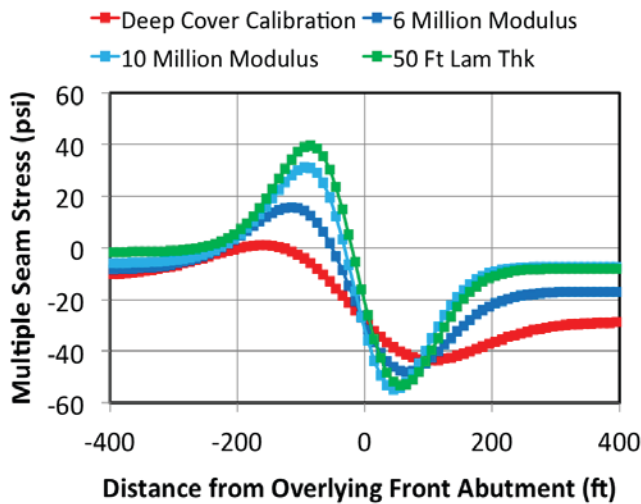


Figure 2. Multiple seam stress by calibration method.

the ARMPS and AMSS database. Essentially, a case study was considered a success when an entire panel was recovered without any significant ground control incidents, and a case study was considered a failure when the pillars collapsed or a ground failure occurred that necessitated leaving pillar or altering the original mine plan (Mark, 2009). In reviewing the database, it was noted that there were two separate populations of failures in the database. In one subset of the database, one or multiple pillars failed resulting in abandoning all or part of the panel. This is the pillar failure subset. In the other subset of the database, all or part of the panel was abandoned due to stress related issues with the roof, floor, or ribs; actual pillar failure did not occur. These cases histories are known as the entry failure subset.

In addition to dependent outcome variable, several other potential parameters were collected or extracted from the LaModel analysis. Independent geometric variables that were collected and considered for the statistical analysis included the following:

- Overburden depth
- Mining height
- Mining geometry (pillar sizes, gob widths, etc.)
- Coal Mine Roof Rating (CMRR)
- Interburden thickness

Independent variables that were calculated using LaModel included the following:

- LaModel stability factor (SF)
- Multiple-seam stress
- Total vertical stress
- Maximum roof compression
- Maximum roof tension

Some of the variables used in the AMSS database, such as the usage of supplemental support, were not available and therefore unable to be included in this database. Additionally, the small size of the shallow cover database precludes the use of too many independent variables in the model; therefore, all of the variables listed above were not ultimately used.

DATABASE ANALYSIS

The first step in analyzing the database was to calculate the LaModel stability factor of the pillar plan (Figure 3). For this database, the stability factor was calculated in the same manner as in the deep cover database (Heasley et al., 2010). Specifically, the stability factor was calculated as the element weighted average of the pillar safety factor over the Active Mining Zone (AMZ). Also, for the analysis of the LaModel pillar stability factor as shown in Figure 3, the failure cases were broken into two groups based on the observed failure mode: pillar failures or entry failures.

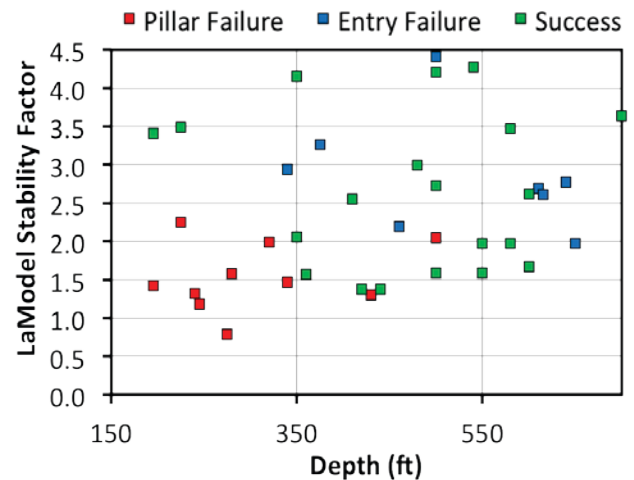


Figure 3. Depth versus LaModel stability factor.

Looking at the LaModel results in Figure 3, first, we see that the majority of the 10 pillar failure case histories cluster at the lower stability factors and lower depths. The pillar failure stability factors ranged from 0.79 to a high of 2.25, and the grey area where the failures overlap with the successful cases ranges from 1.30 to 2.25. With an average stability factor of 2.40, this translates into an uncertainty of about 40%. Looking at the entry failures, as might be expected, we see significant scatter of the stability factors, where the entry failures are occurring with a stability factor as high as approximately 4.5 and intermixed with the successful case histories in regard to the stability factor. Observing only the entry failures, it is clear that the pillar stability factor is not the most significant parameter in predicting success or failure for this subset of failures; however, on the other hand, the majority of the pillar failure cases appear to be readily explained by low stability factor.

This situation where the pillar stability factor was not the most significant independent variable was also encountered when the database for the AMSS program was analyzed (Mark et al., 2007). In the AMSS analysis, a combination of the independent parameters—total vertical stress, interburden thickness, CMRR, undermining vs. overmining, existence of supplementary support, and remnant pillar vs. gob-solid boundaries—was ultimately used to determine entry stability, after a minimum required pillar stability factor for the pillars had been met. The statistically analysis in this research followed a similar approach. First, a minimum allowable stability factor was determined to cover the pillar failure subset of the data. Then, a logistic regression of the

remaining independent variables was performed in an attempt to predict the entry failure subset of the data.

LOGISTIC REGRESSION

Logistic regression is the most common statistical technique for models where the dependent variable is binary (in this case, success or failure). Fortunately, for the geo-mechanical analysis of small databases, the assumptions of linear regression, such as normality, constant variance, etc., are not required for logistic regression analysis. The basic assumption of logistic regression is that the continuous independent variables must be “linear in the logit.” That is, the independent variable must be linear with respect to log-odds or the logistic equation (Mark et al, 2007).

Single Variable Pillar Failure Subset

First, the pillar failure subset of the data was analyzed. To accomplish this, two models were performed. First, the single variable (SF) model using the LaModel SF was created and the results are presented in Table 1.

Table 1. Logistic regression table for the single variable (SF) model.

Summary Table				
Coefficients:	Estimate	Std. Error	Z Value	P Value
Intercept	-3.631	1.783	-2.037	0.0417
LaModel SF	2.243	1.006	2.229	0.0258

Based on the estimates of the coefficients in Table 1, the model equation can be written as

$$g(x) = 2.243SF - 3.631 \quad (1)$$

and solving Equation 1 for the critical SF results in

$$SF = 1.62 \quad (2)$$

A disadvantage of logistic regression is the lack of a well-established measure of model fit (Mark et al, 2007). Most individuals are aware of the R^2 parameter in least squares regression analysis, where a value such as 0.90 implies that the model explains 90% of the variability in the data. Unfortunately, logistic regression does not have an equivalent parameter. Mark et al. (2007), as recommended by Hosmer and Lemeshow (2000), uses the receiver operating characteristic (ROC) curve as an alternate to an R^2 parameter. The ROC curve plots the sensitivity vs. 1-specificity for the entire range of cut points, where the cut point is defined as the probability which separates the predicted values into the given categories. In this case, the predicted values will be classified as either successful (higher than the cut point) or failures (lower than the cut point). The default cut point used in R (R Development Core Team, 2011), and most statistical packages, is 0.5. By changing the cut point, the corresponding intercept of the logistic model is adjusted to move the design line in relation to the desired classification. Here, the sensitivity represents the proportion of correctly classified successes and the specificity represents

the proportion of correctly classified failures. This means that 1-specificity represents the proportion of incorrectly classified failures. The goal of the model is to classify as many successes as possible while minimizing the chance of incorrectly classifying a failure. This means that the (0,1) coordinate on the ROC curve implies perfect discrimination and an area under the curve (AUC) of 1. Goodness of fit between logistic regression models can then be compared by the AUC of the ROC curve, which ranges from 0.5 (no discrimination or same as chance) and 1 (perfect discrimination). The area under the ROC curve for Equation 1 (see Figure 4) is 0.84 implying excellent discrimination.

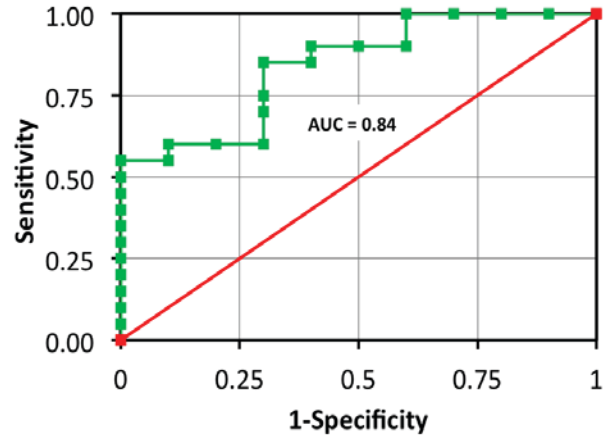


Figure 4. ROC for equation 1.

Further, the logistic regression model’s optimum cut-point can be determined from plotting both the sensitivity and specificity vs. the cut point (Figure 5). Each point on the curve represents either the sensitivity (correctly classified successes) or specificity (correctly classified failures) at each individual cut point. As the cut point increases, the probability of correctly classifying a successful case decreases while the probability of successfully classifying a failure increases. Or in logistic regression terminology, the sensitivity decreases and the specificity increases. The intersection of these two curves maximizes the overall classification accuracy resulting in the optimum cut-point of $p = 0.5$ to 0.6 (Figure 5).

The original model assumes a cut-point of 0.5; therefore, no modification of the intercept in Equation 1 is required. Table 2 shows the percent of correct classifications for the optimum cut-point.

From Table 2, we see that if we use the optimum cut point of 0.5 and the resulting design SF of 1.62, the model correctly predicts 70% of the failure cases. Of less importance, because we are primarily designing to prevent failure, the model also correctly predicts 85% of the successes and 80% of the overall cases.

To improve the classification of the failure cases to 90%, the cut point must be changed to a value between 0.7 and 0.8 as seen in the specificity curve in Figure 5. The QuantPhyc package was used to determine that a cut point of 0.725 was necessary to obtain the desired 90% failure classification rate while retaining the highest

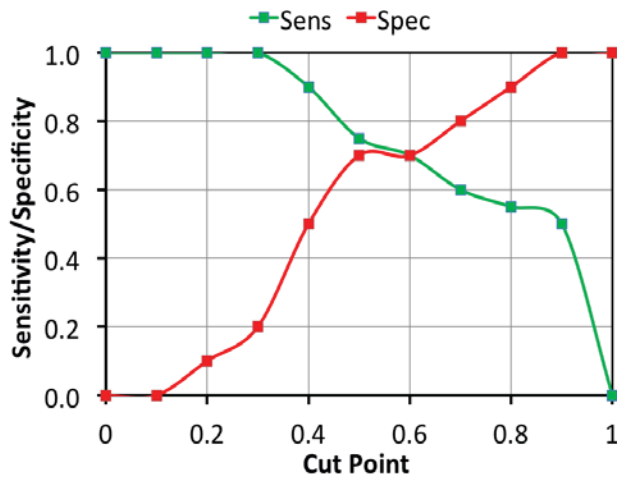


Figure 5. Plot of specificity and sensitivity vs cutpoint for equation 1 showing the optimum cut point to be between 0.5 and 0.6.

Table 2. Performance of the design equation against the database at a cut point of 0.5.

	# Correct	% Correct
Successes	17	85%
Failures	7	70%
Overall	24	80%

classification of successes. Using this new cut point modifies the intercept in the model equation to

$$g(x) = 2.243SF - 4.6 \quad (3)$$

Solving Equation 3 for the critical SF results in

$$SF = 2.05 \quad (4)$$

Table 3 shows the percent of correct classifications for the cut-point of 0.725.

Table 3. Performance of the design equation against the database at a cut point of 0.725.

	# Correct	% Correct
Successes	12	60%
Failures	9	90%
Overall	21	70%

Here we see that if we use a cut-point of 0.725 and the resulting design SF of 2.05, the model correctly predicts 90% of the failure cases. But the accuracy of the model for the successful cases drops to 60% and the overall accuracy drops to 70%. The design guidelines shown with the pillar failure subset of the data for Equations 2 and 4 can be seen in Figure 6.

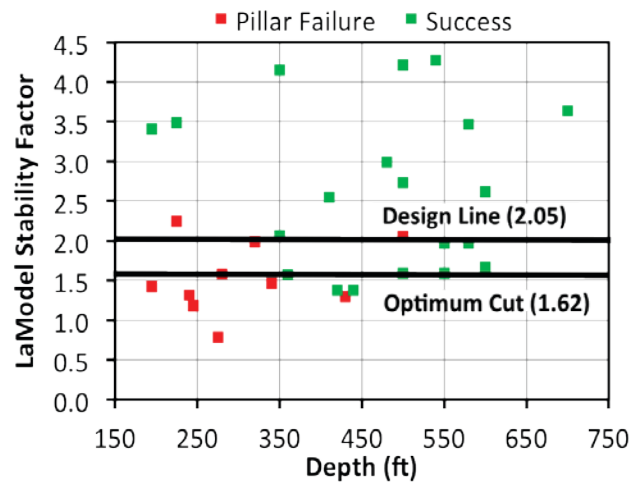


Figure 6. Design guidelines for the 70% correction failure classification (SF = 1.62) and 90% failure classification (SF = 2.05).

Multi-Variable Pillar Failure Subset

The second model that was analyzed for the pillar failure subset of the data also considers the depth (which was the only other independent parameter to be determined as statistically significant from running numerous models). Table 4 shows logistic regression table for the multi-variable model with both the LaModel SF and the depth.

Table 4. Logistic regression table for the multi-variable (SF and H) model.

Summary Table				
Coefficients:	Estimate	Std. Error	Z Value	P Value
Intercept	-10.306	4.205	-2.451	0.014
LaModel SF	2.554	1.161	2.200	0.028
Depth (H)	0.016	0.007	2.199	0.028

Based on the estimates of the coefficients in the above table, the model equation can be written as:

$$g(x) = 2.554SF + 0.0 \quad (5)$$

and solving Equation 5 for the critical SF results in:

$$SF = 4.035 - 0.006H \quad (6)$$

The area under the ROC curve for Equation 5 (See Figure 7) is 0.95, implying near perfect discrimination.

The multi-variable model's optimum cut-point can be determined from plotting the sensitivity and specificity vs. the cut point just like with the single variable model (Figure 8).

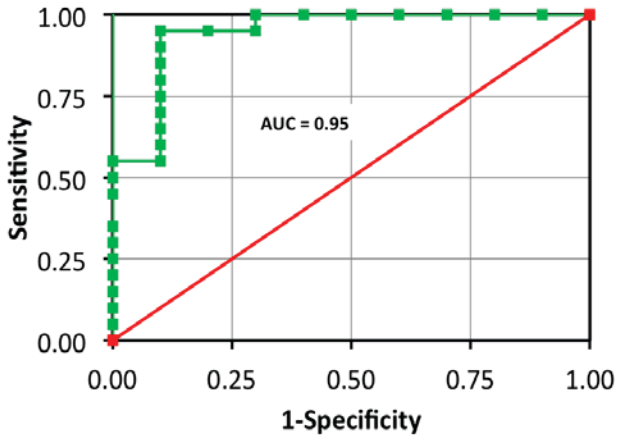


Figure 7. ROC curve for equation 5.

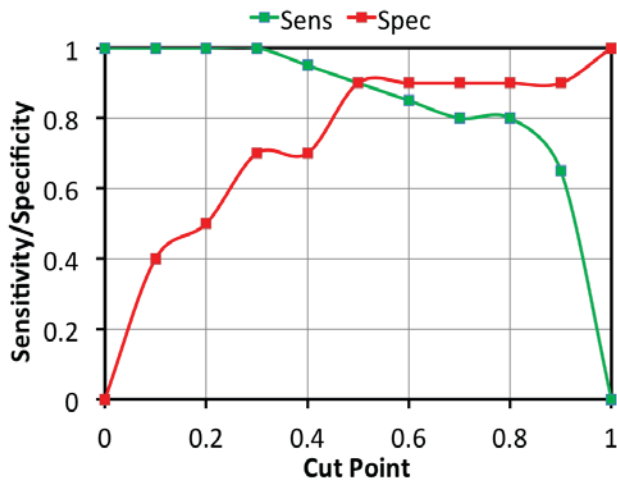


Figure 8. Plot of specificity and sensitivity vs cut-point for equation 5 showing the optimum cut-point to be 0.5.

Again, because the model assumes a cut-point of 0.5, no modification of Equation 5 is required. Table 5 shows the percentage of correct classifications for the optimum cut-point when considering both the LaModel SF and the depth.

Table 5. Performance of the multi-variable model against the database at the optimum cut-point.

	# Correct	% Correct
Successes	18	90%
Failures	9	90%
Overall	27	90%

In this table, we see that with the optimum cut point of 0.5 and the resulting design equation (5), the model correctly predicts 90% of the failure cases, 90% of the successful cases, and 90% of the overall cases. Since the optimum cut-point already correctly classifies 90% of the cases, there is no need to further modify the

cut point to obtain a better classification of the failure cases as was done with the single variable model.

The design guidelines shown with the pillar failure subset of the data for Equation 5 can be seen in Figure 9. Here, engineering judgment determined that there should be some lower bound to the stability factor in the design guidelines, hence stopping the downward trend at the deep cover pillar retreat guideline of a SF of 1.4 (Heasley et al., 2010).

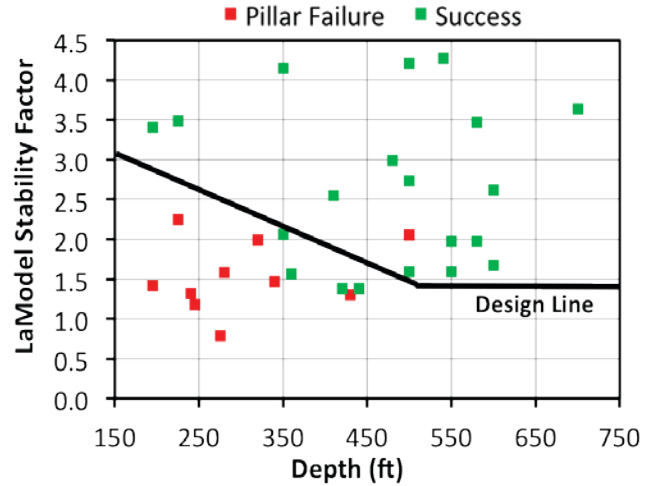


Figure 9. Design guideline for 90% correction failure classification for equation 5.

Entry Failure Subset

Next, the entry stability failure subset of the data was analyzed. Because this subset of the data is treated as a distinct population and a separate statistical analysis is being preformed, cases within this subset where the SF was found to be less than the recommended SF required removal from the database. These cases were removed from the database to eliminate the possibility of attempting to model a sample of the population where the entry stability failure was the result of inadequately designed pillars. These cases would be outside the population of this subset of the database and actually belong in the pillar failure subset, but where a determination of pillar failure was not originally obtainable. This required the removal of the two case studies, which brought the total number of remaining cases in the database to 38.

A host of initial models were used to determine which few of the many possible independent variables contributed the most to the success or failure of the entry failure subset of the database. Ultimately, the two variables that were found to be statistically significant were the multiple-seam stress (MSS) and the coal mine roof rating (CMRR). Due to the nature of the small database, the multiple-seam stress and CMRR were highly correlated, and adding both variables into the model was of minimal value. Therefore, the most statistically significant variable, the multiple-seam stress, was chosen (Figure 10).

From reviewing Figure 10, it is fairly clear there is no clear-cut separation of the successes and failures as a function of the multiple-seam stress and the CMRR. What we see is that most of

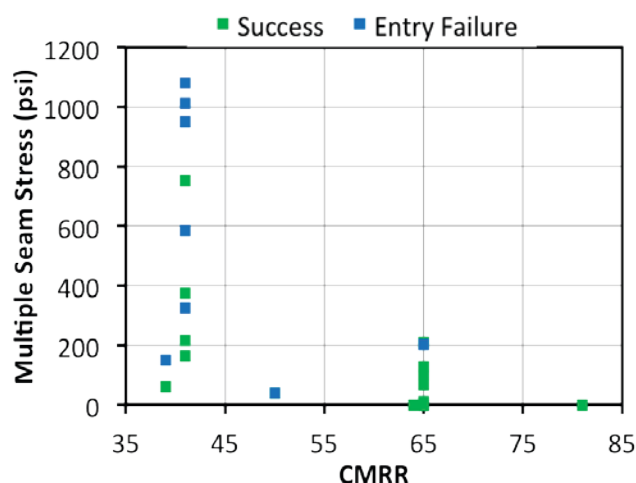


Figure 10. CMRR vs multiple seam stress.

the cases have one of two CMRRs, a low value of about 40 and a high value of about 65. This results in the observed clustering at these CMRR values and the correlation between the CMRR and multiple-seam stress. This database behavior is a result of how the CMRR values were originally determined. Since the authors were not able to view each individual case study site and determine a site specific CMRR, the CMRR values were approximated by using the average CMRR value for the given coal seam in that area. Certainly, the site-specific CMRR value may have improved the accuracy of the database, as opposed to a regional average CMRR value.

The authors believe that the entry failures in the field are undoubtedly controlled by some combination of geology, applied stress, and roof support; however, the small database used in this study along with a limited range and accuracy of the recorded CMRR does not allow any statistical inference to be made at this time.

CONCLUSIONS

Several conclusions can be drawn from the analysis of the shallow cover database reported in this paper. First, when the cover becomes shallow, it is difficult to find many failure instances unless massive pillar collapses and multiple-seam case histories are included in the database. Of course these types of failures add complexities to the database beyond the basic stability of the section pillars.

The massive pillar collapses show up in the database as pillar failures at very low depths (<350ft) and with very thin pillars (w/h < 3). With these narrow pillar widths, any spalling of the pillar or any geologic anomaly in the pillar can greatly reduce its strength; therefore, it seems reasonable to require a slightly higher design safety factor to guarantee section stability. This same situation was found in the analysis of massive pillar collapses by Mark et al. (1997), where they recommend an ARMPS stability of 2.0 to prevent the pillar collapses versus a “normal” recommended ARMPS stability factor of 1.5. In the analysis of the shallow cover database in this paper, it appears that a stability factor of 2.0 should be used up to a depth of about 350 ft and then the recommended

deep cover stability factor of 1.4 appears suitable for all greater depths of cover.

The multiple-seam case histories add the complexity of failures due solely to entry stability where the pillars are stable. Multiple-seam stress impacts frequently affect only a relatively small area and do not necessarily affect the overall pillar stability of the section. However, in the multiple-seam area of influence, the stress affects can cause entry and section failure. Intuitively, the magnitude of the multiple-seam effects on the entry stability should be a function of the applied stress, the geology and the roof support. This was certainly the case found in the database analyzed for the AMSS program (Mark et al., 2007). However, the small database used in this study along with a limited range and accuracy of the recorded CMRR does not allow any statistical inference to be made about the multiple-seam stress or the CMRR for the LaModel shallow cover database at this time.

SUGGESTIONS FOR FUTURE RESEARCH

Just as it is with all research studies, there is always room for improvement. First, there is obviously room for expansion of the database. Increasing the sample size and increasing the range of the database should allow for more accurate design guidelines and include a wider range of actual mine conditions. Additionally some of the suggestions that resulted from the deep cover database analysis can also be applied to this shallow cover analysis, such as determining a better approximation for the abutment loading and exploring a strain-softening coal model. Recent research by Tulu and Heasley (2012) in the area of abutment loading could prove to increase accuracy in analyzing the database. This work only considered elastic-plastic coal elements and ignored the possibility that strain-softening coal elements might provide a more accurate model for the database. Planned improvements to the LaModel program will result in a wizard that will make using strain-softening coal properties much easier to use. This could result in getting better approximations of pillar failures and again improving the database analysis. The lamination thickness controlling rock mass stiffness was held to a constant 50 ft. Certainly other constant lamination thicknesses could be tried, or the lamination thickness might be varied with depth with a revised calibration model. Initially, a softer overburden was desired in order to get better multiple-seam stresses for modeling the entry failure case histories. Because the database was not able to discriminate the entry failure case histories, the deep cover calibrated lamination thickness might be acceptable and thereby allow the deep cover and shallow cover databases to be combined.

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Enlightening Bolts: Using Distributed Optical Sensing to Measure the Strain Profile along Fully Grouted Rock Bolts

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INTRODUCTION

Over the last 30 years, a transition away from end-anchored bolts to fully grouted rock bolt systems has occurred in many underground mines. It seems geomechanically intuitive that the success of such bolts, which interfere with rock movements around underground excavations through a variety of complex interactions, is precisely why they remain challenging to instrument and monitor. Whereas measuring the load profile for an end-anchored mechanical bolt is trivial, for a fully grouted system, the load may vary dramatically due to discrete rock movement at cracks and joints.

Despite the challenges, measuring the strain (i.e., load) along fully grouted rock bolts remains very appealing, and a lot of valuable research has been conducted using foil resistance strain gauges, particularly in US coal mines (Signer and Jones, 1990). Despite good intentions, more widespread adoption of this instrumentation remains unlikely for the following reasons:

- (i) The technology is expensive, partly because Intrinsically Safe (IS) requirements limit the number of suppliers and solutions.
- (ii) The technology is perceived as challenging to successfully implement except by those who are skilled in the art.
- (iii) The reliability of the technology is questionable in harsh environments.
- (iv) The technology relies on discrete pairs of strain gauges that may not capture the complete strain distribution along the bolt.

No existing alternative technology can offer a solution to all of these limitations. However, in this paper, we introduce results from a new Distributed Optical Sensing technology that promises significant potential. Since the technology is capable of distributed sensing, our results suggest that this new technology can, for the first time, reveal the detailed strain profile along fully grouted rock bolts.

SOURCES OF STRAIN IN FULLY GROUTED ROCK BOLTS

The strain developed in a rock bolt results from a superposition of the torque applied to the bolt during installation (Active Load) and the load induced by ground movements during the life cycle of the bolt (Passive Load). It is important that the new technology be able to monitor the complete range of loading modes.

Active Bolt Loading

Almost all rock bolts are installed by a rock drill that secures the bolt into the hole by applying torsion to the bolt. This torque may be used to (i) simply mix a single speed resin cartridge, in which case it may be dissipated, (ii) tighten an expansion shell to provide tension while the resin sets (Double Lock), or (iii) mix a fast resin cartridge at the toe and then torque the bolt to apply tension to the remainder of the bolt (Torque Lock). For (ii) and (iii), a proportion of the applied torque will reside in the bolt and may remain throughout the bolt lifecycle. The corresponding axial load associated with the applied torque during installation is a function of the tension-torque ratio, which depends on factors such as the specifications and properties of the bolt, the plate and the nut assembly, and whether or not they are properly lubricated (Osen and Parsons, 1966).

Due to their high length/diameter ratio, rock bolts are prone to bending during installation. Whereas end anchored rock bolts can bend throughout the life-cycle of the bolt, for fully grouted bolts, it is restricted after the resin has set. However, bending during installation is common for all bolts; in fact, even when appropriate hardware is used, it is almost ubiquitous for bolts that are installed in holes obliquely oriented to the excavation surface.

Passive Bolt Loading Caused by Rock Movement

After installation, the increases in rock bolt loads result from movements in the rock mass that are transferred to the rock bolt through the grout annulus. Windsor (2004) recognized that load transfer involves the following three mechanisms:

- (i) rock movement, which induces load transfer from the unstable rock to the reinforcing element

- (ii) transfer of load through the reinforcing element from the unstable zone to a stable zone
- (iii) transfer of the reinforcing element load to the stable rock mass

Depending on the rock mass characteristics and the stress state around the excavation, rock movements may be either distributed or discrete. It is well documented in the literature (Freeman, 1978; Aydan, 1989; Hyett, Moosavi, and Bawden, 1994; Li and Stillborg, 1998) that each induces radically different strain profiles along a fully grouted bolt.

If these rock movements are coaxial with the bolt, axial loading results. For each increment of rock movement, the corresponding amount of load transferred to the bolt depends primarily on the stiffness of the grout annulus, which, in turn, depends on the material properties of the grout and the mechanical interlock between the grout and the profile of the deformed rebar. If the rock movements are transverse to the rock bolt, shear loading results. In blocky or stratified rock masses, shear deformations localized at joints and bedding planes induce a “dowel” response from the rock bolt (Ferrero, 1995). Numerical simulations (Jalalifar, 2006) suggest that the resultant strain profile may be highly variable.

Most commonly, rock bolts will be subjected to a combination of axial and shear loading, and, during the life cycle of the excavation, rock movements may transition from distributed to discrete as cracks and shear zones localize. Consequently, accurately monitoring the strain profile of fully grouted rock bolts could reveal as much about rock mass behavior around an excavation as about the rock bolts themselves.

DISTRIBUTED OPTICAL SENSING

Several methods are currently available for distributed strain sensing using optical fiber. These include techniques based on Raman, Brillouin, and Rayleigh scattering, as well as those involving multiplexed fiber Bragg gratings (FBGs). Techniques based on Brillouin scattering typically employ optical time domain reflectometry and are, thus, limited in spatial resolution to 0.1 to 1 m. FBG methods (optically analogous to strain gauges) are often limited by the number of gratings that can be multiplexed in a single fiber. Both FBG-based and scatter-based techniques also often require specialty optical fiber.

This paper introduces an emerging optical technology based on Rayleigh backscattering. This solution is attractive for monitoring fully grouted rock-bolts because of the following:

- (i) The instruments are inherently IS certifiable and the readout unit can be set back 50m in fresh air.
- (ii) The distributed strain profile using the Rayleigh technique is measured with a spatial resolution in the order of 5mm.
- (iii) Its operational accuracy is perfectly acceptable (better than +/- 10 microstrain).
- (iv) It is based on low-cost standard telecom fiber. The cost of the analyzer has dropped significantly in recent years.

Test Results

The test data presented below is designed to demonstrate the capabilities of the new technology with respect to the three modes

of bolt deformation: bending, axial loading, and shear loading. It represents a due-diligence evaluation prior to underground field trials. The tests were conducted using a #6 grade 60 rebar supplied by Jennmar Canada Ltd. A pair of diametrically opposed grooves, 3mm wide x 3mm deep, was machined into the rebar (Figure 1). The strength of the modified bar was 116.1kN (yield) and 185.3kN (tensile). The fiber is only 100 microns in diameter so the possibility of smaller grooves exists. A 3.2m long optical fiber was positioned within the groove and encapsulated with a high strength epoxy resin. At the toe of the bolt (2.1m), the fiber was looped to enable monitoring of both grooves.



Figure 1. The diametrically opposed grooves cut in the rebar samples.

Bending Tests

In view of the very low loads (<1kN) required, bending represents the most controlled method for establishing the baseline accuracy of the new distributed optical sensing technique. Two types of test were conducted: (i) Symmetric Simply Supported Beam bending test, and (ii) Cantilever Beam bending test.

The setup and results are shown in Figures 2 and 3. A single length of optical fiber was looped around so that both sides of the rebar are monitored; one side exhibits contraction(-ve) and the other extension(+ve). The lower graphs in Figures 2 and 3 are the corresponding theoretical simulations based on Euler-Bernoulli beam theory. The comparison is within +/- 5%, but, more remarkably, the shape of the two plots is essentially identical. This is important because, for fully grouted rebar, the shape of the strain profile could be more informative than the absolute strain magnitudes.

Axial Pull Tests

The setup, shown in Figure 4, was used to assess the capabilities of the technology to monitor axial deformation. The instrumented bolt was resin grouted into a 31mm reamed hole created in a concrete (40MPa mix) block. The embedment length was 200mm. Axial load was applied to the rebar at a distance of 900mm from the embedded section.

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One general observation, when applying axial load to instrumented bolts, is that it was almost impossible to obtain perfect axial loading without some component of bending. At low axial loads, bending is associated with initial straightening of the rebar, which is, without exception, imperfectly straight. Slight realignments of the sample within the test frame resulted in quite different patterns of bending. In Figure 4, the effect of bending can

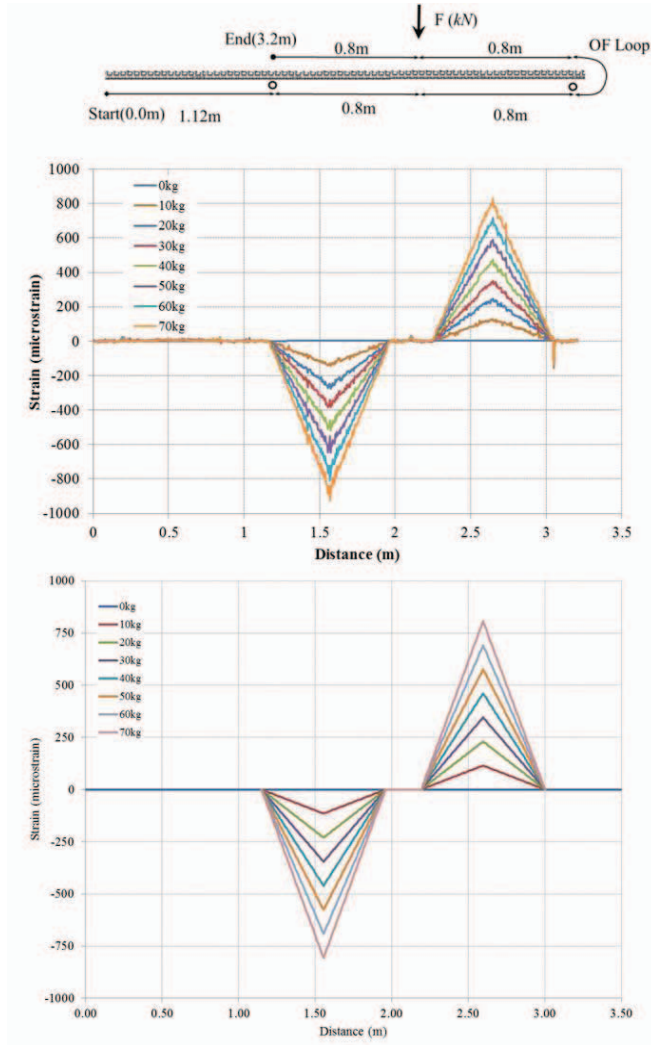


Figure 2. Simply supported beam test. Upper: Test configuration. Middle: Experimental results. Lower: Theoretical results (extension is +ve). 20 kg corresponds to a force of 196N.

clearly be seen by comparing the response of the fiber along the free lengths on opposing sides of the rebar (mirror image between 0.9-1.8m and 2.4-3.3m). These variations derive from limitations with the test setup rather than errors in the instrumentation technology. Our experimental experience strongly suggests that calibration of instrumented rebar may be more accurately accomplished under bending rather than axial load.

Figure 5 shows the detailed strain profile along the section grouted within the concrete block. As expected, the strain decays according to an exponential form predicted by theory (Farmer, 1975; Serbousek and Signer, 1987; Aydan, 1989). However, periodic disturbances in the profile are exhibited that seem to

correspond with the spacing of the rebar ribs. Such an effect was simulated by Jalalifar (2006) using Finite Element modeling (Figure 6).

Figure 7 shows the displacement profile corresponding to the upper plot in Figure 5. At a given point ($x=l$) along the bar, this

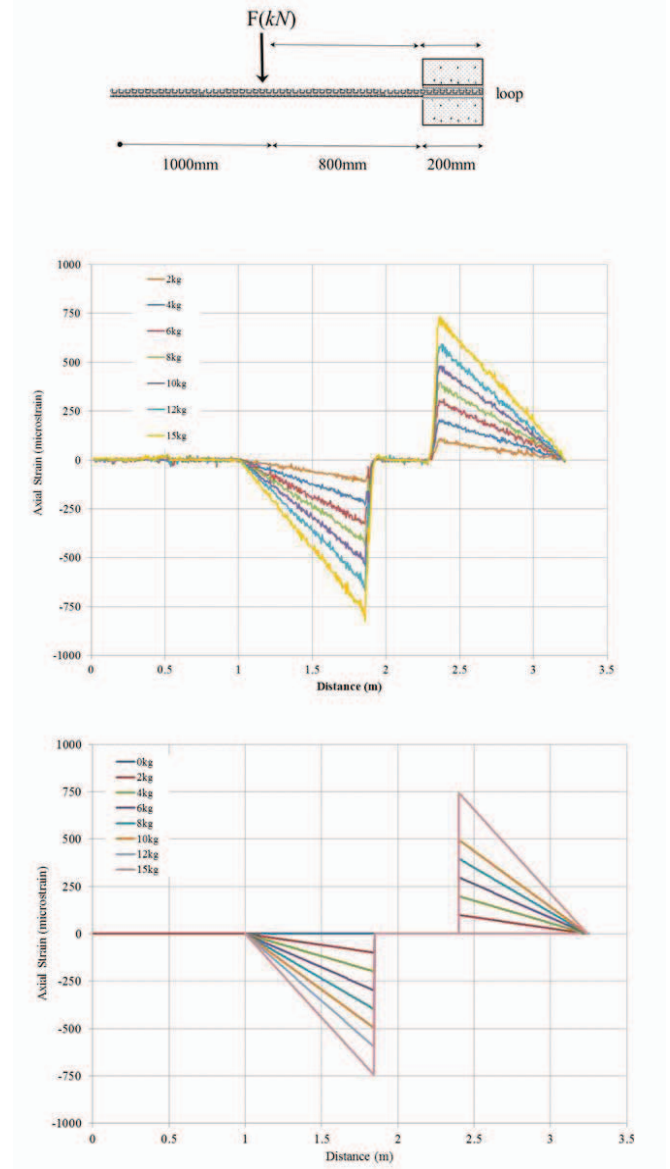


Figure 3. Cantilever beam test. Upper: Configuration. Middle: Experimental results. Lower: Theoretical results.

is calculated from the summation of the product of strain and measurement spacing (L_e):

$$U_{x=l} = \sum_{x=0}^{x=l} \epsilon_x \cdot L_e$$

The displacement distribution is exponential in form as predicted by the literature

Shear Tests—Double Shear Configuration

Shear behavior of the bolt was assessed using double shear tests in which an instrumented rebar was grouted into 3 blocks with pre-formed and reamed boreholes (31mm in diameter). The end of

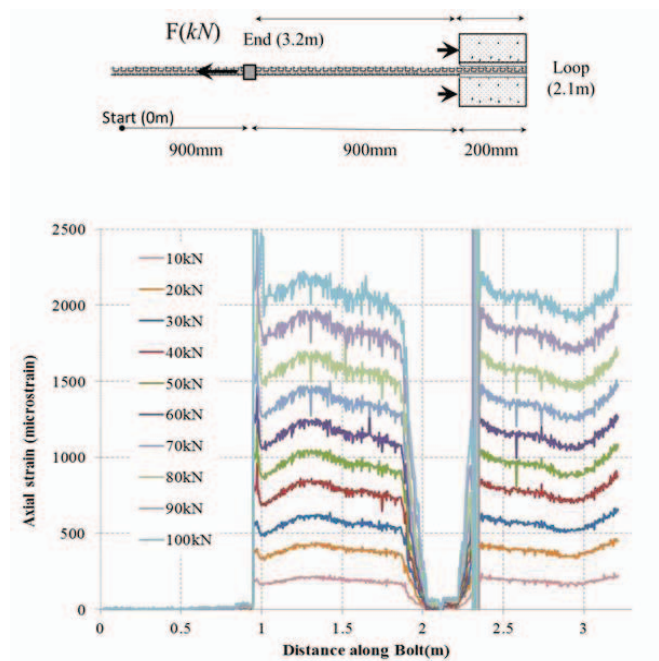


Figure 4. Axial pull test results. Upper: Configuration. Lower: strain profile along whole fiber length.

the rebar was un-tensioned. The configuration is shown in Figure 8 (upper). Dimensions relate to the position along the optical fiber. Load was applied to the sample using a test frame, and the displacement of the central block relative to the end blocks was monitored (Figure 9). The corresponding strain profiles along the rebar are shown in Figure 8 (middle and lower). A distinctive shear couplet across the discontinuity is measured, similar to that which was predicted by Finite Element modeling (Figure 10), conducted by Jalalifar (2006).

DISCUSSION

The laboratory results presented in this paper reveal, for the first time, the complexity related to load transfer along fully grouted rebar subjected to bending, axial loading, and shear deformation. Due to the high stiffness of resin grouted rebar, the loads can vary rapidly along the bolt length.

Perhaps the most “enlightening” application of the results is to provide a rationale for interpreting the results from previously used discrete monitoring solutions based on strain gauges.

Foil resistance strain gauges

As indicated by the results shown in Figures 4 and 8, a typical array of 10mm-long strain gauges with a 200mm gauge spacing may completely miss the initial strain development at a discrete

fracture. Furthermore, it may be completely impossible to relate the measured strain profile along the bolt to the rock-mass structure around an excavation. It is entirely possible that what is often measured, as highly variable strain profile might well be reliable data, which is often dismissed as instrumentation scatter. The best chance of gaining acceptable results using standard strain

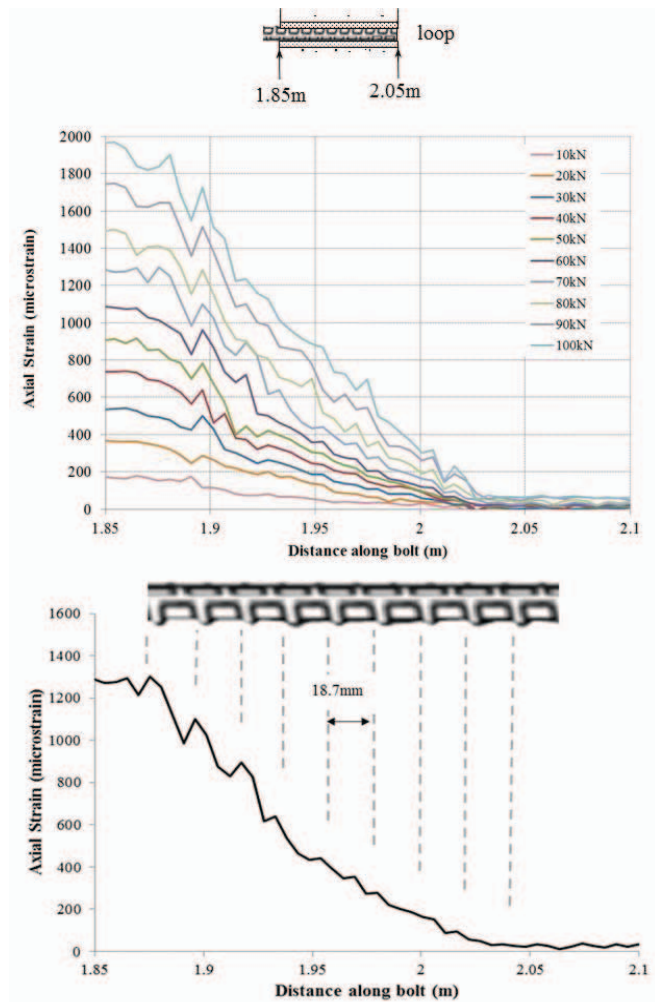


Figure 5. Detailed axial strain profile for the bonded section of the bolt (1.85m to 2.05m along the fiber). Upper: Change in strain profiles with the applied axial load. Lower: Detailed variation of axial strain (at 70kN) related to rib geometry.

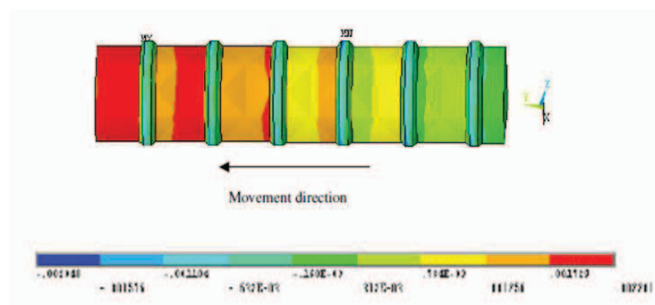


Figure 6. Simulation of the strain profile cause by ribbing using ANSYS FE program by Jalalifar(2006).

technology is when the rock mass behaves as a continuum: This is hardly surprising since the technology was originally designed for such problems.

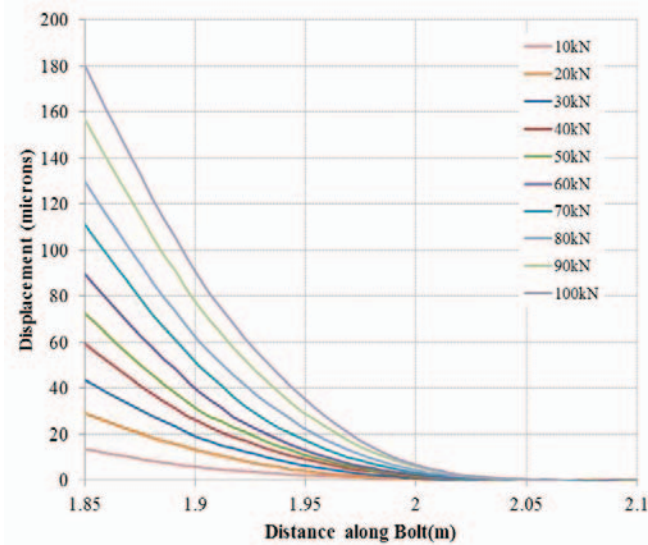


Figure 7. Experimental displacement profile based on integration of the strain profile in Figure 5(upper). Note: These are experimental results.

Long base-length strain gauges

The long base strain gauge monitoring system introduced by Spearing *et al.* (2012) represents an admission that the detailed strain distribution cannot be reliably or cost-effectively measured using electrical methods, and that, from an operational perspective, an average assessment of the bolt behavior will often be more informative. The results in Figure 8 confirm that long base-length strain gauges will effectively cancel out the shear couplet caused due to shear, and, therefore, these instruments can only provide insight into the axial response of the bolt.

If mines can be encouraged to embrace rock bolt monitoring as a means to improving excavation design, we suggest that measuring the strain profile should be the priority rather than accurately monitoring localized strains. Furthermore, since excavation geometries are 3-dimensional, the variation of the strain profile within a cluster of bolts will usually be more informative than the result for any single bolt. Consequently, the cost per instrumented bolt needs to be minimized, which is a feature of the technology introduced above, which is based on plain fiber.

As a final point, since the rock structure and deformation mechanisms are unknown a priori, distributed optical sensing technologies represent a perfect solution for geomechanical monitoring. However, if this technology is to be successfully applied, then protecting the optical fiber by embedding it with support elements undoubtedly provides the most promising deployment option. Thus, instrumented steel elements (i.e., rock bolts and cables) that are capable of distributed optical sensing should be regarded as an enhanced tool for the rock mass, as well as monitoring rock bolts themselves.

CONCLUSIONS

Embedment of fiber optic sensors with steel elements such as rock bolts represent the best opportunity to deploy distributed

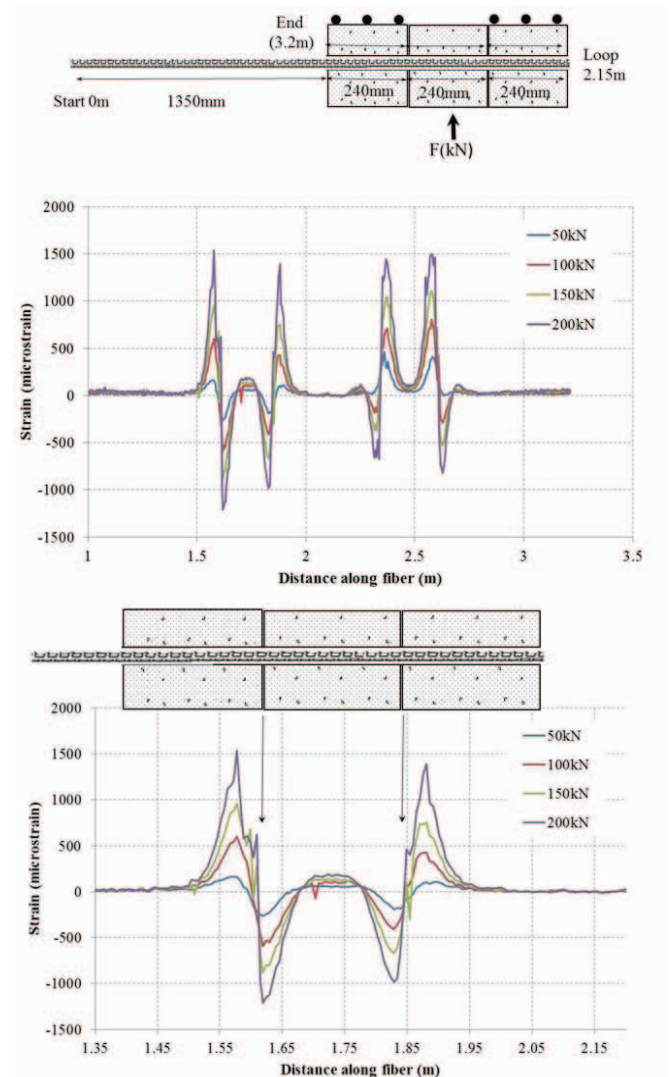


Figure 8. Results from the double shear test. The applied load in KN corresponds to the load-displacement profile in Figure 9. Upper: configuration. Middle: Strain profile along whole fiber. Lower: detailed Strain profile along a detailed section of fiber.

sensing in geomechanics. This paper has evaluated technology based on Rayleigh backscattering that has a spatial resolution of approximately 5mm. For both 3 point bending tests and cantilever bending tests, the measured strain profiles deviated by no more than 10% and were typically within 5% of the results predicted by theory. For short embedment length, axial pull tests the results agreed well with theoretical predictions from the literature and even the effect of the deformed ribs on the rebar could be discerned. Likewise, the strain profiles measured for a simple three block shear test were qualitatively similar to those predicted by theory and numerical modeling.

This test program highlights the potential of the technology for future field trials.

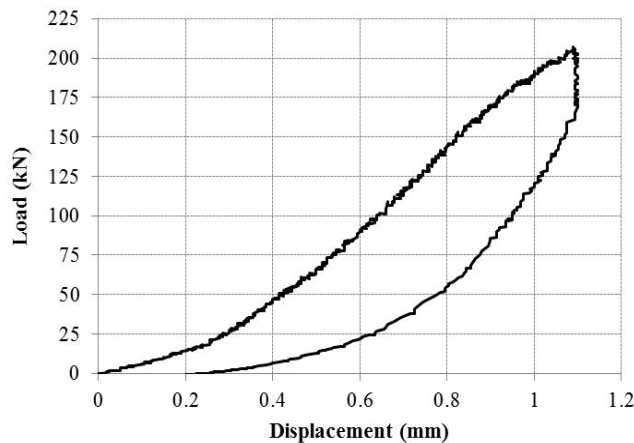


Figure 9. The load-deformation response of the Double Shear specimen. The load(kN) was applied to lower surface of the central block. Displacement(mm) is the relative movement between the central block and the outer blocks.

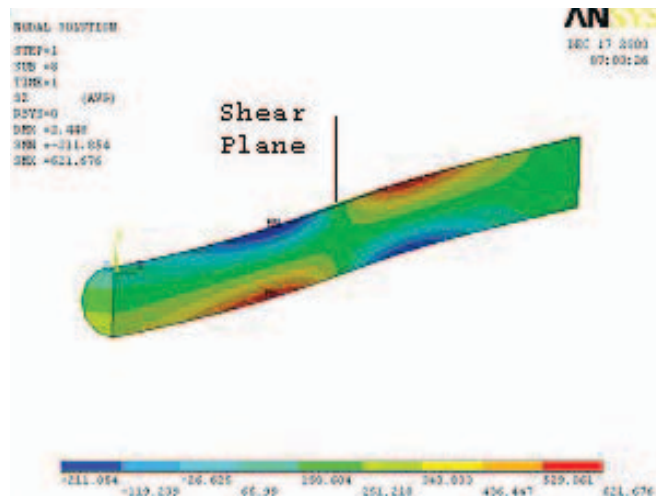


Figure 10. Simulation of shear using Finite Element modeling (Jalalifar, 2006).

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Instrumentation of a Roof Bolter Machine for Void Detection and Rock Characterization

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ABSTRACT

Void detection and identification of bed separation while drilling holes in rocks for roof bolting can be a great source of information for the evaluation of ground conditions and adjustment of ground control or support measures. This can be accomplished through the monitoring and analysis of drilling parameters by the roof bolter. This paper reviews work currently under way in a joint study between Penn State University and J. H. Fletcher & Co.

The detection of voids, along with identification of rock types, can lead to development of 3D roof maps, which can subsequently be used to evaluate ground support and its suitability for given ground conditions in tunneling and mining operations. Existing algorithms for void detection utilize the sensory information of the drill unit, such as feed and rotary pressure, revolutions per minute (RPM), and penetration rate, to detect voids or joints on the roof/ribs. While the void detection algorithms are being revised to allow analysis of available data from the existing sensory systems of the roofbolters with higher accuracy and lower number of false detections, additional sensors have been added to the drill to improve its ability for accurate detection of the voids and joints. This includes installation of vibration and acoustic sensors on a drilling unit at J. H. Fletcher & Co., and pertinent data analysis which are discussed in this paper. Initial results indicate that the data obtained by vibration and acoustic sensors at high sampling rates can be used to improve the performance of the void detection algorithms independently, and when combined with the modified algorithms using existing sensors, it can substantially improve the capabilities of current void detection system.

INTRODUCTION

The applications of rock bolts have become a standard practice in ground support due to their effectiveness in reinforcing the ground and improving the stability of an entry. They are relatively easy to install, lightweight, convenient to transport, and when applied correctly, bolts offer high anchorage capacity and consequently, increase rock mass strength. As such, bolts are an integral part of active support used for ground control in a variety of applications.

Effective installation of rock bolts is highly dependent on correct identification of the geological conditions, including discontinuities because they tend to vary, even within a short distance (Gu, 2003). Currently, many drilling parameters are being collected and processed that can potentially be utilized to estimate the geological conditions at the jobsite (Peng, 2003). A series of studies has demonstrated the potential for analyzing drilling parameters from roof bolters to estimate rock properties and to identify discontinuities (Ichihara, Deguchi, and Itakura, 1995; Itakura et al., 1997a; Itakura et al., 1997b; Itakura, 1998; Itakura et al., 2001; Itakura et al., 2008).

Itakura et al. (1995, 1997a) employed a portable pneumatic roof bolter with the ability to record torque, thrust, revolution, and stroke. Torque and thrust were monitored by using strain gauges installed on the surface of the drilling rod, while penetration and rotation rate were kept constant during the tests. The manufactured blocks included sandstone, sandy shale, and coal samples with three different discontinuity angles of 0 degrees, 30 degrees, and 60 degrees and three types of discontinuities, namely cracks, boundary, and separation of layers. Cracks are discontinuities within a layer. Boundary and bed separation are discontinuities between the layers and often feature an aperture between layers in the rock mass, often intersected by joints. The average value of torque and/or thrust was found to be an indicative index to allow for classification of the rock layers along the borehole. Furthermore, It has been proposed that patterns of thrust or torque along with neural network algorithms may be used to categorize the discontinuities, but the resulting error was rather large (Itakura, 1998; Itakura et al., 2001). Itakura et al. (2008) reported feasibility studies for rock mass characterization while drilling for roof bolts in an underground coal mine in Queensland, Australia. In these studies, 48 holes were drilled using the instrumented drill unit, and one hole was cored in the test area. The system successfully showed the distribution of discontinuities and layer boundaries using the ratio of recorded parameters of torque and thrust.

In a similar work, J. H. Fletcher & Co. developed a system that monitors drilling operations. Fletcher® has been a pioneer in developing instrumented roof bolters to monitor and process drilling parameters, including thrust (obtained from feed pressure), torque (obtained from rotation pressure), rotation rate, and bit position (Gu, 2003). The Fletcher system has been employed to

detect different kinds of discontinuities including voids, fractures, and bed separations, and to estimate the relative hardness of the rock mass (Finfinger et al., 2000). Variation of thrust or feed pressure had been found to be the most suitable identifier of discontinuities (Finfinger, 2003; Gu, 2003; Mirabile, 2003; Tang, 2006; Finfinger et al., 2000; Luo, Peng, and Wilson, 2003; Luo et al., 2002; Peng et al., 2003). For example, Finfinger (2003) proposed a thrust valley concept by which the presence and the size of discontinuities, such as fractures, joints, and voids in the rock, can be evaluated. Based on this concept, thrust decreases rapidly after reaching a void and increases rapidly again when it goes through the discontinuity to keep the preset level of penetration constant. A drop of more than 50% was then considered as an index to detect discontinuity. The distance between the two sides of the valley was also used to measure the discontinuity aperture. Two models were offered for estimating the size of discontinuity.

A secondary parameter of rotational acceleration was also proposed to detect beddings. This parameter could detect 70% of the interfaces designed in the experimental program using layered blocks. The location of 57% of these interfaces was predicted within 2 inches of the actual locations. This system is further developed by Gu (2003, 2005) with the introduction of the drilling hardness (DH) parameter. The DH parameter considers the geometry of the drill bit and contact area between the drill bit and rock, the friction between the drill bit and rock, and the energy lost in kinetic energy, potential, and torsion energies. The slope of the drilling hardness curve and its peak values are used to determine the location of discontinuities and interfaces. Discontinuities were detected using threshold-based algorithms that need to be adjusted for different rock types. This limits the applicability of the system for deployment and utilization in different mining locations. For example, Gu (2003) mentioned that, in one mine site, only 25.86% of the discontinuities were detected by DH method within an acceptable error window. He explains that this failure stems from the fact that the rocks encountered were not weak enough to be detected by the DH slope approach. More recent studies show that the feed pressure is the most indicative parameter for identifying discontinuities, provided that the penetration and rotation rates are constant or under control (Tang, 2006). Tang (2006) developed a method that is able to detect fractures with an aperture of 1/8 inch or larger. However, this approach was found to be somewhat ineffective for discontinuities of 1/16-inch aperture or smaller. Moreover, the accuracy of the drilling parameters recorded by the available system was insufficient to determine the size of discontinuities smaller than 1/2 inch.

Although the instrumented system has been improved to a great extent, there are still some inaccuracies in detecting the location and, especially, the size of discontinuities. Collins, et al. (2004) explained that some voids could not initially be detected by the system during a series of field experiments in a limestone mine, mainly, because of the difference between the hardness of concrete used in the laboratory and the limestone at the roof of the mine. It was found that unlike the usual pattern observed in the laboratory, in which both thrust and torque would drop simultaneously, a sudden rise in the rotation torque happened just before encountering the voids. Meanwhile, the thrust did not have a consistent reaction. New theories were developed later to describe the observed trends. Another problem was reported by Anderson and Prosser (2007), in which the hairline and vertical cracks along with layers of the rocks were not correctly identified. Moreover,

as mentioned before, Tang (2006) elaborated that the applicability of the developed system is limited to voids with size of 1/8 inch or larger.

All the developed methods discussed in this section utilize the drilling parameters to locate discontinuities and estimate rock hardness, which may be affected by many other factors, such as drill size, bit wear and geometry, drill machine mechanisms, etc. (Anderson and Prosser, 2007). In this paper, the installation of new sensors on the Fletcher unit for collecting additional data and monitoring the drill head vibration will be discussed. The initial results will be reviewed and, in particular, the feasibility of using vibration and acoustic signal for void detection, which are independent of the drill unit control system, will be examined. The results of our preliminary analysis indicate that these two parameters are, indeed, informative and can potentially improve the performance of void detection and rock characterization. This refers to increasing the possibility of accurate detection of the discontinuities and reducing the frequency of false detection.

The paper will briefly cover the capabilities of the existing Fletcher roof mapping system. Newly installed vibration and acoustic sensors will be explained, and the results of preliminary testing and data analysis using signal processing tools will be discussed. Finally, some concluding remarks and discussion of future plans for continuation of the research, including full-scale laboratory and field testing will be provided.

FLETCHER INFORMATION DISPLAY SYSTEM

J. H. Fletcher & Co. has developed the Fletcher Information Display System, which uses a programmable logic controller to monitor drilling operations. This system features a drill control unit (DCU) to automate the cycle of drilling and bolting for safety and productivity reasons (Anderson and Prosser, 2007). The DCU processes the drilling parameters including torque, thrust, rotation rate, and position, along with vacuum or water pressure used for flushing, bit breakage, or bending of the drill by controlling the drilling parameters without deteriorating the optimum drilling operation (Anderson and Prosser, 2007). Several modifications have been made to improve the accuracy of measuring bit position and torque (Gu, 2003). The Information Display System was modified to communicate with the DCU to display the information from four separate drill holes side-by-side so that trends could be easily observed in real time. These graphs can show the material hardness and can display the location of voids or other discontinuities in the mine roof structure. Also, rotation events, like stalls, and water events, which may indicate that the drill steel is being plugged with soft material, are marked with colored lines and letters (Anderson and Prosser, 2007). The new Information Display System features a rugged touch screen panel, a solid state flash memory for better durability, uninterrupted power supply, a virtual keyboard for entering additional information to the files, a back-up video display, and a print function (Anderson and Prosser, 2007). The sampling interval in this system is 10 Hz or 0.1 second time interval.

INSTALLATION OF NEW SENSORS

As discussed earlier, a set of vibration and acoustic sensors was installed on the Fletcher drill unit, as shown in Figure 1. An experienced operator can notice changes in vibration of the drill

rod and noise amplitude when the rock strength changes. Previous studies have also examined the advantages of using vibration and acoustic sensors for rock characterization while drilling (Gradl et al (2012), Celada et al, (2012)). Therefore, the drill unit was equipped with additional vibration and acoustic sensors to study the suitability of these sensors for void detection. We will also use the data from these instruments for rock strength characterization in future research.

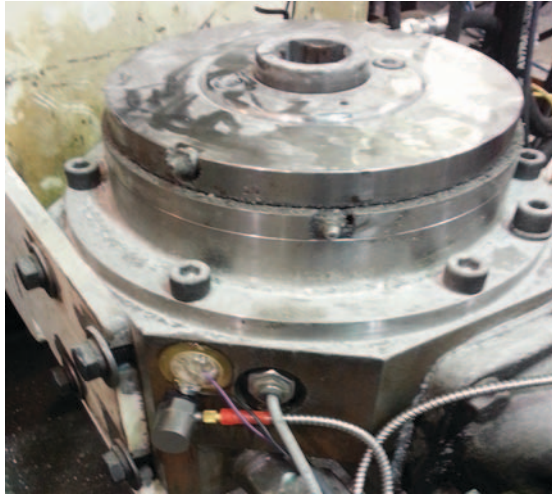


Figure 1. Installation of the vibration and acoustic sensors on Fletcher drill unit.

The additional vibration sensor is a PiezoStar accelerometer with frequency range of 1 to 5KHz with high sensitivity (50 mV/g). The acoustic sensor is a Piezo disc, also known as a contact microphone. It is a small ceramic wafer on a thin metal disc that can be used for acoustic measurements. The sensors are glued to the drilling unit as shown in the Figure 1 to avoid the need for additional machining and to save time. The initial testing and calibration shows that this type of mounting has not impacted the performance of the sensors, but for field deployment, their location should be optimized and a more permanent arrangement should be used to protect the sensors. The sensors, including the accelerometer and microphone, are monitored in parallel to the existing data acquisition device at a high sampling rate of 1KHz. This is done by using a separate data acquisition system (DAQ). The wires from new sensors are directly connected to the new DAQ for excitation and data channels. Also, data from the original sensors of the Fletcher drill void detection system are collected by the new DAQ as analog input from the wiring terminal of the existing Programmable Logic Controller (PLC) of the machine.

The signals from the two data acquisition devices are further synchronized using a relay. However, since the objective of this paper is to illustrate the application of vibration and acoustic sensors for void detection, the data that are usually monitored by Fletcher in their existing roof mapping algorithm are not analyzed here. The two systems will ultimately be integrated to develop a “measurement while drilling” system. This will allow for a better characterization of the rocks and will facilitate optimized ground support design for safe underground mining and tunneling operation. The following section contains the discussion of the

preliminary results of data analysis based on the application of acoustic and vibration sensors for void detection.

REVIEW OF THE RESULTS OF PRELIMINARY TESTING PROGRAM

This section offers the review of the results of the preliminary experiments performed at the Fletcher testing facility in Huntington, WV, on selected concrete blocks. For this purpose, a set of 16 concrete blocks with different strength were poured and allowed to cure for more than 28 days. The blocks are approximately 0.5 x 0.5 x 0.75 m (~20 x 20 x 30 inches), and the concrete mix was designed for various strengths: low (~20 MPa or 3,000 psi), medium (50 MPa or 7,500 psi), and high (70 MPa or 10,000 psi). In this setup, a hard (high-strength) concrete block was placed on top of a soft (low-strength) concrete block. There was a small gap, less than a couple of millimeters, between the two concrete blocks that was considered to simulate a “void” in this study.

VIBRATION SENSOR

As noted earlier, specialized vibration sensors were installed on the drive unit of the roof bolting machine and were monitored with the new DAQ at a sampling rate of 1 kHz. Figure 2 shows the plot of position and vibration signal obtained while drilling a hole with a void. The drill was set with penetration rate of 1 inch per second and a rotation rate of 400 rpm. During the first ~3 seconds of the data collected, bit and drill string is in rotation, as indicated by rotation pressure, and the feed pressure gradually increases. In this part of the test, the position signal shows almost constant value (no actual drilling), and the vibration amplitude is low. As the bits gets in touch with the sample and drilling starts, the amplitude of the vibration increases, as expected. The vibration signal is not a uniform periodic signal because the concrete block is not homogeneous material. However, 26 seconds into the test, where the drill bit reaches the void located 39 inches into the block, the amplitude of the vibration signals decreases. It is logical to anticipate a reduction in the vibration signal when no rock/concrete is being drilled, and the bit runs through the void.

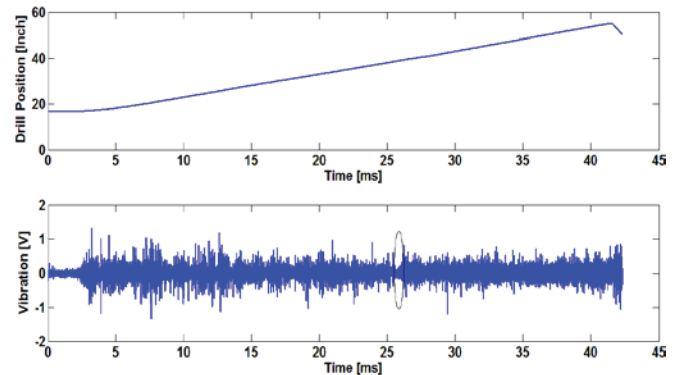


Figure 2. Position and vibration signal obtained from drilling a hole with a void.

This signal provides a measure that is directly correlated with existence of the void, and it is independent of the closed loop control unit of the drill machine. This is the main incentive in using the vibration signal for void detection, rather than using a

process-dependent signal such as feed pressure shown in Figure 3. In other words, although a drop of the feed pressure is usually observed at the location of the voids, as in series of tests, there are cases in which the feed pressure could increase when reaching a void. Having additional sensory information to make a collective decision would potentially increase the detection rate and reduce the false alarm rate. Moreover, the drop in feed pressure is not necessarily due to the voids, but could be caused, for example, by the control unit forcing the drill to operate at optimal range or for a given designed operating point. The results of the Short Windowed Fourier transformation of the vibration signal is shown in 4 to better illustrate the suitability of the vibration measurement for void detection. As Figure 4 shows, two narrow bands can be observed that are clearly distinct from the rest of the signal. The first band is related to when there is no progress in the drilling (i.e., position signal is constant), and the second one is when the drill bit reaches the void. To make the void more visible in the time domain, these two narrow bands are filtered from the vibration signal, and the resulting signal is shown in Figure 5. This figure clearly demonstrates the suitability of vibration signal for void detection.

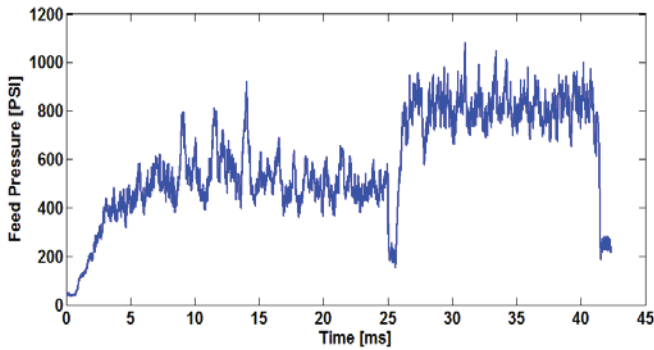


Figure 3. Feed pressir signal coming corresponding to the data shown in figure 2.

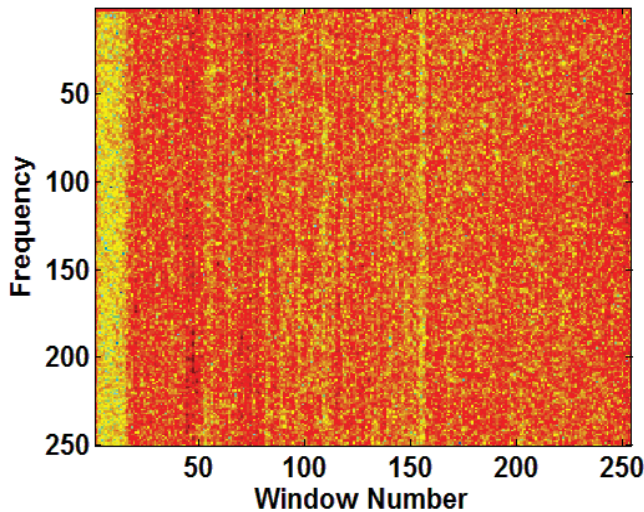


Figure 4. Spectrogram of the vibration signal.

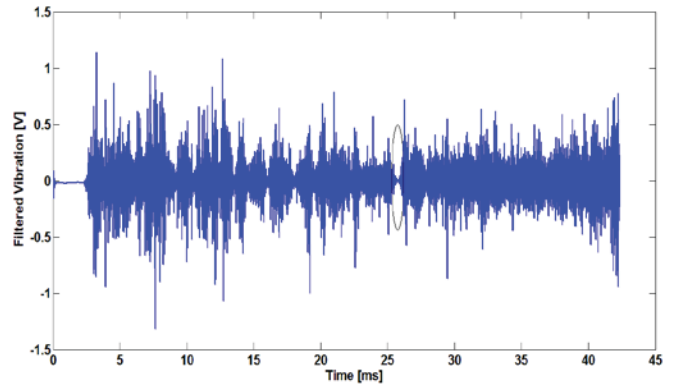


Figure 5. Filtered vibration signal and detected void.

ACOUSTIC SENSOR

Figure 6 shows the acoustic signal during the same experiment where the vibration signal was studied. In contrast to the vibration, the amplitude of the acoustic signal does not change significantly between the first 3 seconds (when no actual drilling is performed) and the rest of the test. The reason for this is that, even if drill bit is not moving forward, the drill bit is rotating and, therefore, generating noise. Thus, it might seem that the acoustic signal cannot be utilized for the purpose of void detection. However, this is not true, which became clear in subsequent analysis. Figure 7 shows the frequency response of the signal, which illustrates that the acoustic signal is, indeed, periodic with important components at a frequency close to 6.6 Hz. This is justified by the fact that the RPM of the drill machine was set to 400 during the drilling of the hole. This is equivalent to almost 6.6 turns a second. This periodic signal exists the whole time the drill bit is rotating, and its amplitude was very big, masking other important information embedded in the signal, including the void information. Therefore, to utilize the acoustic signal for void detection, the signal should be treated with a high pass filter. This is better illustrated in Figure 8, which shows the spectrogram of the signal in the frequency domain. The high value (red part) of the spectrogram at low frequencies demonstrates the same information shown in Figure 6. If this low frequency component is removed, the rest of the spectrogram clearly shows the same two bands observed on the vibration signal. Having a nonlinear filter to further isolate these two bands and reconstructing the signal results in the filtered acoustic signal is shown in Figure 9. This clearly demonstrates the success in the application of the acoustic signal for void detection. Further testing, possibly with different drill strings and with multiple drilling rods in the field would allow for expansion of these measurement to more complex cases and related algorithms for detection should be modified accordingly. It should be emphasized that high frequency components of the acoustic signals are more informative for the application of the void detection than the low frequency components, as discussed here. Therefore, a higher sampling rate than the current 10 Hz would be necessary in identifying voids using acoustic signal. As mentioned earlier, sampling rate of 1 KHz was used in the testing program and the sampling rate will be optimized based on the adopted data analysis algorithm.

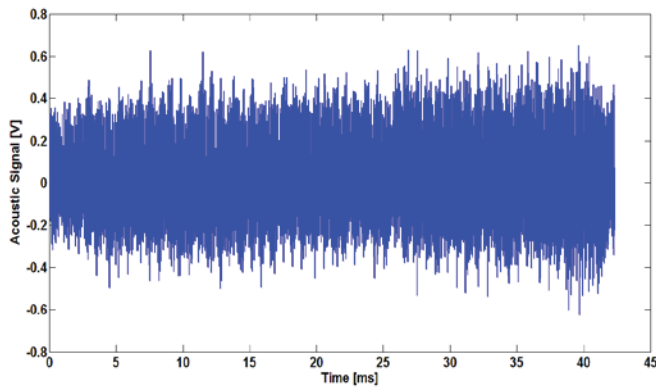


Figure 6. Acoustic signal obtained by drilling a hole with a void.

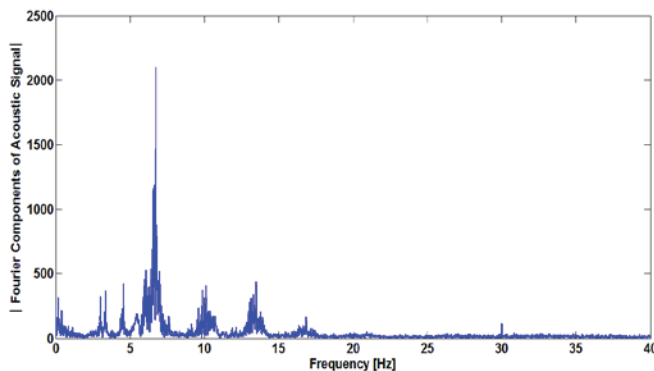


Figure 7. Absolute values of the Fourier components for the acoustic signal.

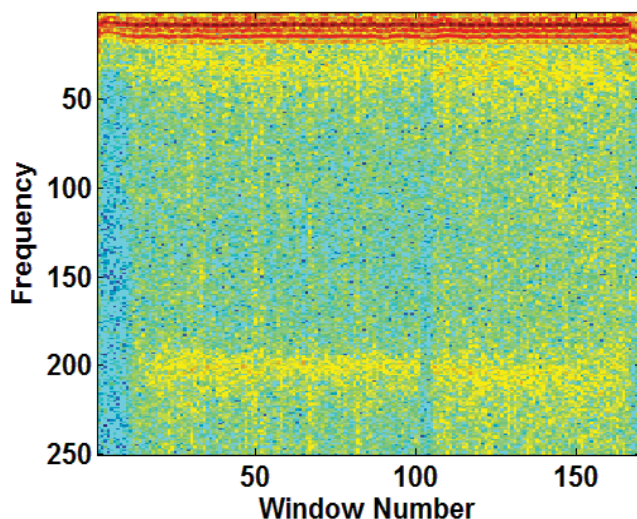


Figure 8. Spectrogram of the acoustic signal.

CONCLUSION

The results of the preliminary analysis of the data from the vibration sensor and the microphone indicate that they can provide a void detection system, which is independent of the existing algorithm that is installed on the J. H. Fletcher & Co. roof bolters with capability of void detection. This allows for the improved detection capabilities of the Fletcher void detection system, which

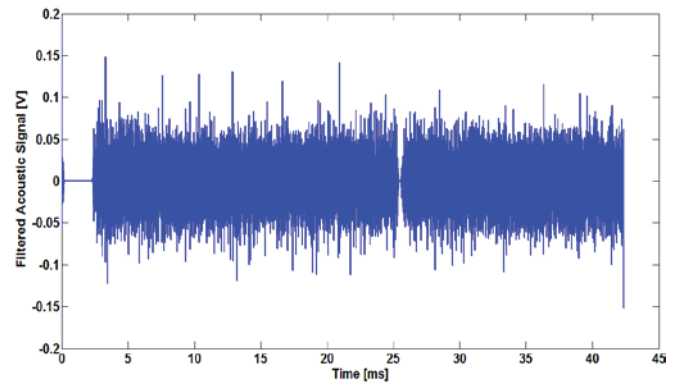


Figure 9. Filter acoustic signal with void detected.

can reduce false detections if used in combination with the existing system. Further testing in a variety of block strengths and also in conditions where the interface between the different layers are at an angle with the drilling path will be examined to further refine the proposed void detection system based on the vibration and noise signals. The developed routine can be programmed as an executable module to allow for installation on various roof bolter units. This allows for the extending the ability for void detection to older units, where, with the addition of an accelerometer and a microphone, the machine can be retrofitted with a PLC or an onboard computer to sense the formations and identify voids. The higher data-sampling rate, which is needed for this type of detection, could create file management problems. Thus a real-time analysis capability within the data acquisition program can facilitate the implementation of the required data filtering and required analysis to convert the high volume of data to a smaller, more manageable set of physical characteristics of the rock formation.

ACKNOWLEDGMENT

This project has been funded by National Institute of Occupational Safety and Health (NIOSH) ground control group under the contract no. 211-2011-41138. The authors wish to extend their appreciation to Professor Asok Ray, Dr. Erick Keller, and J. H. Fletcher & Co. staff and engineers and the NIOSH project manager John Ellenberger for their feedback and support of the project.

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Cross–Correlation Detection of Seismic Events Related to the Crandall Canyon Mine Collapse

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ABSTRACT

On August 6, 2007, the Crandall Canyon Mine experienced a collapse which trapped six mine-workers. An M_w 4.1 seismic event was recorded in conjunction with this collapse. An additional 54 seismic events were detected and located in the vicinity via routine processing techniques during the month of the collapse. Waveform-based cross-correlation detection techniques have been used to uncover 1,328 small, previously undetected seismic events which occurred near the collapse area between July 26 and August 9, 2007. This work has resulted in lowering the magnitude threshold of completeness from M_L 1.6 to M_L -0.5, a 2.1 order-of-magnitude improvement over standard processing techniques. Event locations have been computed using a double-difference inversion. The spatial pattern of events ultimately helps to delineate the extent of the collapsed area. It is shown that many of the large catalogued events ($>M_L$ 1.6) tend to be closely followed by numerous (often 20 or more) smaller aftershocks. Cluster analysis of the known catalog reinforces prior work indicating the existence of seismic clustering on the east and west sides of the collapse. Research now demonstrates that it is possible to differentiate events related to the main collapse from those related to the rescue attempt following the collapse. The cross-correlation techniques described herein greatly improve the ability to detect small mining-induced seismic events. These methods may lead to improvements in assessing seismic hazards for conditions similar to those observed at Crandall Canyon preceding the collapse.

INTRODUCTION

On August 6, 2007, the Crandall Canyon Mine experienced a collapse, which trapped six mine-workers approximately 600 m underground (MSHA, 2007). An M_w (moment magnitude) 4.1 seismic event was recorded in conjunction with this collapse (Ford, Dreger, and Walter, 2008). The mine is located in Emery County, Utah, and, at the time of collapse, was in the process of extracting coal from a portion of the south barrier pillar separating the mains from mined-out longwall panels. The area directly south and the area beyond 0.5 km north of the working section had been previously mined-out via longwall between 1997 and 2003 (Pechmann et al., 2007).

Shortly after the collapse, a continuous miner was deployed in an attempt to reach the suspected location of the trapped mine-workers. On August 16, 2007, an M_L (local magnitude) 1.6 rib-burst occurred at Entry 1, Crosscut 127, which fatally injured three of the rescue-workers. This ended any further attempts at recovering the six trapped mine-workers (MSHA, 2007).

The University of Utah Seismograph Stations (UUSS) operates a regional network of seismic monitoring stations throughout Utah and has been monitoring mining-induced seismicity in the area since 1978 (Pankow et al., 2008). The UUSS has done considerable work in determining precise epicentral locations of the seismic events induced by the removal of coal at the Crandall Canyon Mine between 2005 and 2007 (Pechmann et al., 2007). This work is important for understanding the chronology of seismicity leading up to, and directly following, serious mine collapses (Kubacki, McCarter, and Pankow, 2012).

In response to the August 6 collapse, the UUSS deployed a temporary network of five portable surface stations in close proximity to the mine. Prior to this deployment, the closest seismic monitoring station was situated ~20 km south of the mine. Data from the temporary stations were not fully incorporated into the UUSS event detection system until August 13 (Pechmann et al., 2007). Figure 1 shows the location of the mine relative to the four closest permanent monitoring stations.

Examination of logbooks kept by the rescue workers indicates that many relevant, undetected seismic events occurred in the seven days following the collapse before the temporary network went online. These events were too small in magnitude ($<M_L$ 1.6) to trigger the UUSS detection system, though they have been verified and located through detailed inspection of continuous seismic waveforms recorded at the time (Kubacki, McCarter, and Pankow, 2012). In light of these new detections, it was hypothesized that many more microseismic events may have occurred in the weeks leading up to the collapse than were previously detected.

Waveform-based cross-correlation detection techniques have been shown to perform better than short-term average/long-term average window (STA/LTA) detection algorithms in cases where repeating seismic events occur in tight spatial clusters. Some cross-correlation detection case studies have demonstrated as much as

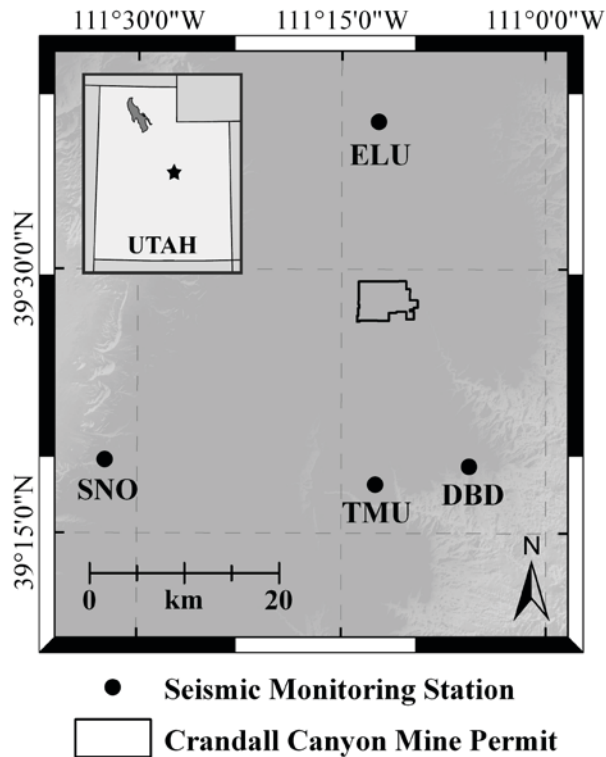


Figure 1. Location of the Crandall Canyon Mine (Emery County, Utah) relative to its four closest seismic monitoring stations: TMU, ELU, DBD, and SNO.

a 1.3 magnitude unit reduction in the detection threshold (Schaff, 2010). Furthermore, highly accurate relative P-wave arrival times can be determined through sliding-window cross-correlation comparisons between detected events (VanDecar and Crosson, 1990). These “lag-times” can be used to invert for ultra-precise relative epicentral locations (Schaff et al., 2004).

Applying cross-correlation detection and location techniques to the Crandall Canyon dataset has ultimately resulted in uncovering over a thousand previously undetected events. These events delineate the pattern of collapse and provide insight into the changing state of stress leading up to August 6, 2007. We show that the use of cross-correlation techniques on data collected from the regional stations is nearly two orders of magnitude more effective than using conventional processing on data recorded by the temporary array.

EARTHQUAKE CATALOG

In order to perform cross-correlation detections, a catalog of known seismic events is needed. These serve as template events that will be compared to segments of continuous seismic data and will only detect new events that are sufficiently similar to the events present in the catalog. For the purpose of locating new detections, it is important to only use well-located events as templates. This is because the locations of the template events heavily influence the estimated locations of the detected events. It

is possible, however, to achieve crude locations using any relevant template event.

A total of 55 seismic events were known to have occurred in the vicinity of the Crandall Canyon Mine during the month of August 2007. Each of these events had been located through routine UUSS procedures; however, better locations were necessary to compare the sequence of events to the sequence of mining. As such, the 55 events, along with 144 other events occurring within a 3-km radius of the mine and dating from August 2, 2005 to July 31, 2007, were relocated (Pechmann et al., 2008) using a double-difference inversion method (hypoDD) (Waldhauser and Ellsworth, 2000). This method involves using travel-time differences for paired events recorded at the same station to invert for relative locations. The relative locations of these events were then shifted by a fixed distance so that the location of the fatal August 16 rib-burst coincided exactly with its known ground-truth location.

The original 55 August events, once relocated, were selected as templates. Since only well-located events that occurred close in space and time to the collapse are useful to this study, only detections made by a subset of 32 out of the original 55 events were considered to be locatable. When compared to ground-truth information reported by the mine, the best-known locations of the original catalog events exhibit an average horizontal error of ~100 m. They form two distinct spatial clusters as seen in Figure 2. One is to the east of the collapsed area and one is to the west.

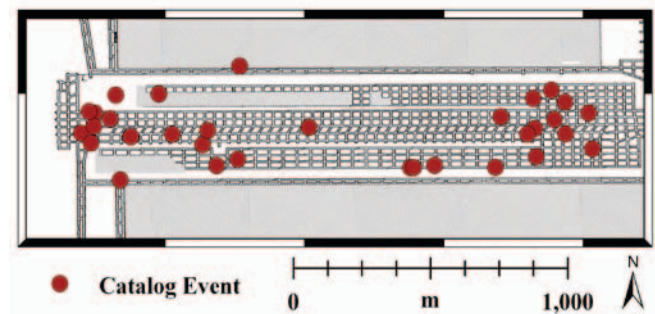


Figure 2. Plan view of the Crandall Canyon Mine workings showing the 32 catalog events forming distinct clusters to the east and west sides of the collapse area. Events occurred during the month of August 2007.

CROSS-CORRELATION DETECTIONS

Sliding-window cross-correlations between the 55 original templates and the continuous seismic waveforms recorded at the four nearest permanent stations (TMU, ELU, DBD, and SNO) have been performed for dates ranging from July 26 to August 9, 2007. The remaining 144 events were not used, as they were deemed to be too separated in space and time from the mainshock to make useful templates. All data were bandpass-filtered between 0.5 and 5.0 Hz before processing. A 15-second window was selected for each template event to be used for cross-correlation. Each window began five seconds after the origin time of the event and contained both P- and S-wave arrivals.

Normalized cross-correlation in the time domain can be computed as follows:

$$C_c(\tau) = \frac{\int u_1(t)u_2(t+\tau)dt}{\sqrt{\int u_1^2(t)dt \int u_2^2(t)dt}} \quad (1)$$

where C_c represents the correlation coefficient (Schaff et al., 2004) as each 15-second template is compared to each 15-second segment of continuous waveform data. Normalized C_c values range from -1 to +1. A result of -1 represents a segment of continuous waveform data that is perfectly out of phase with the template event. A result of +1 represents a segment of continuous waveform data that is identical to the template event (autocorrelation). Figure 3 shows a section of resultant C_c spikes from the hour following the August 6 collapse. Large spikes indicate probable event detections.

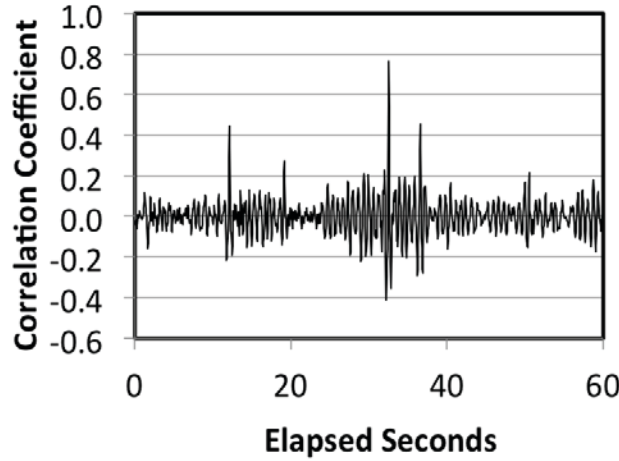


Figure 3. Example of a 60-second cross-correlogram between a catalog template and a section of continuous waveform data recorded at station TMU beginning at 09:00:00 UTC on August 6, 2007. The correlation spike ($C_c=0.77$) at 32.5 seconds indicates a potential event detection.

Slight differences in location, source mechanism, noise content, and prevailing velocity structure between template events and detected events often result in imperfect matches ($C_c < 1$). In many cases, it is difficult to determine whether a C_c spike at a particular time is the result of a true detection or simply the result of well-correlated noise (type I error). When hundreds of detections are concerned, it is unfeasible to manually examine each C_c spike in order to reject all false positives. Therefore, it is necessary to select a C_c threshold with the goal of reducing false positives to an acceptable level while, at the same time, minimizing the number of missed detections (type II error) (Rowe et al., 2012). Only segments of continuous waveform data correlating above this threshold are accepted as true detections.

In order to properly select a C_c detection threshold, we first establish a baseline C_c distribution by cross-correlating a representative template event with one hour of random noise recorded during a period of no known seismic events and minimal mining activity. A histogram of the resulting C_c values is included as Figure 4. Probability distribution fitting analysis (by means of the Anderson-Darling test) of the resulting set of baseline C_c s

(transformed into the $0 \leq x \leq 1$ domain) yields a beta function with shape parameters $\alpha=\beta=233.64$ (see Equation 2). Although it has been suggested that this type of seismic probability distribution function (PDF) can vary widely from day to day (Rowe et al., 2012), examination of noise content during the study period indicates no great deviation from the baseline PDF. Because poorly correlated events can result in large location errors, a conservative C_c threshold of 0.5 was selected as a basis for detections at station TMU. There is a 1.55E-10% chance of a template correlating with a given section of random noise above this threshold.

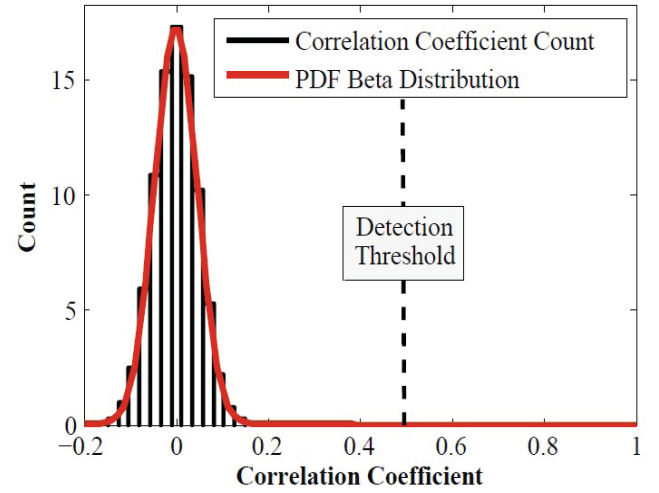


Figure 4. Histogram of C_c values resulting from the cross-correlation of a representative template event with one hour of random noise recorded during a period of no known seismic events and minimal mining activity. The best-fit beta distribution is shown, along with the selected detection threshold of $C_c=0.5$.

$$f(x, \alpha, \beta) = \frac{x^{\alpha-1}(1-x)^{\beta-1}}{\int_0^1 u^{\alpha-1}(1-u)^{\beta-1}du} \quad (2)$$

Using a C_c threshold of 0.5 results in 1,784 detections at station TMU between July 26 and August 9, 2007. Since many of the 55 template events generate C_c spikes occurring within several hundredths of a second for each detected event, k-means clustering (MacQueen, 1967) is used to avoid double counting. This involves automatically partitioning the C_c spikes into distinct detection times for each hour analyzed. For each detection, the most highly correlated template event is recorded and later used for location and magnitude estimations.

MAGNITUDE ESTIMATION

Because all initial detections were made using only the closest station (TMU), stations DBD, ELU, and SNO can be used to further confirm their validity. This is done by comparing magnitude estimates at the four locations. In cases where the estimates are not consistent or not calculable at one or more stations, the detection is discarded. For the purpose of magnitude estimation, it is assumed that every detected event differs only from its associated template

event in amplitude and noise content. This makes it possible to compute amplitude scaling factors as follows (Schaff, 2010):

$$\alpha = \frac{v_{\text{template}} \cdot v_{\text{detected}}}{v_{\text{template}} \cdot v_{\text{template}}} \quad (3)$$

The amplitude scaling factor, α , is equal to the unnormalized cross-correlation coefficient divided by the inner product of the template event (Schaff, 2010). The magnitude of the detected event can be computed using α and the known magnitude of the template event.

$$M_{\text{detected}} = M_{\text{template}} + \log_{10}(\alpha) \quad (4)$$

Magnitudes from the four stations are averaged to produce an estimate for each of the detected events. In cases where clear C_c spikes at other stations cannot be identified close in time to the original TMU detection, magnitude estimates will differ greatly or may not be computable at all. These events have been rejected from the catalog. In addition, all events correlating best with 13 lower-quality templates from the original set of 55 were rejected for the purpose of calculating locations. Combined, this resulted in 1,328 remaining detections. Using the software package ZMAP (Wiemer, 2001), a -0.5 magnitude threshold of completeness (M_{comp}) was computed for this catalog. This represents a 2.1 magnitude unit reduction from the previous M_{comp} or a 126-fold improvement over standard processing techniques. A time-magnitude plot of detected events for the days surrounding the collapse is included as Figure 5.

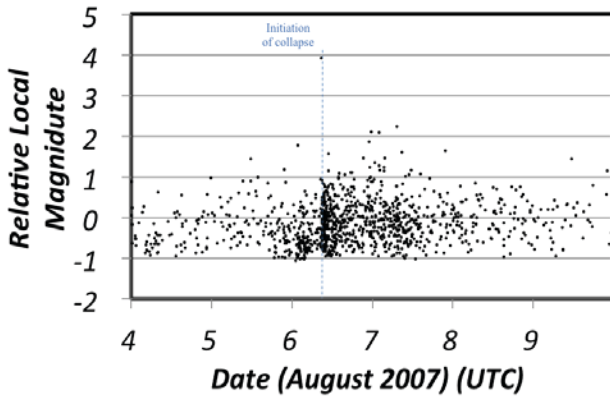


Figure 5. Time-magnitude plot of detected and catalogued events ranging three days before the collapse to three days after. Magnitudes are determined via amplitude scaling factors and are averaged from the four nearest stations. The mainshock is included as an ML 3.9.

DOUBLE-DIFFERENCE LOCATION

The double-difference algorithm, hypoDD (Waldhauser and Ellsworth, 2000), is used to invert for relative locations of the detected events. Each detected event is placed at a starting position coincident with the best-known location of its associated template event. Lag-times at each station are computed between each detected event and all other detected events associated with the same template. In effect, this process pre-clusters the catalog and allows only the most strongly connected events to influence the inversion. The cross-correlation detection technique assumes that all detected events occurred close in space and with similar source mechanisms to their associated template events. Double-difference inversion confirms that the final locations of the detected events remain in close proximity to their starting locations. Events detected by templates from the easternmost cluster tend to remain to the east of the collapsed area, while events to the west remain to the west. A plan view of these double-difference locations is included as Figure 6. While epicentral locations are well constrained, the relative depths of the detected events are largely unknown. It is often difficult to determine precise depths for mining-induced seismic events (Boltz, Pankow, and McCarter, 2012). In this study, depths are simply assumed to be near mine-level.

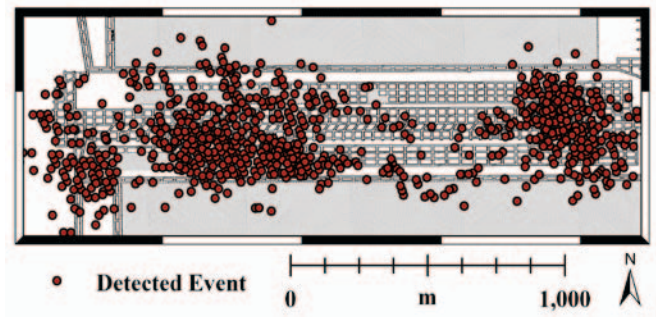


Figure 6. Plan view of the Crandall Canyon Mine workings showing the double-difference locations of 1,328 detected events occurring between July 26 and August 9, 2007.

CHRONOLOGY OF SEISMICITY

A total of 1,328 seismic events were found to have occurred in close proximity to the collapse area between July 26 and August 9, 2007. Of these, 491 occurred on the day of the collapse, and 81 occurred in the hour following the collapse. A large number of the events that occurred in the week preceding the collapse occurred to the west of the collapsed area. This is expected, as much of the mining was occurring on the west. Shortly after the collapse, a large cluster of events began to form on the eastern side of the section. The events already occurring to the west began to cluster farther west than was previously seen. It can be seen that many of the larger events preceding the August 6 collapse tend to be immediately followed by numerous (often 20 or more) smaller aftershocks. The larger events occurring in the weeks after the collapse are followed by significantly fewer aftershocks.

Seismicity is commonly associated with standard coal mining operations. It is difficult to determine which of the newly-detected

events occurring prior to the collapse may have been related to the collapse rather than to daily mining activity.

Cluster analysis of the new catalog reinforces prior work, suggesting the existence of distinct clusters to the east and west of the collapsed area. As expected, events detected on the easternmost side do not correlate well with events detected to the west, and vice versa. Furthermore, events occurring close in space and time to the reported locations of the deployed continuous miner do not correlate well with the aftershocks from the main collapse. This means that it may be possible to distinguish seismic events related to the rescue attempt from those related to the collapse.

A p-value of 0.71 ± 0.14 was computed for this catalog using ZMAP. The p-value statistic represents an exponential aftershock decay rate as defined by Omori's law:

$$N(t) = \frac{k}{(t + c)^p} \quad (5)$$

where $N(t)$ is the number of aftershocks per day. This aftershock decay rate value is similar to the previously reported value of 0.8 ± 0.3 , which used only the originally detected events (Pechmann et al., 2008). In spite of the fact that this new catalog involves an unusually large mining-induced event, the aftershock decay rates reported here are comparable to the median p-value of 0.75 calculated for 11 naturally occurring aftershock sequences in Utah between 1975 and 1992 (Pechmann et al., 2008).

CONCLUDING REMARKS

Cross-correlation detection is a powerful tool for monitoring mining-induced seismicity. Although it is more computationally intensive than standard processing techniques, it allows for microseismic detections of a quality that could only be achieved previously through the use of dense, in-mine arrays. Since in-mine arrays can be expensive and difficult to maintain, it is useful to be able to achieve comparable results using a regional array of seismometers at a distance of 20 to 30 km from the mine.

This study demonstrates that many more seismic events occurred in the days preceding and immediately following the Crandall Canyon collapse than were previously detected. A more in-depth analysis of these 1,328 events could help identify key relationships between mining-induced seismicity and large-scale mine collapses. Temporal and spatial patterns in pre-collapse seismicity are complex, and may not be comparable where mining plans, local geology, and states of stress differ. It is difficult to conclude from a single case whether this collapse could have been predicted. Further study of seismicity in mines with similar geologic environments is necessary before seismic monitoring can become an effective tool in evaluating the potential for a collapse. The spatial pattern of events helps to delineate the extent of the collapsed area. This could be useful in helping rescue workers avoid areas prone to heavy seismicity should collapses similar to that of the Crandall Canyon Mine occur in the future. Efforts may be further aided by the ability to distinguish seismic events related to rescue attempts from those related to true collapse aftershock sequences.

ACKNOWLEDGEMENTS

This research was made possible through funding and support from NIOSH and the State of Utah. We would like to offer special thanks to Mark Hale, Paul Roberson, Katherine Whidden, Jim Pechmann, Meagan Boltz, William Pariseau, Marcela Torres, Jessica Wempen, and Bill Blycker for all of their technical support on this project. We would also like to thank Sam Margulies, Julie Bare, Aaron Young, Ashley Bailey, and Mary Glenn Mellor for their guidance along the way.

The views and conclusions contained in this document are those of the authors and should not be interpreted as representing the opinions or policies of NIOSH or other sponsors of this research.

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Screw Anchors for Ground Control

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ABSTRACT

Due to changing geology, rib bolting is an ever-changing problem in ground control. With the economic status of the coal market, the need for safely increasing productivity in rib bolting is at an all-time high. The current solutions used in the market often require a long installation time (e.g. if collapsed holes have to be redrilled) or result in poor quality installations that can lead to insufficient ground control.

A new bolting product based on the simple screw principle has been developed to bolt ribs faster and with a more repeatable, higher level of installation quality. The paper covers the product invention and development, the fastening principle and its advantages over current systems and first experiences in the market.

The first underground trials have shown that the screw anchors can be installed safely and efficiently, often faster than traditional methods. The screw anchor does not suffer from conventional bolting issues such as collapsed holes due to bad ribs. The screw anchors are designed to obtain steel failure and load testing underground shows consistent fastening in all types of conditions.

INTRODUCTION

Rock bolts for roof bolting have been used in the coal mining industry for decades already.

Due to mining in deeper seams and more difficult geology, the need for rock bolts in the ribs has increased significantly.

Two types of rock bolts have been established over the decades, mechanical and chemical. Both types have their specific advantages:

- Mechanical bolts are often easier to install because there are no specific resin mixing requirements. Further, there are no issues with storage and resin shelf-life.
- Chemical anchors have very good load transfer properties, as they transfer the load over a defined length into the rock and utilize high shear strengths in the bond zone.

However, increasingly difficult geological conditions lead to problems for installation of both types of rock bolts, especially in very fractured coal. The following problems occur more and more frequently and show installation limitations:

- Mechanical friction bolts cannot transfer the required load into the rock; it creeps out of the borehole slowly with increased load.
- Mechanical expansion shell bolts create very high point loads in the borehole, which may lead to failure in the loaded rock.
- Chemical bolts perform much better than mechanical anchors, but often the resin cartridges cannot get inserted into the boreholes properly, as the boreholes collapse immediately after the drilling process.
- Chemical bolts do not have the required encapsulation length, as a lot of resin is lost in the fractured ribs.

The aim for the development of the screw bolt was to provide a solution which could overcome the installation issues and which would be able to cope with the increasingly difficult geological conditions of the coal mines – without losing the advantages of the currently used systems.

SCREW ANCHOR TECHNOLOGY

Screw anchors have been a state-of-the-art technology for fastening applications in concrete for many decades. Similar to rock bolts, concrete anchors are typically subdivided into mechanical and chemical anchors, with similar advantages and characteristics such as the good load transfer of chemical anchors and the ease of installation for mechanical anchors.

The concrete screws help to combine the advantages of both systems:

- The screws are easy to install, typically manual wrenches or impact drivers are used.
- The screws can get loaded instantly, as there is no curing time of resin or mortar.
- The load is transferred over the length of the threads, and not to a single point anchorage.
- Further, the installation torque required is a very good indicator for the successful installation process, as the stall of

the motor at a certain torque equals a pull test of each single bolt after installation

Hilti Research and Development teams have done a lot of research on the theoretical and empirical load transfer behavior of screw anchors. This work confirmed that the load transfer behavior of screws is much like the load transfer behavior of chemical anchors, as can be seen in the 3D modeling of fasteners in concrete below. Figures 1 to 3 show the qualitative results of 3D modeling of a mechanical anchor, a chemical anchor and a screw anchor.

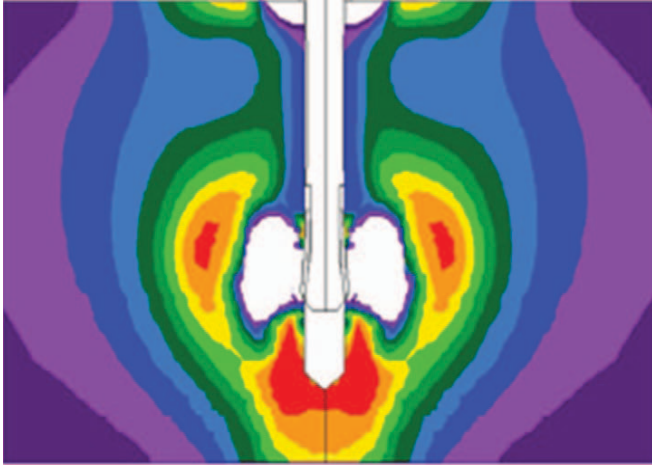


Figure 1. 3D-Modelling of a mechanical anchor.

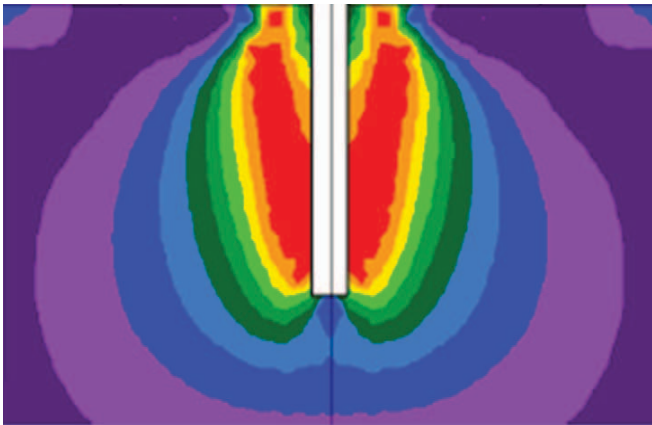


Figure 2. 3D-Modelling of a chemical anchor.

The 3D model of the mechanical anchor shows that the concrete is loaded to very high stresses at a single point. The model of the chemical anchor and screw anchor show very similar stress distributions over the entire length of the anchor, with uniform stress in the concrete caused by the load transfer.

The advantages shown have made screw anchors a very safe, reliable and popular fastening product in concrete, as they combine the advantages of both mechanical and chemical anchors.

DEVELOPMENT OF THE HILTI MINING SCREW

Hilti has done significant research on concrete screw anchors, and they are one of the leading suppliers for these anchors in the

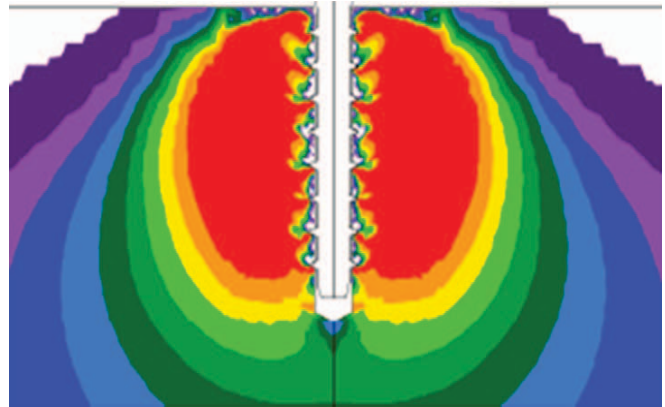


Figure 3. 3D-Modelling of a screw anchor.

global construction industry. Faced with the typical issues which occur during rock bolt installation in difficult geology, the idea emerged to transfer the existing screw anchor know-how from concrete fastening to rock bolting in soft rock mines. The target was to develop a screw anchor for rock bolting applications with the following characteristics:

- Very easy and safe installation
- Suitable for various grounds
- High elongation of the steel tendon
- Load transfer like a chemical bolt
- Exceed the requirements of existing standards, such as ASTM F 432

Very ductile steel which exceeds Grade 75 was chosen as the raw material for the screw, allowing a high elongation for the final product. This ductile steel is fitted with a special thread on the lead end, designed to cut into coal, salt, potash, shale, mudstone, sandstone and concrete. The drive end of the screw is a forged head, which is designed to transfer the full load of the screw and which indicates all of the bolt performance criteria as demanded by ASTM (such as steel grade and length). Finally, a designated heat treatment is applied on the screws to improve the cutting performance and to guarantee the mechanical properties.

The development of the screw anchor has been carried out in constant cooperation with soft rock mines in Europe and the US, to guarantee the fit of the product to the various customer needs. Regular underground visits, installation trials and load tests have also been undertaken during the development process of the Hilti Mining Screw. The final product is supplied with a nominal shaft diameter of $\frac{3}{4}$ in (20mm), a hex head and a thread length of $1\frac{1}{8}$ ft (350mm).

DRILLING SYSTEMS FOR SCREW ANCHORS

Hilti Mining Screws can be installed into both straight and stepped boreholes. Straight boreholes are the standard solution for the 1 or 2 ft screw anchors used for fastening applications, i.e. not for ground control.

Since the thread on screws longer than 3 ft that are used as rock bolts is only applied on the top $1\frac{1}{8}$ ft (350mm), it is advantageous to drill stepped holes into the rock. This allows for a very fast installation of the Hilti Mining Screw, as the rock bolts can be

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pushed into the borehole until the thread diameter step is reached. The rock bolts are then screwed into the narrow part of the borehole until the rock bolts are in their final position and the motor stalls. The size of the hole for the leading section fits the thread of the screw anchor, whereas the outer diameter is significantly larger, typically around $1\frac{1}{4}$ in (32 mm).

Stepped drill bits the required length and diameter have not been available, so new drilling systems have been developed in a parallel work stream. Both vacuum and wet drilling systems have been developed to fulfill customer needs. The drilling systems have been tested constantly in the laboratory and in underground tests in coal and potash mines during the development process.

MSHA TESTING

Following the development, the Hilti Mining Screws were tested in the MSHA laboratory. Since the screws are not continuous and homogeneous reinforcement steel, but a heterogeneous element consisting of a screw part, a shaft and a forged head, an ASTM F432 compliant classification of the entire product appeared difficult. However, tests proved that Hilti Mining Screws easily compare with $\frac{3}{4}$ in grade 75 rebar in accordance with ASTM F432. Figure 4 shows the load displacement diagrams of the pull tests.

The tests, carried out at the MSHA Safety and Health Technology Center in Pittsburgh, PA, confirmed the mechanical properties and their compliance with the requirements of ASTM F432 for straight pull as well as for pulling with an angle deviation of 10 degrees. Table 1 shows the summary of the results and the comparison with the requirements for $\frac{3}{4}$ in Grade 75 rebar.

INSTALLATION PROCEDURE

The setup of the Hilti Mining Screw allows an installation by heavy underground machinery or a completely handheld installation. This capability makes the system unique in the market. If installed from machines like a miner-bolter, the designated drill steel (one of the stepped wet systems) is inserted into the drill motor and the hole is drilled according to the ground control plan. The drill steel is then retracted and removed, and the screw is placed in the drill motor using an installation adapter. The Hilti Mining Screw is then pushed into the borehole until the smaller diameter step is reached and screwed into the narrow top part of the borehole until it is in its final position and the motor stalls. Depending on the torque of the motor, pretension in the range of 1 to 4 tons is applied to the plate. If a higher pretension is required, this can be achieved by using torque multipliers. Figure 5 shows the correlation between the applied bolter torque and the achieved pretension level.

If the bolts are installed manually, such as for repair work in remote areas of the mine, hand-held drilling equipment can be used. Bolt installation is performed with hydraulic, pneumatic or electric (corded or cordless) impact drivers. This allows maximum flexibility and eliminates dependence on heavy, expensive equipment.

In both cases, the design of the bolt tip allows installation even in partly collapsed or completely collapsed boreholes, as the screw anchor will find its way through the rock particles which are

blocking the borehole. A drawing of an installed screw is shown in figure 6.

SYSTEM ADVANTAGES

The Hilti Mining Screw (HMS) is a mechanical rock bolt with the performance of a chemical anchor. It can be installed using heavy underground machines as well as with lightweight hand-held machines. The load transfer is similar to chemical anchors, which allows high bolt loads even in weak rock. The fact that it consists of one single and robust item makes it easy to handle and easy to install. Further, there are no problems with resin shelf life, resin mixing or gloving. Its robust, chamfered tip allows installation even into collapsed boreholes, when installation of expansion shells or resin cartridges is impossible.

Once installed, it can be easily retightened (for example, if ribs collapse behind the mesh and the plate comes loose) thus keeping the designed bolt length inside the rock and without leaving bar protrusions into the roadway. Further, Hilti Mining Screws can be removed completely by just unscrewing them from the rock, which might be necessary if supported rock must be cut (such as for crosscuts) where conventional bolts (mechanical or chemical anchors) would be obstacles for cutting equipment.

UNDERGROUND EXPERIENCE

During the R&D process, there were frequent installation tests in various mine geologies across the US and Europe to confirm the work. In the following case, a large-scale test in a German potash mine is illustrated. This installation was carried out two years ago and monitored continuously. The German potash mine had been using 4ft (1.25m) long rebar with expansion shells with an ultimate load of 32,700 lb (145 kN). The goal of the test was to determine how many of the expansion shell bolts could be replaced by screw anchors. Further it was of interest if there would be any negative side effects from using the new Hilti Mining Screw technology compared with the proven technology of using mechanical bolts with expansion shells.

A total of 1,000 bolts have been installed into the roof, with no other additional roof bolts. Installation has been done with a fully automated drilling and bolting system. Figures 7 to 10 show pictures of the installation.

The entire drilling and installation process has been monitored with data loggers, collecting drill times, installation torque and many other parameters. This enabled a full analysis of the 1,000-piece trial.

Simultaneous to the installation, regular pull tests were conducted. In all pull tests the failure of the bolts occurred by steel failure in the shaft at loads of greater than 45,000 lb (200 kN). Not a single bolt showed pull out behavior of the threads. The pull tests were also used to determine the pretension level on the bolts which was an average of about 3.5 tons. This preload was achieved using only the torque of the bolter.

Since the completion of the large-scale installation more than two years ago, constant monitoring with ultrasound devices has been in place to monitor stress changes in the bolts or other abnormalities potentially caused by blasting in nearby areas. None

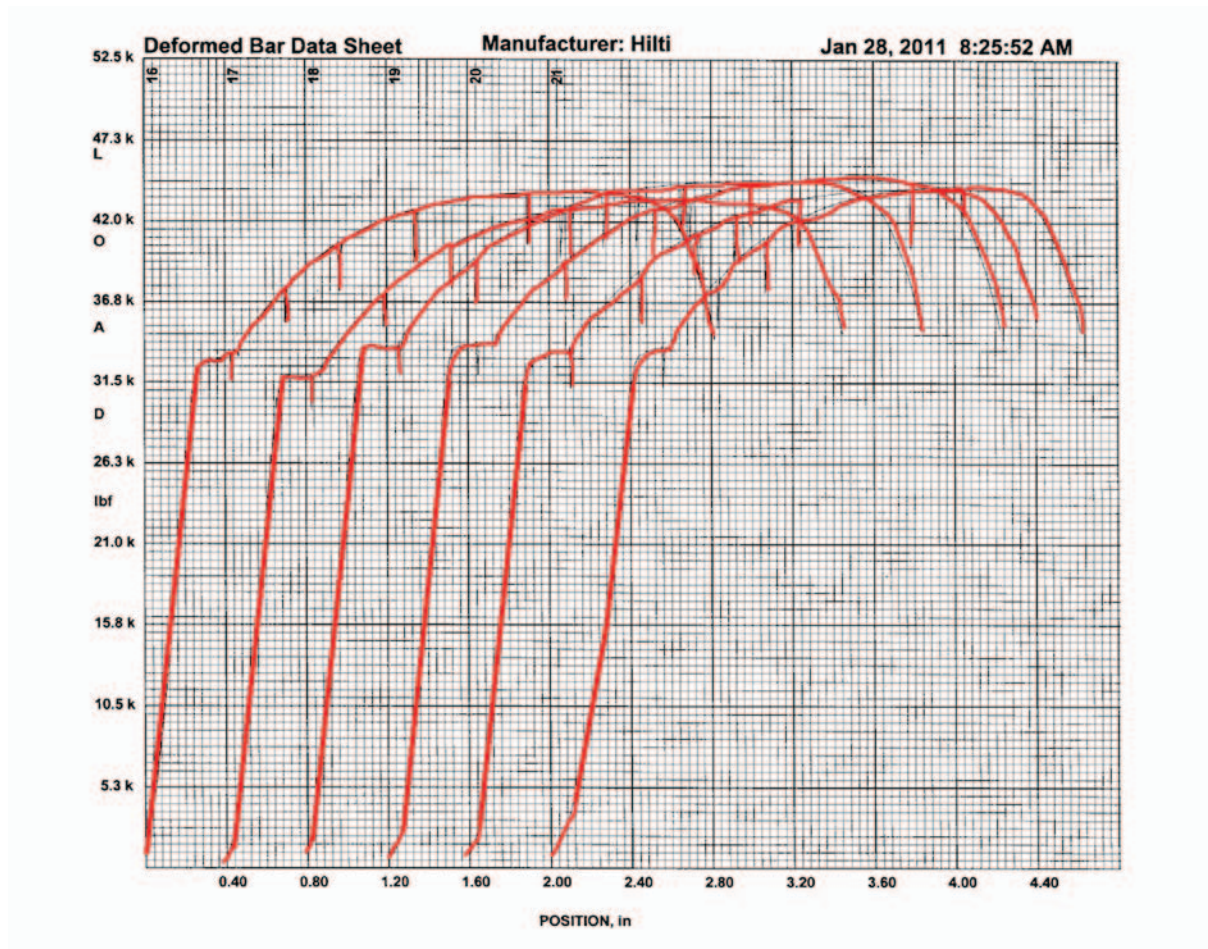


Figure 4. Load-displacement diagrams from MSHA.

Table 1. Test results from MSHA.

Parameter	Requirement (3/4", gr. 75)	Actual
Yield load	25,100lbf (112 kN)	~ 33,000 lbf (147 kN)
Ult. load	33,400lbf (149 kN)	~ 45,000 lbf (200 kN)
Elongation of the shaft	8%	18.8 %

of the monitored bolts has ever shown any signs of failure or other suspicious behavior, and the bolted area is still in perfect condition. This large-scale underground test has been considered a big success for the screw anchor technology.

SUMMARY AND CONCLUSION

The Hilti Mining Screw is a newly developed product for ground support. It combines the advantages of a mechanical bolt with the bond properties and load distribution of a chemical bolt. The bolting system has been developed together with special drill steels to deliver a complete system to mines. Initial underground tests in

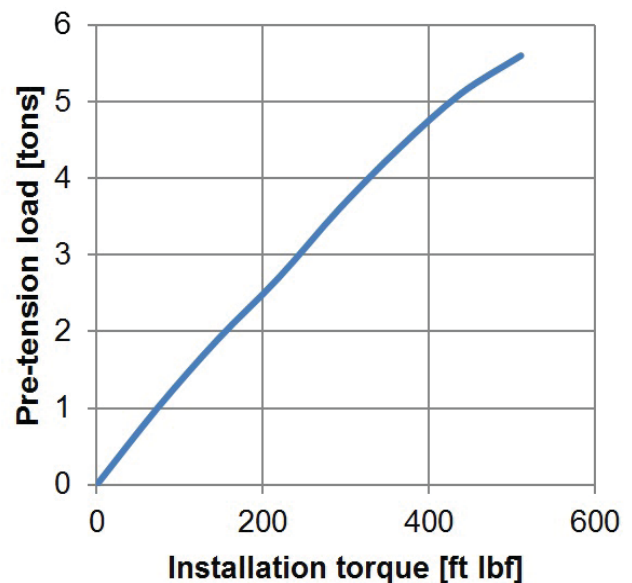


Figure 5. Correlation torque - pretension.

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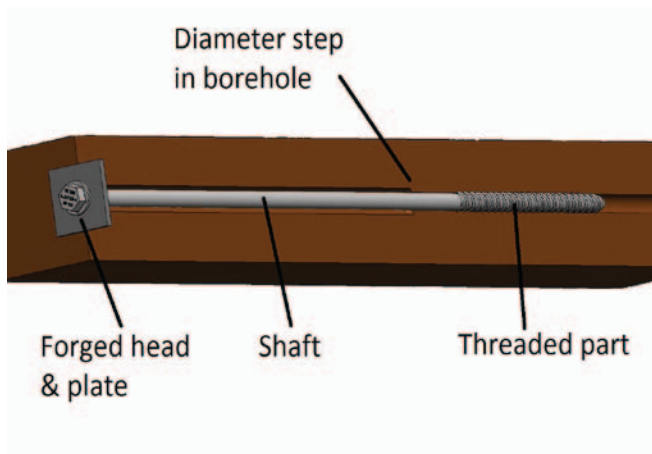


Figure 6. Installed Mining Screw.



Figure 8. Stepped drill bit in operation.



Figure 7. Fully automated drill rig.

the US and in Europe as well as all laboratory tests at MSHA have confirmed the suitability of this system for applications in roof and rib support for all types of soft rock mines.



Figure 9. Installed Screw Anchor.



Figure 10. Pull-Testing.

Case Studies of Innovative Cable Bolt Technology

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ABSTRACT

Over the past three decades, the U.S. coal mining industry has widely used cable supports to supplement traditional primary bolting systems, especially in difficult ground conditions. However, increased mining activity in adverse conditions has underscored the limited effectiveness of the non-tensioned, partially grouted cable supports used to supplement primary bolts. Roof failures in conditions such as corrosive environments, highly laminated strata, low overburden depth and water intrusion have demonstrated that the need for cable technology has reached beyond the traditional non-tensioned, partially-grouted cable tendon. Furthermore, declining market conditions and increased competition for coal customers has demanded that new cable bolt technology be cost-effective and readily adaptable to the mining cycle.

Jennmar Corporation has developed several cable bolt technologies that can be used as stand-alone solutions or in concert with one another to mitigate adverse conditions. These technologies include rapid tensioning of cable supports, structural surface control and polyurethane injection. The case studies presented in this paper use one or more of these technologies to address adverse mining conditions or special ground control problems.

INTRODUCTION

Based on customer purchase decisions and successful completion of special ground support projects, three recent developments in cable bolting technology have proven especially beneficial to the U.S. coal mining industry:

- Rapid tensioning of cables has been successfully employed to support highly laminated strata.
- Structural surface control used in conjunction with cable supports is regularly used to support weak or disturbed strata.
- Also, polyurethane injectable cables have been used for corrosion protection, mitigation of water intrusion and consolidation of roof strata.

The Jennmar INSTAL CC cable bolt is a hybrid of a traditional cable bolt and a conventional roof bolt (Figure 1). A bail-type expansion shell is attached to a heavy-walled stiffener tube. This expansion shell can be located from four to six feet above the roof

line, depending on mining height. The cable tendon passes through the heavy-wall stiffener tube, and extends from the barrel-and-wedge assembly at the roof line to the maximum length of the cable support. The expansion shell allows tensioning of the cable, up to approximately 10,000lbf. Tensioning is completed as the bolt is installed and does not require additional equipment or time after initial installation. Traditional post-tensioned cables require individual cables be tensioned with a hydraulic tensioner after the resin grout has cured. This additional installation step limits the installation rate and overall deployment of tensionable cables. Operators often have difficulty maintaining a cable bolt installation rate that is consistent with the mining rate. This situation often results in un-tensioned cables or cuts that sit unsupported for an extended length of time.



Figure 1. Jennmar INSTAL CC cable bolt.

The application of structural surface control is simply a marriage of two existing ground support technologies. Truss systems have been used to support mine roofs for a number of years. Angle bolts, anchored into undisturbed strata above the mine ribs, support a horizontal cross-member that is designed to support and confine the immediate roof. In an ideal situation, mine roof can be supported without having to suspend weak lower strata from more competent strata higher in the mine roof. Traditionally, trusses have been employed when it is impractical to drill vertical holes deep enough to reach competent, undisturbed strata.

One of the major disadvantages of traditional roof trusses is evident in uneven mine roof (Figure 2). Often there will be large, unsupported gaps between the cross-member and the roof. Additionally, traditional bar or cable cross-members are not inherently strong against bending moments. These factors make a traditional truss system very flexible when subject to roof loading. As will be shown in the case studies, the Jennmar engineering staff often specifies the use of cable bolts and roof channel in the

B-Rigid truss configuration as a stiff, high capacity truss alternative (Figure 3). A three- or four-hole roof channel is used to apply structural surface control. The cross-section of the roof channel is designed primarily to resist bending moments. Two cable bolts are angled over the mine ribs to anchor into undisturbed strata, similar to a traditional roof truss. One or two cable bolts are installed vertically toward the center of the channel, and these vertical cables provide additional stiffness to the center of the channel and may or may not anchor into competent strata. The B-Rigid truss utilizes off-the-shelf roof control products, and many mines already have the assorted components in stock in the mine supply yard. Additionally, no special tools are required to install the B-Rigid truss. It can be used in conjunction with all cable bolt systems including INSTaL CC cables, fully grouted cables, post-tensioned cables and non-tensioned cables.



Figure 2. Channel forms to the roof line; truss cross-members leave unsupported gaps.

Jennmar Corporation began development of the Fully-Grouted Cable Bolt (FGCB) in 2010. It was developed in response to coal operators' concerns regarding cable corrosion in workings designed to remain open long term. The FGCB consists of standard seven-wire cable with the addition of a bottom-hole plug and modified head. The FGCB is installed in exactly the same manner as a non-tensioned, partially grouted cable bolt with a 48 in resin cartridge for bolt anchorage. The bolt head is modified to allow the attachment of polyurethane injection equipment. Because the FGCB is installed with the same method as a non-tensioned partially grouted cable bolt, there is little to no disruption of the mining cycle on miner sections where cable bolts are normally installed. Polyurethane can be injected into the bolt annulus at any time after the resin grout has cured. This feature allows mine operators to install the FGCB in areas where adverse conditions are anticipated, then inject the bolts on an as-needed basis.

CASE STUDY 1

Case study 1 is a longwall headgate in the Eagle seam, in southern West Virginia. Supplemental support in the headgate was installed approximately 9 months after the headgate was developed. The roof in this area consisted of highly laminated sandstone with mica streaks (Figure 4). During installation of supplemental support, mining-induced roof fractures were discovered up to 11 ft above the roof line, using test drilling with a roof bolter. Fracture

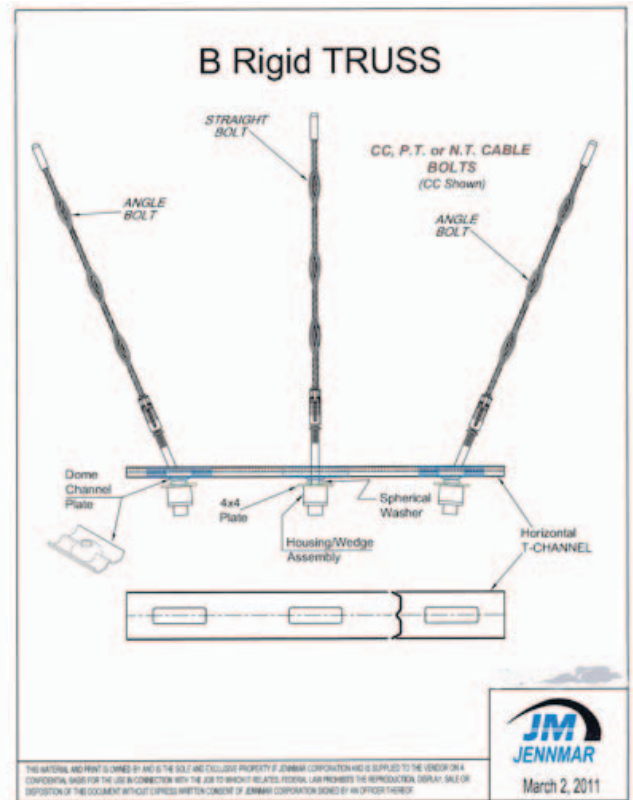


Figure 3. Jennmar B-Rigid Truss.

location and density were confirmed with a video borescope. Fracture density was very high in the first four feet above the roof line, with some test holes showing 6 fractures per foot of test hole. Polyurethane resin was specified to consolidate the highly fractured immediate roof, and 0.7 in x 120 in grade 270 FGCB cables were used to inject polyurethane. This bolt length produced an injection zone to 6 feet above the roof line when used with a standard 1 1/4 x 48 in resin cartridge for anchorage. A single cable was installed approximately every 8 feet near the center of the entry. In this case, polyurethane was used in conjunction with previously installed supplemental support. After all supplemental support was installed; polyurethane was injected into the FGCB cables, and allowed to migrate into the roof (Figure 5). Migration of the polyurethane was evaluated with a video scope and by day lighting of the polyurethane in adjacent bolt holes. With these methods, the polyurethane was found to migrate in an approximately 10ft radius around the bolt (Figures 6 and 7). In this case, the FGCB was used as a cost-effective and efficient method for consolidation of the immediate roof. The installed cost of the FGCB was approximately 75% of a traditional inflatable packer polyurethane injection system.

CASE STUDY 2

Case study 2 is a long-term belt corridor in the Eagle seam, central West Virginia. Constraints on the mining reserve required that the corridor be developed underneath a valley where the original overburden depth was less than 50ft (Figure 8). Compounding the difficulty was the fact that the valley had been filled in the 1950s and the fill depths reached approximately 150ft in the belt corridor area.

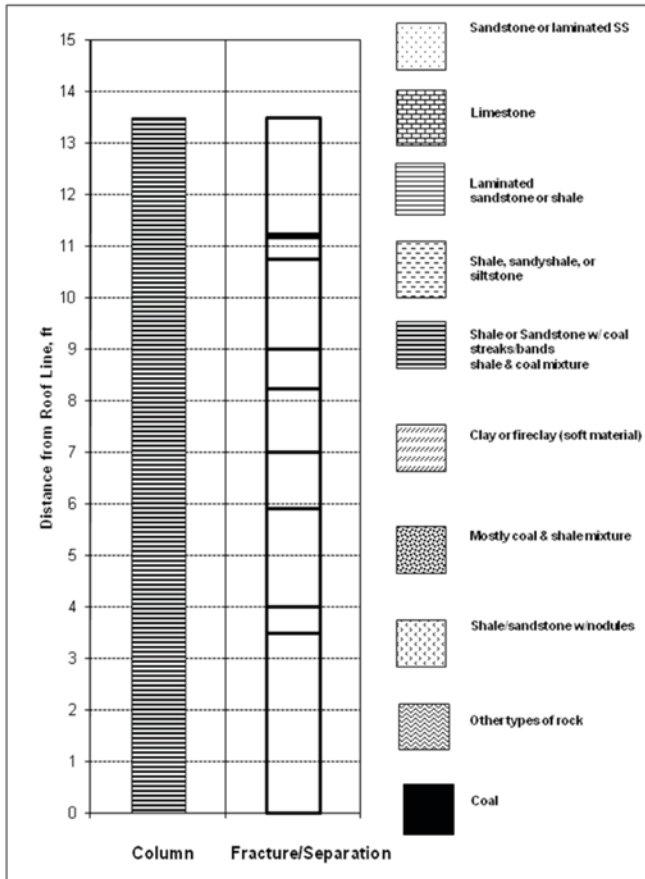


Figure 4. Roof Geology -- Case Study 1.



Figure 5. Polyurethane injection for strata consolidation.

Two coreholes were drilled near the location of the lowest overburden areas of the mining projections. In addition, a borescope was used to examine the roof in developed areas of the mine adjacent to the belt corridor. Both the coreholes and borescope observations indicated the mine roof consisted predominantly of massive sandstone. Some indications of weathering, such as high angle fractures and water staining, were found in the upper portion of cores retrieved from the coreholes. Due to the low overburden depth, it was considered very likely

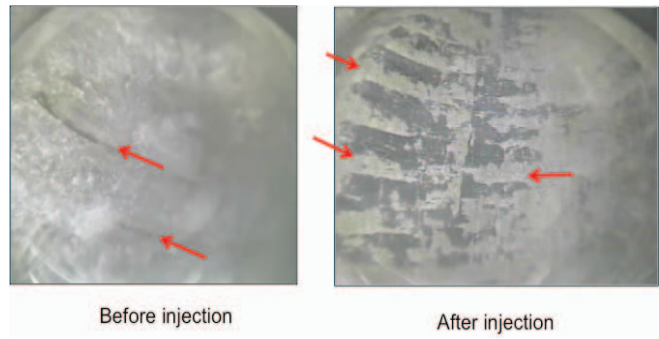


Figure 6. Polyurethane found in adjacent test holes.

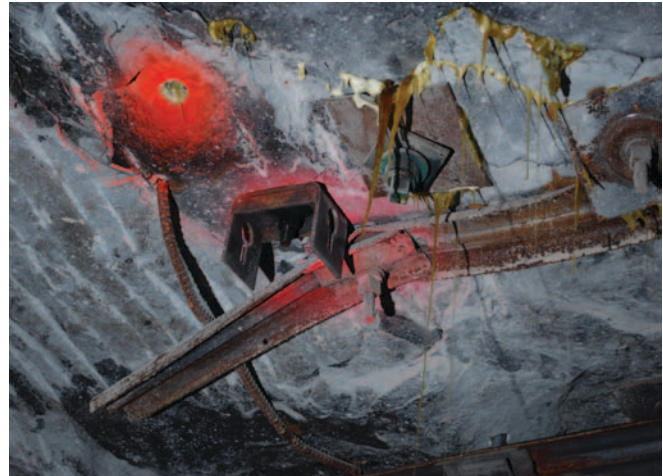


Figure 7. Polyurethane migration at roof line.

that significant weathering and unconsolidated strata would be encountered during development of the belt corridor.

In cooperation with mine engineers, Jennmar Corporation engineers specified a bolting system that would resist long-term corrosion, and have the ability to consolidate highly weathered and broken strata. Primary support consisted of (3) $\frac{3}{4}$ in x 72 in grade 60 fully grouted rebar bolts per row with four-foot spacing between rows of bolts. Supplemental support consisted of (4) 0.7 in x 144 in grade 270 FGCB bolts used in conjunction with T5 x 192 in roof channel in a B-Rigid Truss configuration. The final spacing between B-Rigid trusses was 4ft. The B-Rigid Truss system with FGCB bolts provided confinement to the immediate roof, and allowed polyurethane to be injected into the roof for strata consolidation and water control.

Development of the belt corridor began in late 2012. As the corridor approached the 100ft overburden line, polyurethane pumping equipment was installed in preparation for adverse conditions. A borescope was used to evaluate weathering in the roof on an ongoing basis. As the #3 entry approached the minimum overburden area, weathering was found in the underground borescope holes (Figures 9 and 10). This weathering was consistent with weathering observed in coreholes prior to mining. Indications of weathering included: water staining, presence of high angle fractures, and clay infilling of fractures. As the section advanced, polyurethane was injected into the roof thru the Fully-Grouted Cable Bolts to consolidate weathered areas and mitigate water intrusion. In general, bolts were injected no more than 100ft



Figure 8. Mining projections for belt corridor under very low cover.

from the active face. Polyurethane migration was observed in test holes and vertical fractures in the roof (Figures 11 and 12). To date, the belt corridor has advanced beyond the minimum overburden depth and has experienced no roof control issues.



Figure 9. Roof weathering observed in test hole.

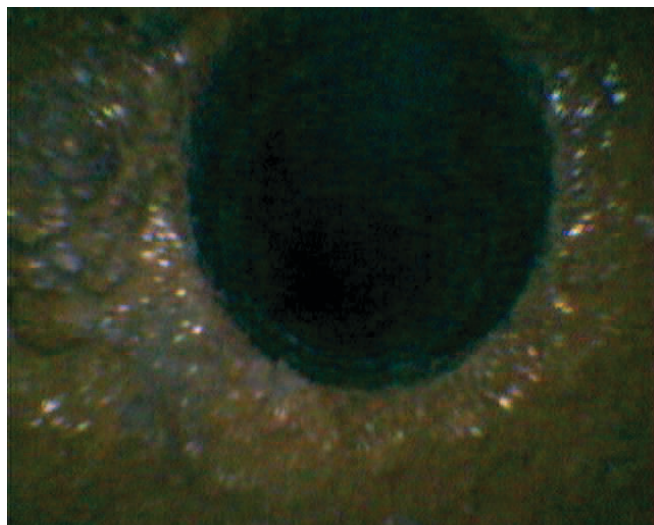


Figure 10. Roof weathering observed in test hole.



Figure 11. Polyurethane migration at roof line.



Figure 12. Polyurethane migration at roof line.

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CASE STUDY 3

Case study 3 is a longwall setup face in the same mine as Case Study 1. The roof in this area consists of highly laminated sandstone with mica streaks. Equipment size required that the setup face be developed to a final width of 26 ft, but it was initially developed to a width of 18 ft.

Three stages of roof support were installed in the setup face; the first stage consisted of (4) $\frac{3}{4}$ in x 72 in grade 60 fully grouted torque tension rebar bolts installed on a T5 x 192 in roof channel. Spacing between rows of primary roof bolts was 4 ft. Additionally, (4) 0.7 in x 144 in grade 270 non-tensioned cable bolts were installed on T5 x 192 in roof channel between every row of bolts. After the first stage of roof support was installed, the setup face was advanced again at an 18ft width. Before the slabbing cut was taken, the second stage of roof support was installed.

The second stage of roof support consisted of (4) 0.7 in x 192 in grade 270 post-tensioned cables installed adjacent to each non-tensioned cable bolt. The slow installation procedure for post-tensioned cables prompted the mine to install them outside of the primary bolting cycle. According to mine management, the second stage of roof support was installed in large batches sometime after the setup face had advanced. Typically, the second stage of roof support was installed in crosscut-to-crosscut batches. A 6 foot slabbing cut was then taken to widen the setup face to the final 26ft width. For the final stage of roof support (2) additional $\frac{3}{4}$ in x 72 in grade 60 fully grouted SuperTwist bolts were installed in the roof above the slabbing cut with (1) 0.7 in x 192 in grade 270 post-tensioned cable for supplemental support. The slabbing cut was then cribbed off until longwall equipment was installed (Figure 13).



Figure 13. Initial support plan for case study 3.

As the setup face was developed, vertical fractures, cutters and other signs of roof movement developed outby the active face (Figure 14). Based on interviews with face personnel, most of the roof movement developed between installation of the first and second stages of roof support. After the second stage of support was installed, further indications of roof movement ceased.



Figure 14. Signs of roof movement.

The roof in the setup face was examined with a borescope. The immediate roof consisted of approximately one to two feet of silty shale, and a roof of laminated sandstone with mica streaks above that. Mining-induced horizontal fractures were found up to approximately 10ft above the roof line (Figure 15). Timing of the roof movement and appearance of cutters indicated that applying tension to the supplemental roof support in a more timely fashion would improve roof conditions.

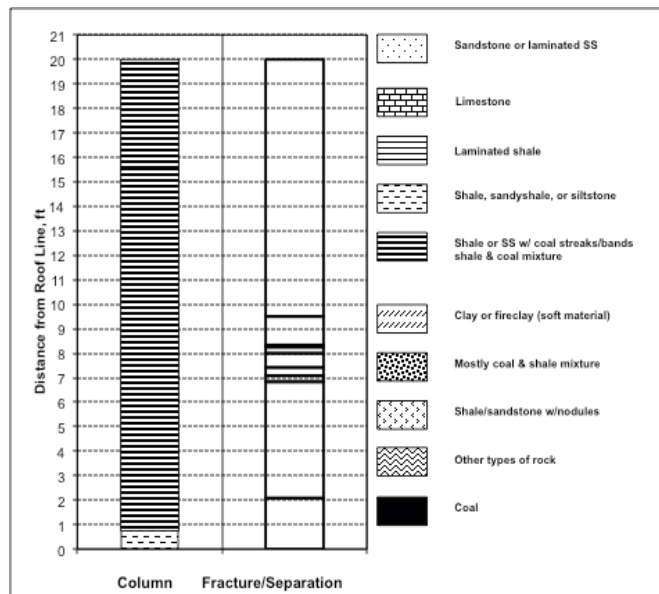


Figure 15. Fractured roof observed in initial support plan area.

Because stability of the longwall setup face was critical, post-tensioned cables were not eliminated from the roof support plan. Instead, the (4) 0.7 in x 144 in grade 270 non-tensioned cables with T5 x 192 in roof channel were replaced with (4) 0.6 in x 144 in grade 270 INSTáL CC cables. These cables allowed application of tension to the supplemental support during the first stage of roof support. In addition, installation time was reduced by approximately 20 seconds per cable.

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After adoption of the new support system, signs of roof movement were greatly reduced (Figure 16). Scope data indicated similar geology, with a reduction in fracture density of about 63%. Additionally, the maximum fracture height was reduced from approximately 10 ft to approximately 3 ft (Figure 17).



Figure 16. Revised support plan.

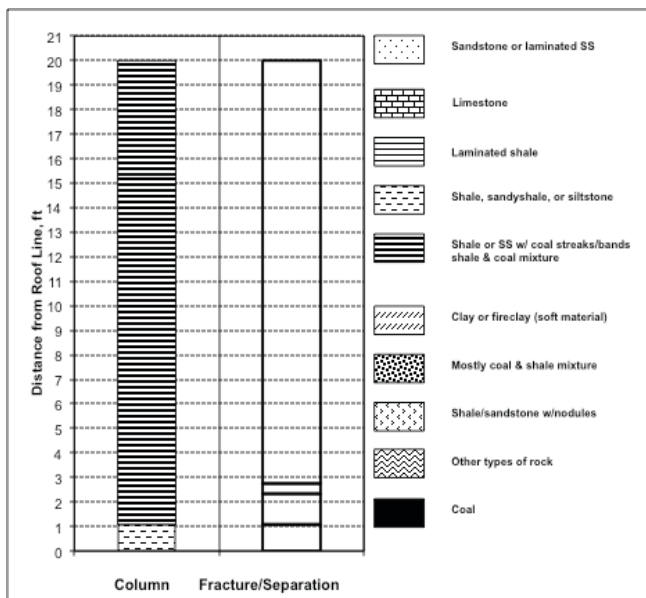


Figure 17. Revised support plan showed 65% reduction in roof fracturing.

CONCLUSION

Recent innovations in cable bolting technologies from Jenmar Corporation have assisted mine operators with supporting mine roof in a safe and efficient manner. The B-Rigid Truss system effectively applies structural surface control to weak, fractured strata. The INSTaL CC cable bolt provides a means to rapidly apply tension to the cable support. This rapid deployment of cable tension has proven effective in controlling roof movement

and mining-induced fractures. Where polyurethane injection is necessary, the Fully-Grouted Cable Bolt can be used as both a supplemental support and an injection point. In addition, the Fully-Grouted Cable Bolt is approximately 75% of the cost of an inflatable packer injection system. All of these technologies have been designed to dovetail into the mining cycle, and present minimal interruptions in mining activity.

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Development of CAN Manipulator

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ABSTRACT

Underground mining requires a variety of specialized equipment. With ever-changing mining conditions and methods, some mines have had to resort to purchasing existing industrial equipment and adapting it for underground use. Since this equipment is not made for underground mining or their specific task, the result is often lost time, money, and sometimes an unsafe working condition. One such application is the setting of a CAN. A CAN is a type of cylindrical secondary roof support made from a cement composite and encased in steel. The CAN has become popular because of its simple one-piece design that can handle heaving and sagging better than other available designs. "Because of this exceptional ability to sustain load through extended convergence and displacement, the CAN is one of the most robust and stable standing supports in use today." It also promotes better air flow due to the size and shape of the CAN. CAN size and weight have increased over time to handle more difficult mining conditions.

Significant down time and inefficiencies with existing CAN setting machines at Twentymile Coal were directly due to the item in use not being designed to be task-specific. They were undersized with very limited movement. The miners had to take extra steps and precautions when setting CANs. Maintenance delays were significant.

Twentymile Coal Company realized that they required a safer, more reliable and more efficient machine to handle and set CANs, and looked for a partner to develop a new design-specific machine. They worked jointly with J.H. Fletcher® engineering and sales to design and manufacture a machine specifically made for setting CANs.

The objective was to produce a machine that would handle the conditions and methods that the mine required for setting CANs during longwall retreat. The resulting design had to be safe, stable, have a wide range of motion, be ergonomic, simple to use, and easy to maintain. The machine also had to improve efficiency.

The resulting machine has been operating for approximately one year at Twentymile Coal and has been proven to improve safety, reliability, and efficiency.

In conclusion, with the task-specific CAN Manipulator, the mine is able to better plan daily tasks with the knowledge that the machine will be available throughout a shift, and the employees will be safer because of the machine's ergonomics, stability and safety. This has resulted in better productivity and a safer environment.

INTRODUCTION

Twentymile Coal is located approximately 19 miles southwest of Steamboat Springs in Colorado, USA. The mine opened in 1983 as a room and pillar operation. They began developing the longwall in 1987; the face was commissioned in 1989.

Twentymile is one of the largest underground mines in the United States. It has a workforce of approximately 470 and extracts coal from the Wadge seam, which averages 9 feet in thickness. The mine has one of the fastest, most powerful shearers in operation, and the longwall panels are 1,000 feet wide and more than three miles long. The mine's crews can produce 22,000 tons of coal a day.

Twentymile was the first to develop the scissorveyor belt technology, which reduces the number of belt moves required on the longwall. The longwall operation has grades up to 25%. The height of the mine ranges between 8 feet and 12 feet, and the width of the mine entries is 18 to 22 feet.

There are a number of different types of roof control technologies, and they differ from mine to mine. The decision largely depends on the type and condition of the mine. Some of these are roof bolts, cable bolts, mesh and standing roof supports. One type of standing roof support is the CAN. The CAN is a yieldable secondary roof support structure. The CAN incorporates a cylindrical design and is available in a variety of diameters and vertical heights. They are manufactured to accommodate the site-specific characteristics of a particular mine.

The CAN was developed by Burrell Mining Products. Burrell started in the mining industry in the 1980's by selling mortar and concrete mixes to the mining industry. Burrell advanced to selling Burrell Bond mortar; a fibered Gunitex mix. The next

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step in the company's evolution came when Burrell Mining was approached by the U.S. Bureau of Mines in Washington. They were challenged to come up with a roof support block using the fiber technology Burrell was already making. The result is now known as the Fibercrib. Burrell was also making the Omega Block for underground ventilation devices. This block was light weight for easier handling. Burrell decided to combine the best of these items into one design, the CAN. The CAN has a yieldable confined core of cementitious composite encased in a cylindrical steel canister, which resembles a can.

Longwall mines frequently use secondary roof supports in headgate and tailgate entries (Figure 1). The most common standing support system used in the United States is the CAN; approximately 65 percent of the longwall tailgates in the nation use this type of support.



Figure 1. Tailgate support.

There are many advantages to using the CAN; no support exists that can handle floor heaving and roof sag better, and CANs do not fail violently like a rigid standing supports such as concrete donuts (and they weigh less than the concrete donuts). The CAN handles lateral floor movement better than a pumpable crib, and the material handling problems associated with stacking donuts or assembling wooden cribs is negated. A CAN will actually give mine personnel a visual indication if any loading is taking place in a given area of the mine. The outside metal shell of a CAN will wrinkle when taking compression from roof or floor movement; it will handle 50 percent convergence without failure. The smooth, round shape of the CAN assists the ventilation system due to its low friction factor in mine airways when compared to other structures, such as cribs. Besides being used in longwall headgates and tailgates, CANs can be used in bleeder systems, escapeways, air shaft bottoms and heavy areas that are susceptible to squeezes.

Coal mining is a continually changing industry. Mining conditions, installation and removal processes, safety and productivity continue to improve with each year. In underground mining, roof control is heavily involved in all these processes. Coal companies are always searching for something better, which led to the introduction of the CAN. With some new products, the equipment used to handle them has not caught up with the task.

Coal miners are some of the world's best at taking a piece of equipment designed for something else and adapting it for their use; sometimes this works out, but other times the equipment is just not correctly suited for the job. The installation of the CANs fell into this category.

Twentymile Coal decided to switch from donuts to the CAN as a secondary support in 2003. They were using a piece of equipment not specifically designed for handling the donuts or the new CAN, a machine that had been converted from a surface piece of equipment. Since this machine was not designed for this particular job, it broke down frequently and was unstable with the loads being handles.

Twentymile Coal approached J.H. Fletcher & Co. in the spring of 2011 to see if a machine could be made to handle these new CANs. The machine had to be stable enough to handle the grades, strong enough to lift and support the loads of the CANs, tram with the CANs, and was reliable, ergonomic and compact enough to fit in the area for setting the CANs. It also had to be able to pick up a CAN and set it anywhere in the entry.

Twentymile Coal is one of the most productive longwall mines in the world, so it was very important that the machine help with production and not hinder operations – such as was being experienced with the current machines. The machine was to be used outby and have a diesel engine package, and machine speed was to be around 70 feet per minute.

The CANs were delivered and placed on the floor along the rib of the entry, running parallel with the entry. The CAN Manipulator was to then pick the CAN from the floor, lift and rotate it to a vertical position, tram it and then place it in position in the proper location. After the CAN was put into place, it was to be held in place and wedged on top with a combination of cribbing, caps and wedges.

Average installation (using two of the previous machines) was 15 CANs per shift unless all dedicated sources were applied, then 30 placements could be achieved. The CANs are set at 11ft from the rib to the edge of the CAN. Each CAN is 10 feet center-to-center. Typically, three CANs are set on the opposite side at the seals and also if a wide corner is encountered. The machine must be able to tram between the staged CANs against both ribs. In addition to picking up the CANs and setting them along the rib, the CAN Manipulator must be able to set them against the seal (Figure 2).

Chassis

The chassis size was limited to 6 feet, 1 inch in height in order to maneuver around in the mine and travel through low areas. The width of the machine was limited to 96 inches to allow room between the CANs and the opposite rib. The machine had to have at least 12 inches of ground clearance for debris. The overall length was restricted to 26 feet. Speed was to be 90 fpm, as the machine was intended to set CANs at slow pace and allow miners to walk along at the same speed. The machine was to have an outby permissible diesel package for ease of use.

The operator was to set in a cab on the right side of the chassis for visibility on the side that the CANs would typically be set (Figure 3). The cab had to have a MSHA certified canopy over it.

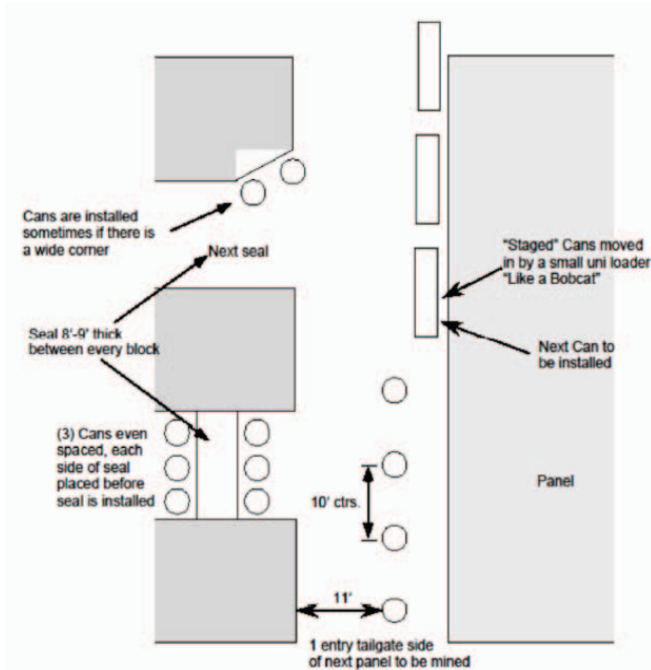


Figure 2. Setting Can supports adjacent to mine seal.

The cab had to be ergonomically designed so the operator could get in and out easily and have a good view of the CANs being handled. The machine functions were placed so that they could only be controlled from inside the cab area, and the control valve and engine panels were put in the cab within easy reach of the operator in order to maintain a good controlling environment. A back-up camera screen was also mounted in the cab to provide good visibility of the rear area when moving in that direction. Pressure gauges were mounted in a fashion to permit the operator to see them for verification that all pressures were normal. The seating area was also ergonomically designed for easy access in and out of the cab. Twentymile operators came to the J.H. Fletcher plant in Wurtland, KY during construction of the CAN Manipulator and advised on the placement of the seating area. With the previous machine, the operator was introduced to a slipping hazard by being forced to step on crawlers before stepping into and exiting the cab.

Twentymile Coal and J.H. Fletcher decided a Cummins 4.5 liter, 130-horsepower engine with a DST exhaust filtration system would be used. This helped by keeping the engine and exhaust components the same as those used on other equipment. The engine and components were installed on the rear of the machine, away from the operator, to reduce noise, improve visibility and provide ease of maintenance.

The tram system would be a set of 96inch-long, 16inch-wide J.H. Fletcher crawlers to keep the height down, handle the weight and maneuver over ground conditions. The chassis was to be strong and heavy enough to handle the loads of the CANs at any distance or angle that was possible with the machine. Removable stab jacks were added to the machine for maintenance purposes. A separate control valve was added to the side of the machine for controlling these jacks so the maintenance personnel could operate these while in view of the machine.



Figure 3. Operator located on right side of machine for enhance visibility.

Boom

The boom was an important element of the machine's design. It had to lift and extend enough to maneuver the nine-foot-long CANs when they were vertical. After doing some modeling, J.H. Fletcher decided on 105 inches of lift and 22 inches of boom reach. This would allow the operator a lot of flexibility in clamping the CAN while still being able to maneuver it around. It was decided to mount the boom in front of the operator for visibility.

A swing of 180 degrees was required for the operator to be able to pick up the CANs lying on one rib and set them on the opposite side at the seals. This had to be done without turning the machine. Having the boom mounted in the front part of the chassis allowed the use of a rotary actuator, and this eliminated the room required for cylinders to swing the boom or the need for a bearing on which to mount a boom pivot. In selecting the correct rotary actuator, the amount of weight of a CAN at the maximum distance was used and the mounting style was taken into account.

J.H. Fletcher tackled the need for picking up a CAN lying horizontally by adding a tilt cylinder from the boom to the clamping-mounting plate. By doing this, the operator can lower the boom and tilt the clamping device to pick up a CAN from the ground, but also allows the tilting of a CAN when stacking them vertically. The tilt is independent of the clamping device.

External cylinders were used on the boom for lift, extend and tilt. This style was used for ease of maintenance.

Clamp

One of the single most important designs on the machine was the clamping device. One common practice on previous machines was to take a paper mill clamp design and adapt it to the boom. In this instance, we decided to design and build our own clamping device in order to make it rugged enough to handle mine condition with the CAN.

The clamp needed to open wide enough to accommodate a wide variety of situations when picking up a CAN, and two cylinders were used to control the opening and closing of the clamp arms.

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The cylinders were positioned so they were protected from possible damage when manipulating CANs.

One problem encountered in the field with using the paper roll clamp arms was that the CAN could slide out. This led designers to look at installing abrasive tape or a rubber-type coating of the arms, but ultimately the best choice was to weld on grip spots across the length of each pad. By welding on the grip spots, the mine did not have to continually replace the worn-out abrasive tape or recoat the arms with rubber. Twentymile Coal sent a CAN to use while building the machine; this helped greatly in the design. When gripping the CAN, we tested picking it up with the clamps and holding it in all positions (Figure 4). We set the pressure on these clamp cylinders, where the gripping spots would make indentions into the metal sleeve of the CAN but not crush the material inside. This prevented the CAN from slipping out.



Figure 4. Testing manipulator ability to hold Can support in any position.

With the existing CAN handling machine, one big failure point was the slew drive. The mine was failing one slew drive every two months. By working with our vendors, we came up with a strong design for CAN handling. This was achieved by bringing in the vendor early in the design phase; by explaining what we were trying to achieve with the machine, they were able work with us to get the proper drive. The slew drive supplier was given the maximum weight of the CAN and then directed to use a safety factor of 2 for the design. Twentymile needed 185 degree of rotation to set CANs, so stops were welded in place to keep the unit from turning more and twisting hoses. The result was a reliable, strong slew drive.

Controls

In order for the operator to maneuver the machine with safety and efficiency, the controls needed to be laid out in a fashion that was familiar and easy to use. After consulting with Twentymile Coal, the controls were placed directly in front of the operator at a height suitable for a variety of operators but did not impede their view (Figure 5).

At first, the control directional movements for the functions were laid out to be like a roof bolter. On that unit the control functions are laid out so that if an operator falls into the lever, it directs the

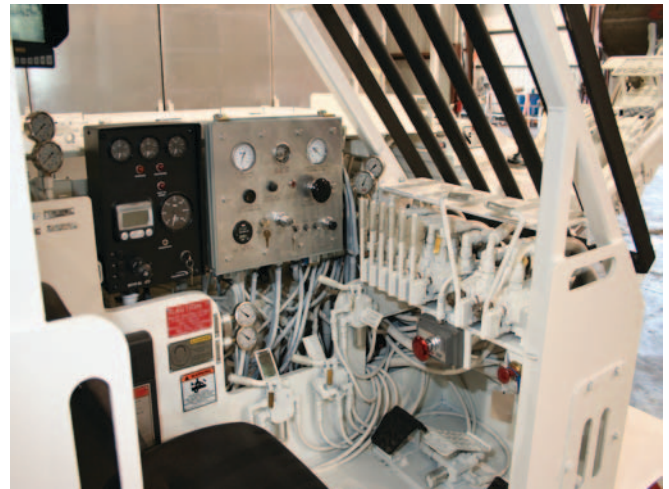


Figure 5. Operator controls.

motion away from them. After Twentymile operated the machine, it was their suggestion to change these directions to resemble the functions on an excavator and other similar equipment. This was done with safety in mind so that the operators did not have movement in a direction they did not expect.

Safety

The primary concern when building the machine was safety. All aspects of the machine were discussed at length by Twentymile Coal and J.H. Fletcher & Co. to prevent any accidents that might occur. The machine had to keep the operators and those around it in a safer environment.

The machine speed was decided to be split between a higher speed for tramping without a CAN and a lower speed when a CAN was in the clamp. This would introduce a much safer environment when having a heavy, bulky load on the front of the boom.

This was achieved through the hydraulic circuit. When pressure was applied to the clamp cylinders, a pilot signal was sent to the tram motors to shift them into low speed, about 28 fpm. After releasing the clamps, the machine shifts back into high speed.

Once the machine had the CAN clamped, we had to look out for the operator's safety and those helping set the CANs. In order to prevent accidental release of the CANs, safety measures needed to be installed. The best avenue was to install a two-hand clamp retract valve. This prevented the operator from accidentally bumping the retract lever with his body, prevented the operator from mistaking this lever for another, and made sure the operator made a conscious effort to release the CAN.

It takes two people to set the CANs: one operator and one helper to direct and crib the CAN. With the old machine, there was no indication if it was safe for those involved to properly position or set. Lights were added to the machine to alert the operator and helper to the conditions on the machine.

When setting CANs with the previous machine, there were no indications if the CAN was clamped and held into place. If the cylinders or valves leaked, the CAN could fall out and cause possible injury to those in the area. We decided to add a pressure

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switch to the clamp cylinders that would turn on a blue light for all to see. The blue light indicated the CAN was clamped and the pressure was holding.

After the operator picked up the CAN and set it into the proper position as indicated by the helper, the operator sometimes would get out to assist in its placement; this was introducing a hazard, since there was no way to determine the hydraulics were disconnected. To counter this, the operator would normally shut the machine down each time the CAN was being cribbed, causing a delay in setting time, engine wear and tear and increased expense. A green light was installed on the boom to show all around that the hydraulics was disconnected and it was safe to enter the area.

Parts

While setting CANs with the old machine, the mine had to have a spare on standby and in close proximity. This served two purposes; if the second machine was operational, it could be put into service when the first one broke down, or it could be robbed of parts to get the first one operational. Because the machine broke down frequently, parts were a priority. However, since this was a piece of equipment the mine had adapted to use underground, the parts were unique and not common.

When designing the CAN Manipulator, spare parts were taken into account. The crawlers were the same as on a J. H. Fletcher & Co. roof bolter, so the maintenance personnel were familiar with them. Most of the cylinders and components were the same as on the roof bolters. This cut down on spare parts inventory since they could be used on multiple machines.

DISCUSSION TOPICS

With any design, there is room for improvement. It could be in the process of using the equipment, or even the equipment itself. There will also always be hazards in mining. It is up to us to work together to make these hazards as minimal as possible while not making the job impossible. Do we need to look closer at handle movements as it relates to each individual piece of equipment,

or do we need to make a standard across all similar equipment? Can we come up with a better way to wedge CANs to help the operator's assistant? This might involve a remote platform or other device. Should a permanent leveling display be put on the machine to show the position of the CAN? What other hazards could we eliminate?

SUMMARY

The development of the CAN Manipulator was essential because many mines are troubled with purchasing construction equipment and adapting them for underground use. This can result in loss of time, money, and sometimes an unsafe work condition.

Without machine breakdowns, personnel normally tasked to work on broken machines could be assigned to work in other areas of the mine. To date, there has been a 35 to 40% reduction in downtime. This helped overall production and maintenance.

The machine was designed first with a focus on safety. We feel this was achieved by adding all the indication lights, lock-outs and ergonomics. The Fletcher CAN Manipulator reduced cycle time by reducing the extra steps and precautions miners had to previously take when setting CANs. Since the introduction of the CAN Manipulator, the limiting factor of the number of CANs set per shift has transferred from the equipment to the supply chain.

With the ever-changing mining environments, it is crucial that the customer and the manufacturer work together to come up with the safest and most productive equipment for the job instead of always adapting existing equipment not necessarily meant for the application. Without the help of Twentymile Coal and Burrell Mining Products, we could not have had the success we did. We feel the Fletcher CAN Manipulator has proved this.

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Longwall Shield Recovery, Using Phenolic Foam Injection for Gob Control As An Alternative to Recovery Mesh

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INTRODUCTION

Following a methane ignition in the active panel of a longwall coal mine, the affected panel was successfully isolated from the rest of the mine. This allowed for the remainder of the mine to return to regular operation, and the conventional practice of having to seal the portals for a defined period of time was avoided. After the affected panel had remained isolated for a regulatorily mandated period of time, it was reopened in preparation for longwall face recovery. As it was not possible to move the shields from their existing positions to facilitate the installation of recovery mesh prior to shield recovery, alternative methods had to be considered to stabilise the gob and minimise flushing, thereby improving operator safety.

The objective of the paper is to show how stabilisation of the gob was achieved during shield recovery, thereby minimising flushing, and improving operator safety.

Phenolic foam was chosen and injected from between the shields into the gob behind them. This is a technique that has been used to great effect in Germany, to a lesser extent in Australia, but not knowingly to date in the United States. To ensure nothing was left to chance and thereby guarantee the project's success, the procedure used in Germany was employed, down to the application of equivalent hardware, which was sourced through the North American supplier.

PANEL SEALED OFF FROM SURFACE

Immediately after the ignition the mine was successfully evacuated in a safe and timely manner, without incident or injury. Within four hours of the occurrence, nitrogen was being injected directly into the area of the ignition, in quantity and from the surface (Figure 1). Full control of the situation was established within 48 hours of the incident, at which point the mine was faced with an exciting and stimulating technical possibility: if the affected longwall panel could be successfully isolated from the rest of the mine, then the remainder of the workings could be returned to regular operation and the conventional practice of having to seal the portals for a defined period of time could be avoided.



Figure 1. Nitrogen injection on gob vent borehole.

Cementitious and foam products were tested to determine their ability, and suitability, to seal the active longwall panel remotely from the surface (Figures 2 through 6).

Phenolic foam was chosen, and the longwall panel was successfully sealed with five in-mine ventilation control structures, remotely placed from the surface at depths of approximately 750 to 900 ft (230 to 275 m). The first two structures were placed using two injection boreholes and two video camera boreholes each (Figures 7 and 8). The last three structures were each placed with a single injection borehole and a single video camera borehole.

Initial evidence of the effectiveness of the sealing came from video footage and differential pressure gauge readings. This was corroborated with direct observation and measurements underground by the mine rescue teams when the mine was reentered.

After the in-mine ventilation control structures had been emplaced, they were backed up with stoppings and approved 50 psi (0.34 MPa) seals.

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Figure 2. Test trench.



Figure 5. Finish of phenolic foam testing.



Figure 3. Test trench with nitrogen inertisation.

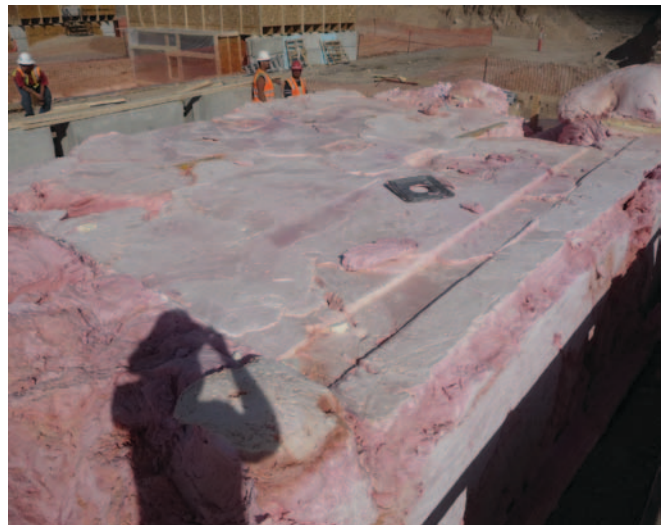


Figure 6. End result of phenolic foam testing.



Figure 4. Start of phenolic foam testing.



Figure 7. Lowering hoses from surface.



Figure 8. Pumping from surface.

Success was achieved in the face of both technical and historically driven adversity. In spite of repeated assertions that there was little chance of success, perseverance and demonstration of proof of concept prevailed and delivered a product and technique that not only worked, but did so better than expected.

PANEL REOPENED UNDERGROUND

After the regulatorily mandated period of time had elapsed, access and ventilation were reestablished to the affected longwall face in readiness for gob consolidation. At the time of the ignition, the longwall face was approximately 3,100 ft (945 m) short of its planned recovery line, and the headgate drive and crawler-mounted tailpiece were partway along a chain pillar between crosscuts. A new set of recovery chutes were driven, and the pillar was split. The 50 psi (0.34 MPa) seals, stoppings and in-mine ventilation control structures were breached (Figures 9 through 12).



Figure 9. Test breaching 50 psi (0.34 MPa) seal.



Figure 10. Test breaching 50 psi (0.34 MPa) seal.



Figure 11. Phenolic foam underground.



Figure 12. Admiring our handiwork!

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GOB STABILISATION

Once access and ventilation were reestablished to the affected longwall face, the process of gob consolidation commenced. Under conventional recovery conditions, recovery mesh would have been used. However, as this was not an option in this case, because it was not possible to move the shields from their existing positions to facilitate the installation of recovery mesh prior to shield recovery, alternatives methods were considered to stabilise the gob and minimise flushing, thereby improving operator safety.

Building on the success already achieved with phenolic foam for panel isolation, it was decided to use it again for another of its designated purposes: gob stabilisation (Figure 13). The technique has been used to great effect in Germany, to a lesser extent in Australia, but not knowingly to date in the United States. Additionally, it already had a proven track record at the San Juan Mine for void filling (Panel 202 tailgate fault (August and September 2007), the Panel 201 headgate fall (November and December 2008) and the Panel 301 headgate fall (November and December 2010)), and had just been tested again and approved for use under the federal 103(k) order outstanding from the ignition.

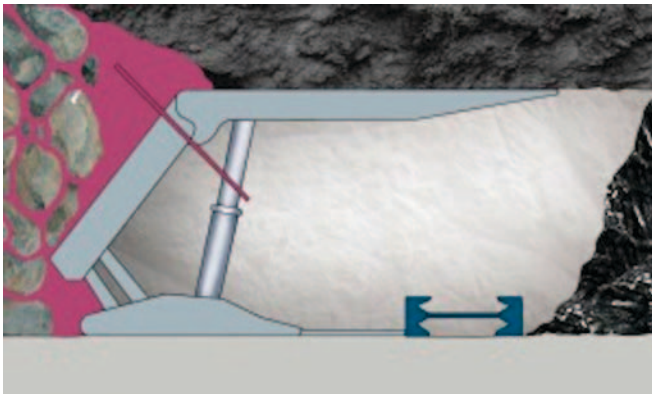


Figure 13. Concept of gob stabilisation using Rocsil foam (A. Weber SA, 2013).

Accordingly, phenolic foam was injected from between the shields, into the gob behind them. To ensure that nothing was left to chance, and thereby guarantee the continuing success of the project, the procedure used in Germany was used, down to the use of equivalent injection hardware.

PHENOLIC FOAM AND INJECTION HARDWARE

The phenolic foam chosen was Rocsil® foam. It is manufactured by A. Weber SA in Rouhling, France, and represented in the US by Jennchem. It is a two-part foam, consisting of a resin and a catalyst. The theoretical mixing ratio is four parts of resin to one part of catalyst, but under actual conditions this ratio is typically nearer to 3.5-3.6:1. The expansion ratio and finished product density depend upon a number of factors, and are typically in the ranges of 20-30:1 and 1.9-2.8 lbs/ft³ (30-45 kg/m³). Compressive strength at 10% deformation is 14.5-29 psi (0.1-0.2 MPa).

The injection hardware consisted of externally threaded hollow bar, 1-5/8 in (42 mm) button bits and threaded couplers. The bar has an external diameter of 1-3/16 in (30 mm), an internal diameter

of 5/8 in (16 mm), and was supplied in 10 ft (3 m) lengths. Some of these were then cut into 6 ft (1.8 m) and 4 ft (1.2 m) sections. This system was used at the recommendation of A. Weber SA, as it was equivalent to that used to great effect in Germany. It was sourced from Europe through the North American supplier.

The initial single-use 'sacrificial' drill strings consisted of a 1-5/8 in (42 mm) button bit, a 6 ft (1.8 m) section of bar, a threaded coupler and a 10 ft (3 m) section of bar. This allowed for the operators to stand back under the safety and relative openness of the canopies when drilling, rather than having to work in the more confined areas around the legs.

DRILLING

Drilling was accomplished with jackleg drills, striker bars and threaded couplers. Water flushing, with a flow rate of at least 2 gpm (7.6 lpm), was used to prevent any sparking or heat-related methane ignition potential. None occurred.

It was intended to drill two holes between, or at the edge, of each shield. An upper one, towards the back of the canopy/caving beam and at an angle of approximately 45°, and a lower one at chest height straight back into the gob. Holes were initially drilled to a depth of approximately 6 ft (1.8 m). Each drill string was drilled in until the proximal end of the threaded coupler was flush with the back of the shield/gob line. The drill string was then disconnected from the threaded coupler on the end of the striker bar and left in position, ready for injection. Further threaded couplers were modified by the addition of staple-lock couplers to one of their ends for connection to the mixing gun. The mixing gun could then be connected to these modified threaded couplers with a single staple, and pumping started. This system allowed for a safe and simple transfer from one drill string to the next (Figure 14). Once pumping on a specific drill string was finished, the proximal 10 ft (3 m) section was then wound back out of the threaded coupler whenever possible, leaving the distal 6 ft (1.8 m) section behind in the gob. This helped to clear the working area of unwanted obstructions, and minimised the potential risk of drill string hang-ups on the shields when they were pulled, and then possibly getting thrown around the work area. It also allowed the remaining 6 ft (1.8 m) sections to act as 'rib' bolts, thereby helping to reinforce the gob (Figure 15).

Where shields pairs were so close together that one or the other, or both, of the holes could not be drilled between them, additional foam was pumped into adjacent drill strings on either side, or both sides, to compensate.

FOAM CALCULATIONS

The shields at San Juan Mine are 5 ft 9 in (1.75 m) wide, and at the time of the ignition were set to a height of 10-12 ft (3-3.7 m). Initial calculations for the volume of foam required behind each shield were based on a width of 6 ft 7 in (2 m), a set height of 13 ft (4 m), a desired gob consolidation ('wall') thickness of 6 ft 7 in (2 m), and an assumed void space in the gob of 50%. The shield width and set height were both deliberately chosen to be on the conservative side, and the void space was reckoned to be overly so. This gave a volume of approximately 280 ft³ (8 m³) per shield, or approximately 140 ft³ (4 m³) per drill string. At an assumed mixing ratio for the resin to catalyst of 3.5:1, this equated



Figure 14. Pumping and changing drill strings.



Figure 15. Drill strings in gob between shield pairs.

to approximately 4 drums of resin at a density of 2.5 lbs/ft³ (40 kg/m³), or approximately 4.5 drums at a density of 2.8 lbs/ft³ (45 kg/m³). The decision was made to start with five drums of resin per drill string.

FIRST FOUR SHIELDS

The first four shields on the tailgate end of the longwall face were pumped with 108 drums of resin in 9 drill strings, equating to approximately 12 drums per drill string, with no show of resin anywhere on the face, or between or into the shields. As a consequence additional resin and catalyst were ordered to cover the new anticipated maximum usage of 15 drums of resin per drill string. The first four shields were pulled without incident. Unfortunately, no photographs of this were taken, but the tailgate corner had a near perfect right-angle at the contact between the roof and the gob in the tailgate entry.

REFINEMENT

Having seen no show of resin with the first four shields, drill strings were then tried at depths of approximately 3 ft (0.9 m) and approximately 6 ft (1.8 m) into the gob to see if there was any change or improvement. If they were drilled too far into the gob,

then it might have created the potential for better consolidation of the gob and a thicker 'wall' further back into the gob, thereby reducing the possibility of any major flushing or failure of the gob. However, at the same time, it might have created poorer consolidation of the gob immediately behind the shields, thereby increasing the possibility of some minor flushing or dribbling.

Conversely, if they weren't drilled in far enough, it may have created the potential for better consolidation of the gob immediately behind the shields, thereby reducing the possibility of some minor flushing or dribbling. At the same time, it may also have created poorer consolidation of the gob and a thinner 'wall' further back into the gob, thereby increasing the possibility of some major flushing or failure of the gob, and also lead to possible inundation of the shields. In reality there was no appreciable difference in behaviour observed between the approximately 3 ft (0.9 m) holes and the approximately 6 ft (1.8 m) holes.

STEADY STATE

Within a short space of time, a steady state practice was established whereby a maximum of 15 drums of resin were pumped through each drill string. This was unless additional resin was being pumped to compensate for adjacent drill strings in the pattern that could not be drilled between shield pairs due to the shields' proximity, or unless inundation of the shields with foam occurred prior to the desired quantity of resin being reached. In the former case, the quantities were increased by differing amounts based on the different possible scenarios. These could include 'partner' drill strings missing between one shield pair, adjacent drill string missing on one side or other, adjacent drill string missing on both sides, adjacent drill strings missing on one side or other, adjacent drill strings missing on both sides, etc. The maximum number of drums of resin pumped into any one drill string was 50. This was the upper drill string between shields 5 and 6 on the headgate end, and even then it only showed a modicum of return into the shields before pumping was halted. At the end of the foam injection phase of the project, a total of 3,305 drums of resin, and 904 drums of catalyst had been pumped, at a ratio of 3.7:1. Close to the original estimate of 3.5-3.6:1.

FACE WIDENING

The shearers, drives and panline were recovered once gob consolidation had been effected. The face was then widened by about 10 ft with a continuous miner, bolted and dusted to provide sufficient clearance for shield recovery (Figures 16 through 19). Widening was done one section at a time: tailgate to Chute 1, Chute 1 to Chute 2, Chute 2 to Chute 3, and Chute 3 to headgate. To prevent damage to the shields, the left hand end of the miner drum was modified.

SHIELD RECOVERY

Once the face had been widened, bolted and dusted, the shields were recovered. As with the face widening, shield recovery also took place one section at a time. Shield recovery was carried out in the same manner as it would have been for a 'normal' recovery, using a mule and a pair of walkers.

Due to the size of the project, and the number of holes that needed to be drilled, it was inevitable that there would be some



Figure 16. Miner tramming to take cut.



Figure 17. Cut taken.



Figure 18. Bolting.

missing drill strings at various locations along the face. In these areas, where the injection of foam was maybe not quite as uniform as planned, the recovery face conditions were good. Only a minimal amount of flushing occurred in isolated pockets, and there was never any inundation of the face with gob. As shields were pulled, the roof above them also stayed in place. Where the



Figure 19. Dusting.

injection of foam was as uniform as planned, the recovery face conditions were excellent. So good, in fact, that the gob was held back at a negative 'angle of repose', with the backs of multiple shields quite clearly imprinted in it. Most importantly, all the conditions were significantly better than they would have been without recovery mesh or foam. Without either of these, gob flushing would have been almost inevitable, and the face could have been inundated (Figures 20 through 27).



Figure 20. Walkers at Shield 116.

CONCLUSION

Success was achieved in the face of technical adversity. In spite of the fact that this method of gob stabilisation had not been knowingly successfully demonstrated in the United States to date, adherence to the procedure and hardware successfully used in other coal mining regions of the world delivered a product and technique that not only worked, but did so better than expected. It is not very often that you can get to see a negative 'angle of repose' on the longwall gob.

The entire face was recovered without incident, accident or injury of any kind. It serves as the most fitting testament and legacy of the efficacy of the foam as an alternative to the use of recovery mesh for longwall gob consolidation.



Figure 21. Minor flushing.



Figure 24. Walkers at Shield 59.



Figure 22. Minor flushing, and excellent gob control.



Figure 25. Minor flushing.



Figure 23. Minor flushing, and excellent gob control.



Figure 26. Minor flushing.

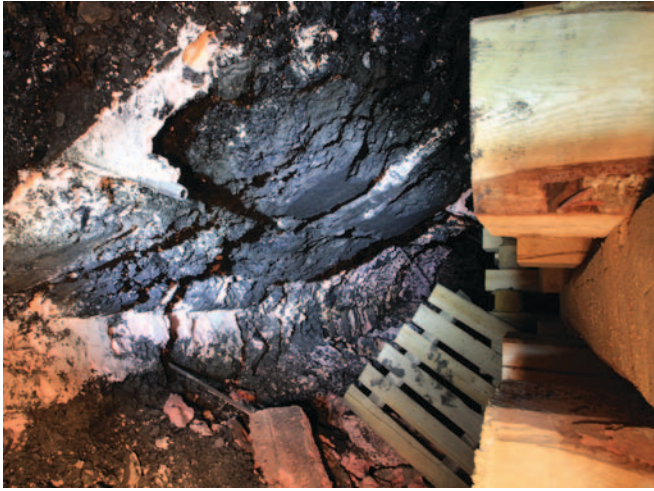


Figure 27. Negative angle of repose?

Effects of Draw Slate Thickness on Longwall Mining

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ABSTRACT

The Pittsburgh seam is the most extensively mined coal bed in the Appalachian Basin. Unlike most other prominent coal seams, the Pittsburgh seam typically includes a sequence of immediate roof coals and intervening rock binders (or “draw slate”), most of which are within mineable horizons. Furthermore, unlike the main bench, the thickness of the overlying roof coals and their intervening rock binders can vary significantly. For instance, in southwestern Pennsylvania, the Pittsburgh seam main bench typically ranges from about 5.0 to 6.0 feet thick with minimal variation over short distances. On the other hand, the Pittsburgh seam roof coals and immediate roof binder typically range from a few inches to several feet, and in certain geologic settings can vary significantly over short distances. Additionally, in some locations, very thick draw slate can totally replace the roof coals. Since the draw slate, which typically consists of claystone or clay-laden shale, is relatively weak, the presence of thick and variable draw slate in the roof and rib can significantly affect mine safety and productivity. This paper presents the adverse effect of thick draw slate on a 1500-foot-wide longwall face in a southwestern Pennsylvania coal mine. Mapping of the thick draw slate, numerical models which illustrate stress distribution and failure associated with thick draw slate, and operational adjustments instituted to mitigate the negative impact of thick draw slate will be discussed in detail.

INTRODUCTION

Longwall mining is the safest and most productive underground coal mining method. However, even the best designed, maintained and operated longwalls can be significantly impacted by poor near-seam geology such as paleochannels, faults and thick draw slate (Su et al., 2004, 2009).

Ten out of eleven CONSOL Energy longwall faces are located in the Pittsburgh seam. The Pittsburgh coal bed typically consists of a 5.0 to 8.0-foot main bench, a draw slate of variable thickness, and overlying roof coals ranging from a few inches to several feet. In October 2012, one of CONSOL Energy’s southwestern Pennsylvania longwall panels encountered difficult face conditions starting about 3,124 feet in by the longwall recovery. Upon

examination by the CONSOL Coal Operation Support personnel, unusually thick draw slate was determined to be the main culprit.

Draw slate is a layer of claystone or clay-laden shale between the main bench and overlying roof coals, which is structurally weak and degrades rapidly when exposed to moisture. Under typical conditions, this Pittsburgh seam longwall, Panel 15, extracts the entire main bench (5.0 to 6.0 feet thick), the entire draw slate (6 to 12 inches thick), and the bottom portion of the roof coals, leaving about 6 inches or more roof coal to help maintain roof integrity. However, in the affected area of Panel 15, the draw slate thickened to 3.0 to 4.0 feet, and the resulting cumulative height of the main bench and draw slate significantly exceeded the 90-inch maximum reach of the face shields. This resulted in severe roof pot out at the face from Shield 120 to the headgate.

After a couple of weeks of difficult mining, in early November 2012, CONSOL Energy’s Coal Operation Support personnel were requested by mine management to investigate the thick draw slate condition, project trends and help mitigate future impacts. This paper presents the results of detailed in-mine mapping and longwall monitoring, numerical analysis designed to evaluate the effect of thick draw slate on longwall face stability, and the ensuing operational adjustment undertaken to mitigate the effect of thick draw slate on the longwall face.

GEOLOGIC SETTING

The thick draw slate occurred in a unique geological setting, an area dominated by after coal deposition fluvial processes. Paleochannels flowed through the outby end of Panel 15 in a general northeast-to-southwest trend following the mains and cutting through the roof strata and into the roof coals in the most invasive areas. The sandstone within the paleochannel is very-fine to fine-grained, medium-bedded and averages 10 feet thick and 100-300 feet wide (Figure 1). The presence of the paleochannel within the mains altered the typical roof geology in adjacent easterly areas within Panels 15 and 16. Beyond the sand influence of the paleochannel, an alluvial plain developed within the roofline of the outby area of these panels. Because of the paleo-environment, unconsolidated sedimentary deposits of mud and silt accumulated along the banks of the paleo-river, forming a thick sequence of dark thinly-bedded shales and claystones (Figure 2).

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One manifestation of these thick clay deposits is the presence of a very thick draw slate that roughly parallels the paleochannel.

shift (fps). However, within the 3.0 to 4.0-foot thick draw slate zone (i.e., weeks 3 to 7), face advance rate reduced dramatically

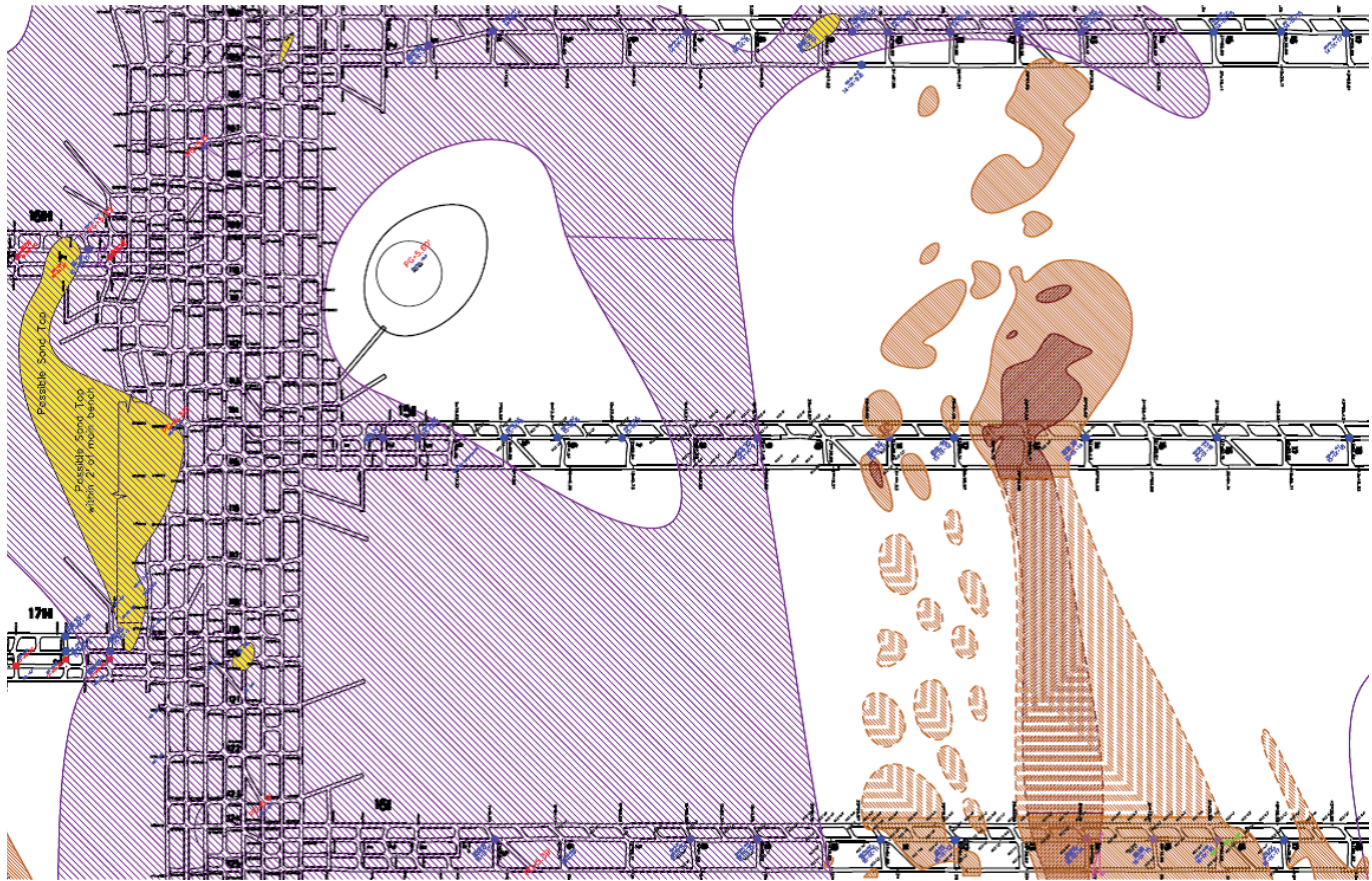


Figure 1. Paleochannel in the mains and the panels.

GEOLOGIC MAPPING AND LONGWALL MONITORING

Initial investigation indicated that thick draw slate was the primary culprit of difficult face conditions within the affected area, which was readily apparent from roof potting and face spall at the draw slate horizon. Subsequent mapping of the longwall face substantiated that face conditions deteriorated to a minor degree in areas where draw slate thickness exceeded 2.0 feet and to a much larger degree where draw slate thickness exceeded 2.5 to 3.0 feet. In addition, the worst face conditions were noted beyond 50 shields from the headgate and near mid-face, likely as a result of higher mid-faced stress. Similar conditions and relationships had also been observed at prior CONSOL longwall faces. Figure 3 shows the extent of thick draw slate within Panel 15. Single-hatched light-brown shaded areas show zones with 2.0 to 3.0-foot thick draw slate. Double-hatched darker brown shaded areas show zones with more than 3.0 feet of draw slate. The green hatched area is a shale channel, where the roof coal is absent in an otherwise normal shale roof.

Figures 4 and 5 show the impact of the thick draw slate on face advance and rock-related delays as a function of time. Prior to and after encountering the thick draw slate, the advance rate of the 1500-foot-wide Panel 15 longwall face was about 17 feet per

to about 6 fps (Figure 4). Conversely, rock-related delays dramatically increased in this area (Figure 5). Roof cavities up to 10 feet high were constantly present from shields 80 to 120, and large chunks of silty shale often fell on the panline, which could not pass underneath the shearer. Polyurethane injection at the face posed significant safety risk to the workers. In summary, presence of the thick draw slate not only severely affected the longwall productivity, but also the health and safety of the longwall crews.

Figure 6 illustrates the relationship between the Panel 15 face advance rates and the draw slate thickness. It is clear from Figure 6 that thickening draw slate correlates well with decreased advance rate. The worst conditions (slowest rate) were encountered in the vicinity of +3.0 feet of draw slate, particularly where the very thick draw slate extended beyond Shield 50. Note that the thickest draw slate (up to 5.0 feet thick) occurred near Crosscut 13 on the headgate side. In this zone, the average shift advance rate was only 6 fps, 11 fps lower than the average for this face in normal geology. When the longwall face moved further outby, sporadic thick (+2.0 feet) draw slate continued to cause minor delays, and the face advance averaged 6 to 14 fps. In general, this analysis showed that the draw slate started to affect the longwall productivity when the thickness was more than 2.0 feet. When the average thickness was more than 2.5 feet, productivity and face conditions were severely affected.



Figure 2. Thick draw slate more than 3ft.



Figure 3. Thick draw slate and advance rates in Panel 15.

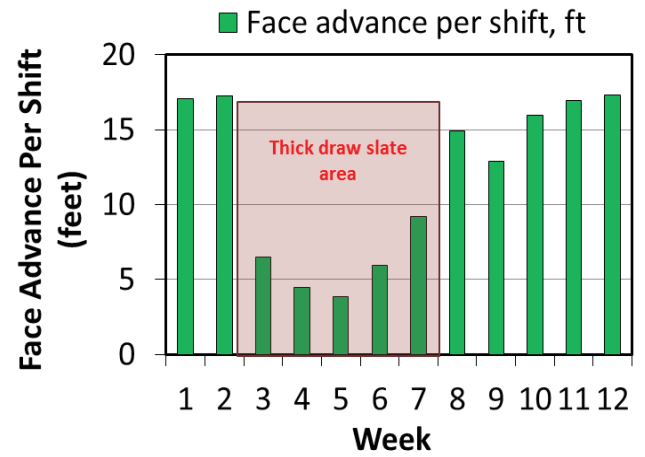


Figure 4. Face advance per shift.

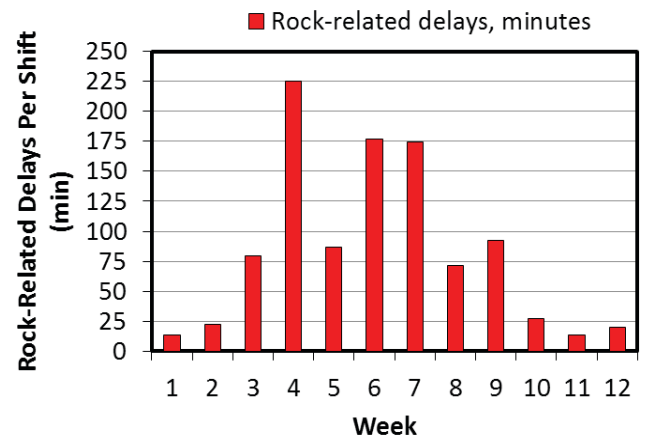


Figure 5. Rock-related delays per shift.

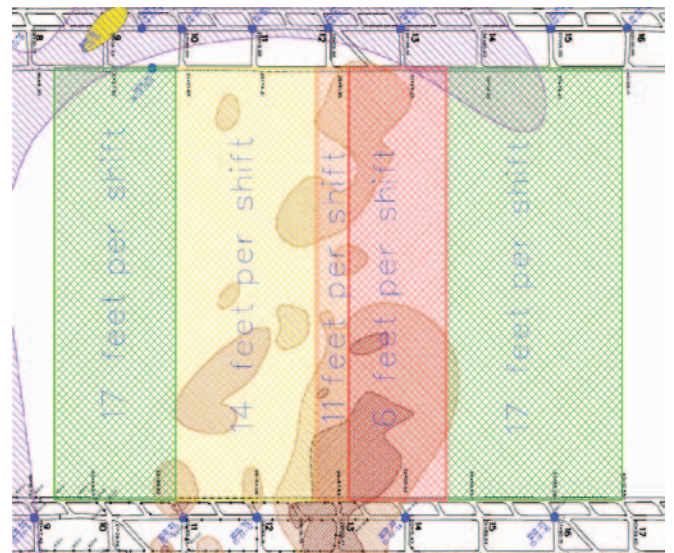


Figure 6. Geologic mapping for thick draw slate and weekly advance rate in Panel 15.

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FUTURE PROJECTIONS AND PROGNOSTICATION

Given the trends and impacts of the draw slate within Panel 15, it became apparent that Panel 16 could also encounter similar thick draw slate, which was readily confirmed by additional in-mine reconnaissance. Figure 7 and Figure 8 show the projection of draw slate thickness and superimposed advance rates for the adjacent Panel 16. Conditions in Panel 16 are expected to be more extensive than those in Panel 15. The +3.0-foot thick zone of draw slate is expected to cross the entire panel, with most of the face exposed for about 250 to 300 feet. In addition, a total “rock” zone width of 1390 feet is projected, extending from about Crosscuts 11 to 16. The most severe portion will extend about 510 feet from Crosscuts 13 to 14.5.

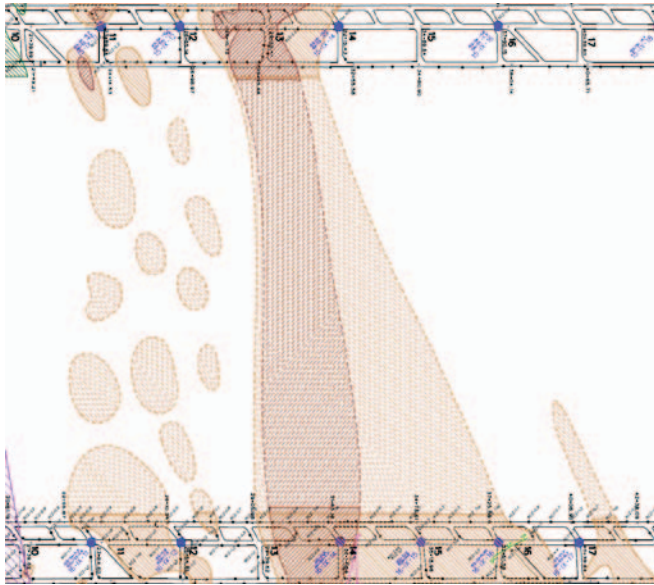


Figure 7. Projected thick draw slate in Panel 16.

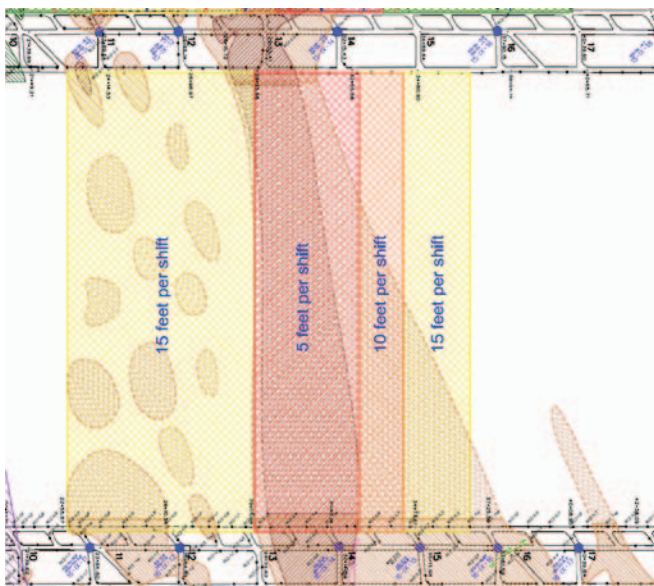


Figure 8. Projected thick draw slate and advance rate in Panel 16.

Based on these mapped trends and retreat rates from Panel 15, retreat rates for Panel 16 could be expected to average 9.5 fps (range from 5 to 15 fps, depending on condition) over the 1390-foot-wide rock zone, if no operational adjustments were made. Alternatively, if conditions can be improved through planning and tactics, then the actual rates should be better than those projected. Such adjustments will be discussed in subsequent sections.

OPERATIONAL ADJUSTMENT

After the 1500-foot-wide Panel 15 longwall face entered into the thick draw slate zone in October 2012, the longwall crew spent the next two to three weeks dealing with face cavity, jammed panline and face gluing. With recommendations from CONSOL's Coal Operation Support personnel, the longwall crew started to implement a shield double-pull procedure within the face areas affected by the +2.5 feet of draw slate. With the double-pull procedure, the shields were pulled forward immediately behind the leading drum, then pulled forward one more time after the shearer had passed by about 100 feet. The first pull-in reduced the tip-to-face distance from 70 to 46 inches and the second pull-in reduced the tip-to-face distance from 46 to nearly 0 inches. As a result, the adverse effect of the thick draw slate was somewhat mitigated. However, since the PM4 shields at Panel 15 possessed a large tip-to-face distance of 28 inches, the effect of shield double-pull was limited. Based on the Panel 15 mining experience and the thick draw slate projection for Panel 16, mine management has decided to deploy a new set of shields with a higher reach of 108 inches and a shorter tip-to-face distance of 20 inches in Panel 16. The employment of the new shields is expected to further mitigate the effect of the thick draw slate and improve the longwall productivity and safety. The next panel is scheduled to mine through the thick draw slate zone in June 2013.

NUMERICAL MODELING

From the geological mapping and face monitoring, it is apparent that the thicker the draw slate, the worse the advance rate. To further understand the effect of the thick draw slate, different thicknesses of draw slate at the longwall face were simulated with Rocscience finite element models. Shield double-pull was also simulated to analyze its effect on the stress distributions at the longwall face.

Table 1 shows the water sensitivity and compressive strength testing results of a few typical in-mine samples (Hasenfus and Su, 2005). Clearly, the draw slate is very sensitive to water and its compressive strength even under dry conditions can be as low as 1,400 psi. The compressive strength of the draw slate in the FE models was reduced from 3,942 psi to 2,000 psi to reflect its in-situ strength and the in-situ shear strength of the draw slate was calculated to be about 1,000 psi. Major rock properties which were used in the models are shown in Table 2. The Mohr Coulomb failure criterion was employed to simulate rock failure.

Effect of thick draw slate

Four models, which have different draw slate thicknesses of 1.0, 2.0, 3.0, and 4.0 feet, were constructed to simulate local roof geology (Figure 9). A 5.0-foot-thick Pittsburgh seam main bench and variable roof coal thickness were simulated in the models. For

Table 1. Draw slate properties testing results from lab.

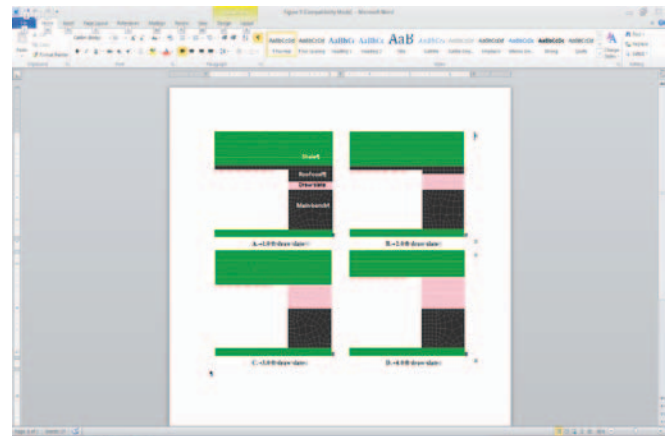
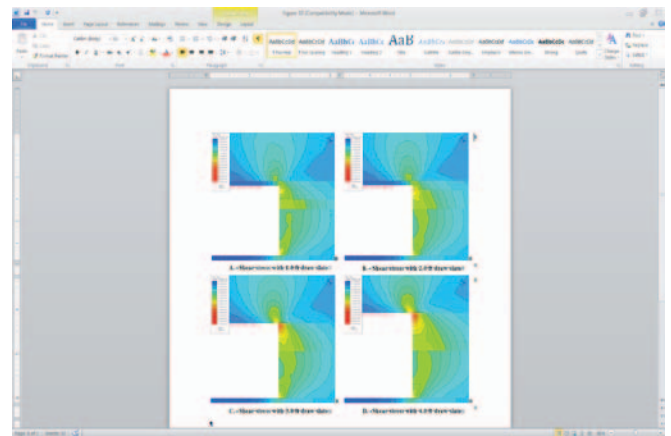
Sample ID	Water Sensitivity Index (%)	Uniaxial Compressive Strength (psi)	Mass Density (pcf)
1	100.0	2,600	152.9
2	100.0	3,300	153.0
3	100.0	2,000	150.7
4	100.0	6,100	167.5
5	100.0	2,400	160.8
6	100.0	3,800	163.6
7	100.0	5,500	154.6
8	100.0	3,600	154.7
9	100.0	6,500	153.6
10	89.6	8,200	149.0
11	100.0	1,400	156.0
12	100.0	1,900	151.9
Average	99.1	3,942	156

Table 2. Major rock properties used in the model.

Rock Type	Young's Modulus (psi)	Cohesion (psi)	Friction angle	Poisson Ratio	Tensile Strength (psi)
Shale	870,229	450	35	0.25	250
Draw slate	400,000	289	30	0.25	50
Coal	210,000	50	35	0.25	50

the models with 1.0 feet and 2.0 feet draw slate, 2.0 feet and 1.0 feet of roof coal were simulated (Figure 9, A and B), respectively. Typically, the mining height was 7.0 feet (84 inches) at the Panel 15 longwall face and 6-8 inches of roof coal were left to protect the roof. However, when thick draw slate was present, roof coal usually became thinner or even totally disappeared. To simulate the worst-case scenario, no roof coal was present in the models with 3.0 feet and 4.0 feet draw slate (Figure 9, C and D). Since the draw slate would not remain stable after being undercut, the resulting mining heights were typically up to 8.0 feet (96 inches) and 9.0 feet (108 inches) for the models with 3.0 feet and 4.0 feet draw slate, respectively. A 70 psi compressive pressure was applied to the roof to simulate the shield supports' setting load density, and a 28-inch tip-to-face distance was simulated to reflect the typical tip-to-face distance of the PM4 shields employed at the Panel 15 longwall face.

Since the shear strength of the draw slate is about 1,000 psi, a 1,000 psi contour line was plotted to express the shear failure zone (green and warmer color) of the draw slate. Figure 10 illustrates the comparison of shear stress within the coal seam and the immediate roof surrounding the longwall face with different draw slate thickness. Comparison of the stress contours clearly indicates that with increasing draw slate thickness, higher shear stress will concentrate at the face and more draw slate will fail and fall onto the panline. Potting out of the draw slate creates larger tip-to-face


Figure 9. FE models with different draw slate thickness.

Figure 10. Shear stress distributions at the longwall face with different draw slate thickness.

distance, which further increases the stress within the draw slate and in the roof.

Figure 10 also shows that if roof coal is present at the face, the maximum shear stress at the longwall face is 1,600 psi and 1,800 psi, respectively, for the 1.0 foot and 2.0 feet draw slate models (A and B). On the other hand, if the draw slate is too thick and the roof coal is no longer present, the maximum shear stress at the longwall face increases to 3,000 psi. Since the draw slate is very weak and not self-supporting, the rest of the draw slate will fail, and the resulting mining height will be about 8.0 to 9.0 feet (96 to 108 inches) for the 3.0 and 4.0 feet-thick draw slate seam conditions, which is above the shields' maximum reach of 7.5 feet (90 inches). Consequently, this portion of the face becomes unsupported, resulting in even higher shear stress (C and D) at the face. Under such circumstances, roof cavity may be formed if the resulting shear stress is higher than the shear strength of the roof rock. Also, since the shields cannot support the roof, more pressure will be transferred to the face and cause more draw slate sloughing and roof pot-out.

Effect of shield double-pull

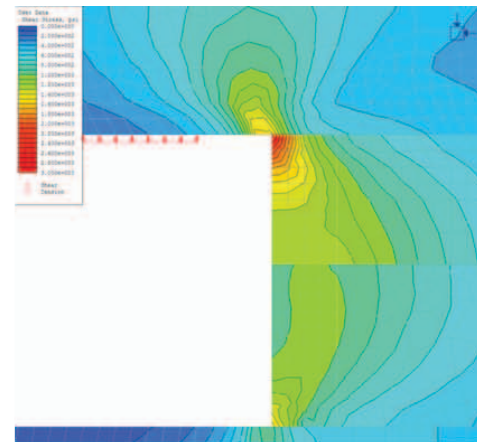
Double-pull of the shields behind the drum is an effective measure to mitigate the effect of the thick draw slate. Since double-pull of the shields would pull the shield forward immediately after the leading drum has passed, it reduces the tip-to-face distance and exposure time of the roof, thus reducing roof stress. Also, after both drums have passed, the shields would be pulled one more time, which further reduces the tip-to-face distance.

Figure 11 shows the shear stress surrounding the longwall face with different tip-to-face distance for the model with 4.0 feet of draw slate and 5.0 feet main bench. If the longwall face employs the normal shield-pulling sequence, the tip-to-face distance is 28 inches. Since the draw slate will not stay after cutting, the resulting mining height will be 108 in, which is above the maximum reach of the shields (Figure 11 A). As a result, more draw slate will fall from the face. Assuming that 2.0 feet of draw slate have fallen from the face, Figure 11 B shows the shear stress distribution at the face, which clearly indicates that both shear stresses in the roof and rib increase significantly and additional pot-out may occur.

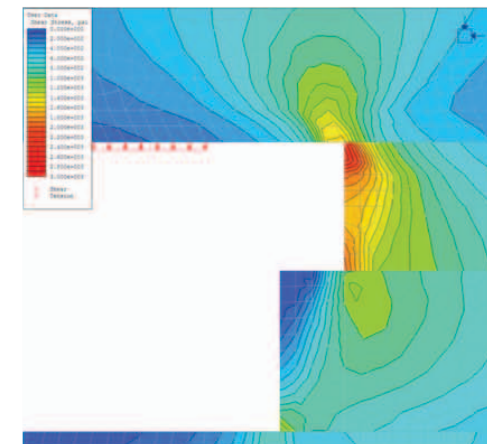
Conversely, if the face employs double-pull of the shields, the shields are pulled in immediately behind the leading drum, which reduces the tip-to-face distance from 70 to 46 inches. Since the shields will be pulled forward one more time after both drums have passed about 100 feet, the resulting tip-to-face distance will be nearly 0 inches. Figure 11 C shows that, with double-pull of the shields, much less shear stress distribution was observed at the face when comparing Figures 11 A and 11 B. Because of the timely shield support, the total mining height may still be kept at 7.0 feet, and the shields can provide effective roof support. Of course, the model in Figure 11 C is an ideal scenario, but it clearly shows that double-pull of the shields would reduce stresses in the draw slate and roof and would improve face conditions.

CONCLUSIONS

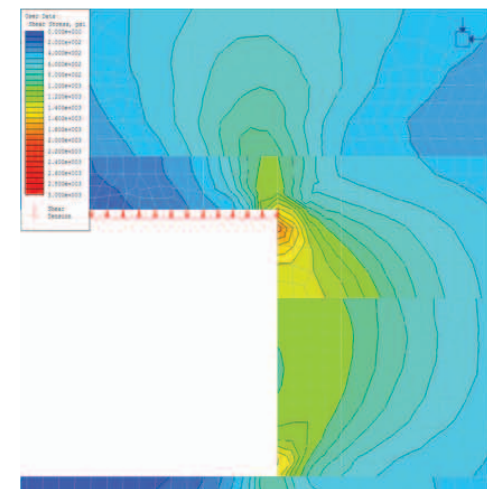
Draw slate in the Pittsburgh seam is generally inconsistent and very weak. When the draw slate is very thick, it will adversely affect the longwall safety and productivity. From CONSOL Energy's longwall mining experience, the critical thickness of the draw slate which may affect longwall safety and productivity is about 2.5 feet. When the draw slate thickness is more than 2.5



a) 28 inches Tip-to-Face



b) After 2.0 ft draw slate spalling



c) 0 inches tip-to-face

Figure 11. Shear stress distributions at the longwall face with different tip-to-face distance.

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feet, longwall productivity will be adversely affected; the thicker the draw slate, the slower the face advance rate. In-mine geologic mapping can be an effective tool for predicting the draw slate thickness in the future longwall panels, which will enable mine management to take proactive actions when mining through those areas.

Results from the numerical models show that as the draw slate thickness increases, higher shear stress will be present at the face and more draw slate will fail. Double-pull of the shields behind the drums will reduce the tip-to-face distance and roof exposure, thus resulting in better roof stability at the longwall face.

With the longwall panels getting wider and wider (up to 1,500 feet), longer times are needed to complete a face pass. This causes a longer exposure time for rock and coal at the face. Since the draw slate is very sensitive to moisture, more draw slate will fall off from the face because of the longer exposure time. In order to mitigate the effect of the thick draw slate, it's important for mine management to maintain a steady advance rate and reduce the face down time, such as weekend shutdowns and belt issues.

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Ground Control Practice at Leeville Underground Mine

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ABSTRACT

The geotechnical properties of mining zones at Leeville Underground mine (or simply called Leeville mine through this paper) are different from one zone to another. The typical ground condition is poor to very poor. Therefore, an appropriate ground control practice is the key for the successful and smooth ore production at Leeville mine. Based on the available geotechnical data, the empirical and numerical methods are used to design the right ground support systems. The main ground control practice at Leeville mine includes geotechnical data collection, evaluation of the stiffness of the support system, selection of rock bolt types, design bolt length and bolting patterns, development of the ground support standards, and performance of quality controls on bolt installation to ensure proper rock anchorage, and strength of shotcrete and backfill. Through all these activities, an appropriate ground support system has been established and applied at Leeville mine for the past four years. So far, a good result related to successful ground control and smooth ore production has been achieved, although the system is still being improved.

The authors of this paper will summarize the ground control practice conducted in the last four years and will discuss the procedures and standards of establishing the appropriate ground support systems for Leeville mine.

INTRODUCTION

The Leeville Underground mine is located 43 km (27 miles) northwest of the town of Carlin, Nevada in the Lynn Mining District, Eureka County, shown in Figure 1.

The Leeville ore deposits are located at the northern of the Carlin Trend in the Tuscarora Mountains, which are also host to Newmont's Blue Star/Geneses, Deep Post, Deep Star, Carlin East and Barrick's Betze, Rodeo, and Meikle mines. Leeville mine was developed in 2002 and put into mining operation in 2006. Personnel, equipment, and materials can be delivered from the main product shaft, ventilation shaft, and the main decline from the portal of Carlin East mine.

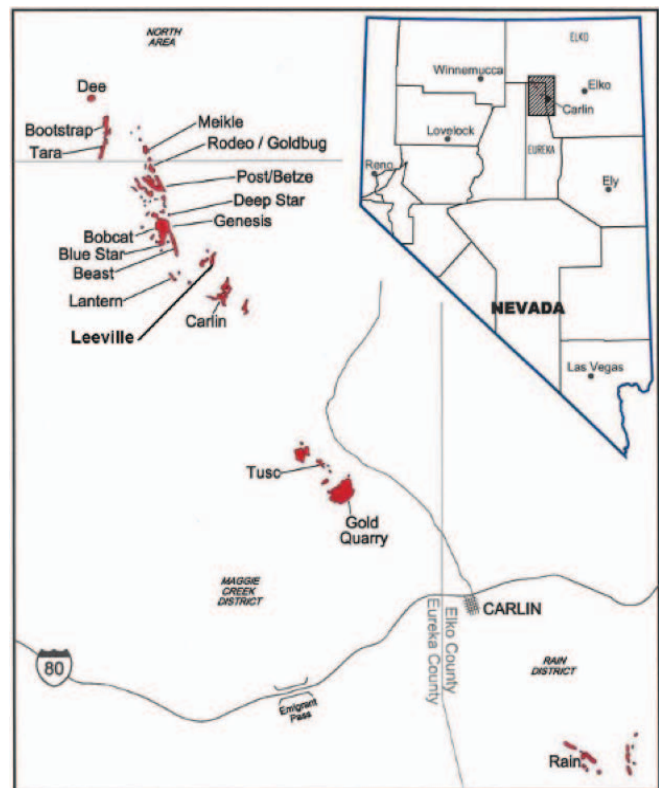


Figure 1. Location map of Leeville Underground mine.

Geology

Leeville ore deposits include three parts: West Leeville, Turf, and Four Corner, as shown in Figure 2. The West Leeville ore deposit accounts for the majority of the Leeville ore reserve and occurs at depths of 427 to 488 m (1,400 to 1,600 ft.) and is hosted by the flat lying, silty limestone of the upper Silurian-Devonian Roberts Mountain Formation. The Roberts Mountain Formation is subdivided into four lithological subunits: DSr1, DSr2, DSr3, and DSr4.

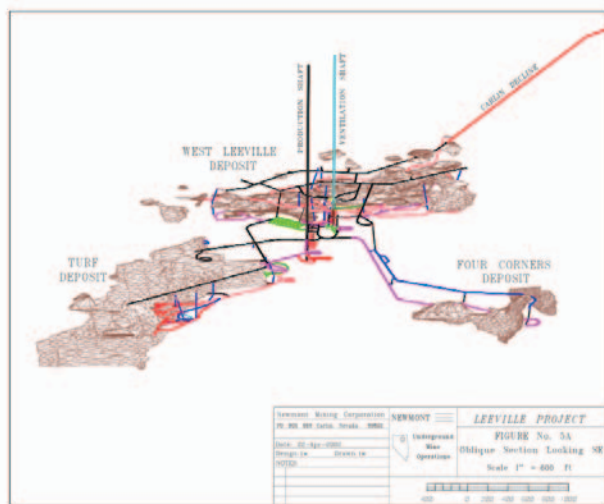


Figure 2. Deposits of Leeville Underground mine.

The West Leeville ore deposit occurs in two strata-bound zones near the transitional contacts between these units. The upper zone contains the bulk of the mineralization and occurs within the DSR2 and along the DSR2/DSR3. Lower zone mineralization occurs at the contact between the DSR3 and DSR4 (Jackson et al, 2004).

Geotechnics

Several regional and mine-area faults control the West Leeville deposit. The West Leeville deposit is a gold-hosted system extending northwest from the Carlin deposit. This large Roberts Mountains Formation-hosted gold system is known as the Leeville Corridor and occurs within a northwest-trending horst between the N30°W-striking, 60° NE-dipping Leeville fault, and the N55°W-striking, 60°W-dipping Clifford fault. The West Leeville deposit lies in the immediate footwall of the N10°E-striking, 60° NW-dipping West Bounding fault along the western margin of the Leeville Corridor. The West Bounding fault consists of at least two splays in the deposit area. Drill intercepts indicate approximately 45 m (150 ft.) of apparent normal displacement. Several west-northwest-striking faults appear to have been important controls on gold mineralization. The structurally controlled mineralization was recognized along the Packer, Mongo, and BarX faults, as illustrated in Figure 3. The three most important alteration processes were decalcification, silicification, and argillization.

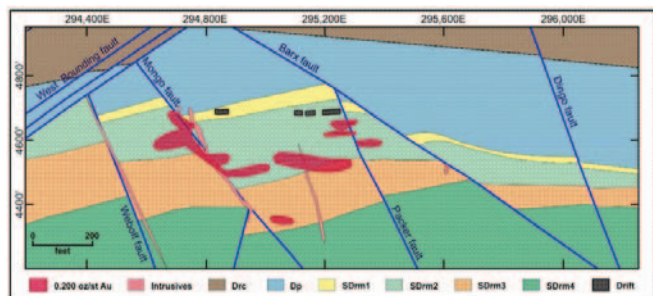


Figure 3. Major faults in the mining area of Leeville Underground mine.

High-density joints occur in the hosted rock. The geotechnical data of the West Leeville deposit are listed in Table 1. Based on the RMR rate and underground observations, the ground condition of Leeville Underground mine is poor to very poor. The typical rock core samples are shown in Figure 4. Substantial ground support is required for the mining operation at Leeville mine.

The West Leeville deposit is divided into 6 mining zones. The mine produces 3,176 t (3,500 short tons) of ore per day, mostly using long-hole stoping (95%) and other mining method with cut-and-fill (5%). Average ore grade is 10 g/t (0.32 opt). The mining area is 762 m (2,500 ft.) in length, 152 m (500 ft.) in width, and 6.1 to 61 m (20 to 200 ft.) in thickness. Access is from a production shaft and a ramp driven in the mining zones. Ore zones with a height in excess of 10.6 m (35 ft.) are considered for long-hole stoping. A typical stope measures 6.1 m (20 ft.) wide, 45 m (150 ft.) long, and 20 m (65 ft.) high, depending on the ground conditions and thickness of the ore body. This typical stope usually contains about 13,611 t (15,000 short tons) of ore. Some tertiary stope are double wide, 12 m (40 ft.).

Ore zones greater than 10 m (35 ft.) are extracted by a long-hole stope. Development of a long-hole stope requires a top cut for drilling and blasting the slot raise and long-hole rings and a bottom cut for extraction. Top cuts are typically 6.1 m (20 ft.) wide by 4.5 m (15 ft.) high with ground support. Blast rounds of 2.4 m (8 ft.) are standard but may be reduced as ground conditions dictate. Ground support in the top cuts includes 2.4 m (8 ft.) long Omega bolts (needs to be Omega-Bolt® in the back and the rib down to the sill plus wire mesh for the primary support and 3.6 m (12 ft.) Super bolts (what are super bolts?) in the back for the secondary support. Only primary support is applied for the bottom cut. The mining of a long-hole stope is shown in Figure 5. Currently, the mining operations are being carried out in Zones 1, 2, 3, 4, and 5. Three kinds of mining (mining) sequences are used at Leeville mine. The first is primary-secondary-tertiary (P (20-ft.)-S (20-ft.)-T (20-ft.)); the second is primary-secondary (P (20-ft.)-S (40-ft.)), and the third is primary-secondary (P (30-ft.)-S (30-ft.)), as illustrated in Figures 6, 7, and 8.

Backfill

Cemented rock fill (CRF) is used to fill the primary and secondary stopes for the P-S-T sequence and the primary stopes for the P-S sequence. Waste rock (gob) is employed to fill the tertiary stopes for the P-S-T sequence and the secondary stopes for the P-S sequence. The CRF system at Leeville mine was originally from Carlin East mine. The similar mining methods as at Leeville mine were used at Carlin East mine. Consequently, the original backfill strength data at Leeville mine was from Carlin East backfill system, too. The target strength of the backfill at Carlin East mine for both the long-hole stope and the cut-and-fill is 5 MPa (735 psi).

Obviously, the backfill with a strength of 5 MPa (735 psi) for the long-hole stope is much stronger than it needs to be. For the long-hole stope operations, personnel activities occur beside the backfill not underneath. Therefore, the CRF system for Leeville mine was redesigned for the strength at 2 MPa (300 psi) based on the self-stand-up stable method (Mitchell 1983). Not sure if this paragraph is needed – it seems messy and disjointed

Table 1. Geotechnical data.

Location	Variable	Min	Max	Average
All Leeville	RQD (%)	0	100	21
All Leeville	FRACTURE FREQ./ft)	0	30	10.3
All Leeville	UCS (PSI)	58	25,000	10730
All Leeville	RMR	0	74	30

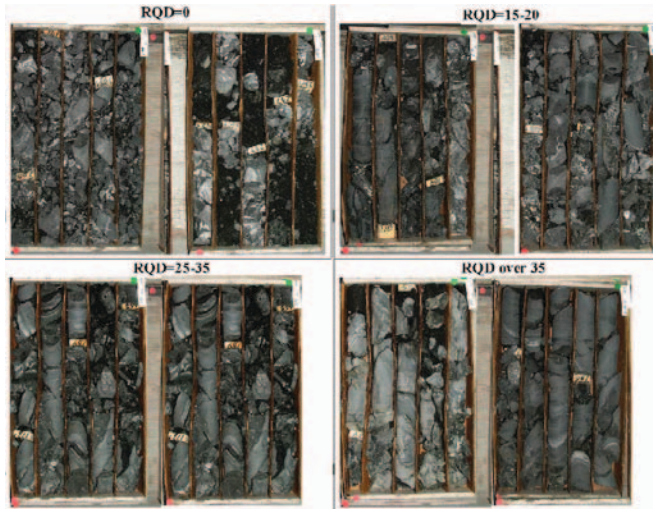


Figure 4. Typical rock core samples at Leeville Underground mine.

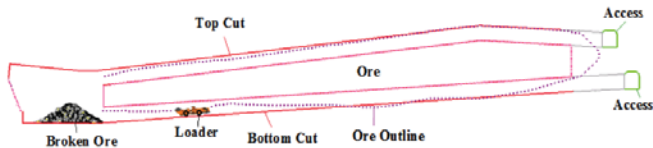


Figure 5. A typical long hole stope used at the mine.

TC	TC	TC	TC	TC	TC	TC	TC
20' Primary	20' Secondary	20' Tertiary	20' Primary	20' Secondary	20' Tertiary	20' Primary	20' Secondary
CRF	CRF	Gob	CRF	CRF	Gob	CRF	CRF
BC	BC	BC	BC	BC	BC	BC	BC

Figure 6. Mining sequence of P (20-ft.)-S (20-ft.)-T (20-ft.).

GEOTECHNICAL DATA COLLECTION

Based on the characteristics of the ore bodies, the Leeville deposits are divided into several mining zones. Since the geotechnical properties of the mining zones are different, a complete set of geotechnical data must be collected for classifying and evaluating the rock mass condition for each mining zone.

TC	TC	TC	TC	TC	TC	TC
20' Primary	40' Secondary	20' Primary	40' Secondary	20' Primary	40' Secondary	20' Primary
CRF	Gob	CRF	Gob	CRF	Gob	CRF
BC	BC	BC	BC	BC	BC	BC

Figure 7. Mining sequence of P (20-ft.)-S (40-ft.).

The methods used to collect the geotechnical data at Leeville underground mine are summarized as follows:

- Data collected by core logging. Most geotechnical data are collected by this method from the exploration drill holes. Parameters of RQD (Rock Quality Designation), FF (Fracture Frequency), UCS (Uniaxial Compressive Strength), and RMR (Rock Mass Rate) are obtained and estimated.
- Data collected by site investigation. More detailed geotechnical data and geo-structure data are collected by this method. Parameters of FF, RMR, Q (Rock Tunneling Quality Index), and geo-structure data are estimated in the underground drifts and stopes.
- Data collected by extra drill holes for the critical infrastructure facilities. To know more detailed geotechnical information in some areas where the critical infrastructures are located, this method is used.

The basic geotechnical data collected and estimated by core samples are listed in Table 2.

Table 2. Geotechnical data (average values) at Leeville mine.

Zone	RQD (%)	UCS (MPa)	FF (#/ft.)	RMR	Q
Zone 1	24.6	65.2	11.6	35.6	0.71
Zone 2	25.1	74.5	8.8	28.6	0.35
Zone 3	16.7	80.1	9.0	30.8	0.44
Zone 4	13.1	97.0	12.0	29.2	0.38
Zone 5	22.0	67.2	10.9	28.4	0.35
Zone 9	23.6	60.7	9.8	27.1	0.30
All Zones	21.0	74.0	10.3	30.0	0.41

GROUND SUPPORT SYSTEMS

Design of Ground Support Systems

Leeville ground support systems are designed based on the Tunneling Quality Index (Q) for the determination of rock mass characteristics and tunnel support requirements (Barton *et al*, 1974). In relating the value of the index Q to the stability and support requirements of underground excavations, an additional parameter is the Equivalent Dimension, D_e , of the excavation. This dimension is obtained by dividing the span, B, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, ESR.

$$D_e = \frac{B}{ESR} \quad (1)$$

The value of ESR is related to the intended use of the excavation and to the degree of security that is demanded of the support system installed to maintain the stability of the excavation. The recommended values by Barton *et al* (1974) are listed in Table 3.

Table 3. ESR determination.

Excavation Category	Description	ESR
A	Temporary mine openings.	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels, drifts and headings for large excavations.	1.3
D	Power stations, major road and railway tunnels, civil defense chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Then, a support system is determined by a chart, the equivalent dimension D_e versus the value of Q, as shown in Figure 9. Q values at Leeville mine are 0.1 to 4. The average Q value is 0.4. The spans of drifts are between 4.5 to 12 m (15 to 40 ft.). Thus, for short term excavations, D_e is 1.52 to 4.06 m, and for long term excavations, D_e is 2.86 to 7.62. The support system can be determined, as shown in Figure 9. The spacing of bolts is 1 to 1.4 m (3 to 4.6 ft.) for both long term and short term structures. The shotcrete thickness is 50mm (2 inches) for short term structures and 50 to 100 mm (2 to 4 inches) for the long term structures.

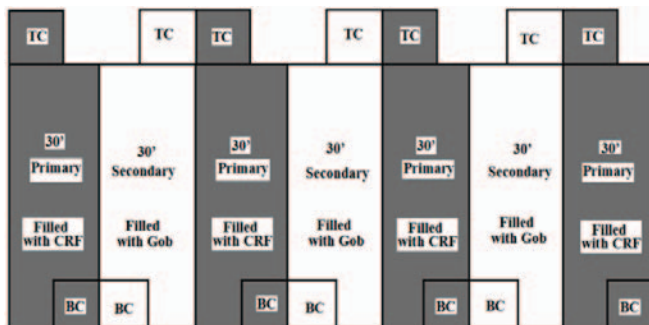


Figure 8. Mining sequence of P (30-ft.)-S (30-ft.).

Rock Bolt Selection

Several types of rock bolts are used at Leeville mine, including Split Sets, Swellex, resin bolts, cable bolts, and MAI, a self-drive grout bolt, as illustrated in Figure 10. Based on the geotechnical investigation, the rock condition at Leeville mine is poor to very poor. Although the intact rock is hard, the rock mass is extremely weak and highly fractured with an average RQD of 21. To select an appropriate support system, the concept of the interaction of ground reaction curve and support load-displacement characteristics is applied. The general concept is shown in Figure 11. Because the rock mass is very weak, a yield support system is selected at Leeville mine. The yield support system can allow the ground to deform more and decrease the support pressure requirement.

To select the appropriate rock bolt type, the anchorage characteristics of rock bolts are evaluated. Stillborg (1994) carried out a number of tests on rock bolts and dowels installed across a simulated "joint," using two blocks of high strength reinforced concrete. The results of Stillborg's pull tests are summarized in Figure 12. Based on Stillborg's test results, the Swellex and Split

Sets are good for the yield support system. However, the anchorage of the Split Sets is too low (0.5 to 1.5 T/ft.), and also it is hard to be installed for the extremely fractured rock. So the Swellex bolt is selected at Leeville mine. The Swellex bolt has a high anchorage (4 to 6 T/ft.), is easy to install in the fractured rock, and installation quality control is easy to guarantee by operators. A typical field pull test result is shown in Figure 13.

Length of Rock Bolts

Several empiric methods to determine the length of rock bolts are available in the literature and summarized as follows:

- Barton method. The length L of rockbolts can be estimated from the excavation width B and the Excavation Support Ratio ESR:

$$L = \frac{2 + 0.15B}{ESR} \quad (2)$$

- Modified Barton method. The length of rock bolt is determined by the span, B, of drift:

$$L = 2 + 0.15B \quad (3)$$

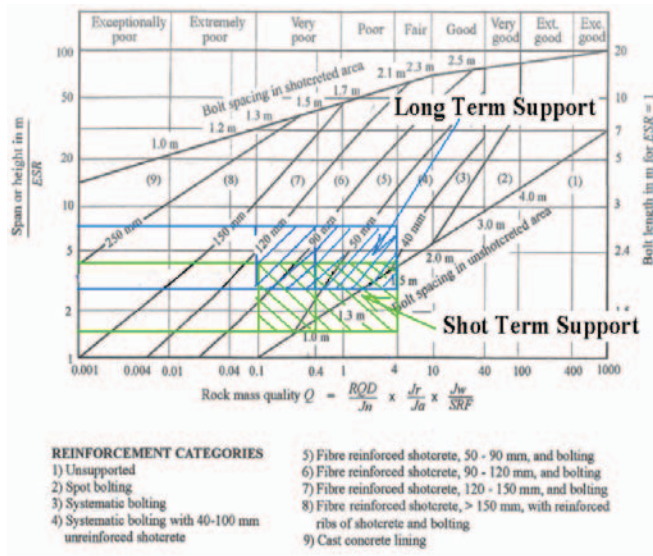


Figure 9. Estimated support categories based on the tunneling quality index Q (after Grimstad and Barton, 1993, reproduced from Palmstrom and Broch, 2006).

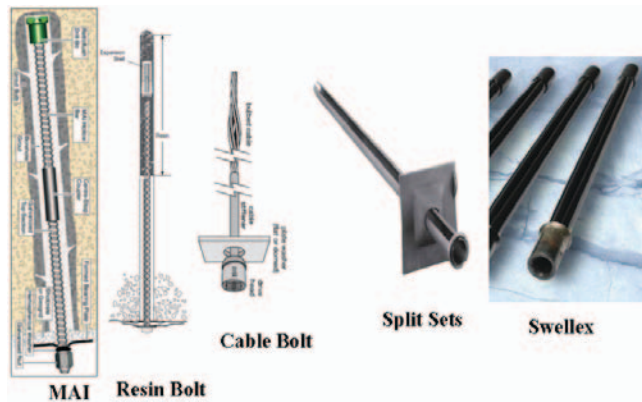
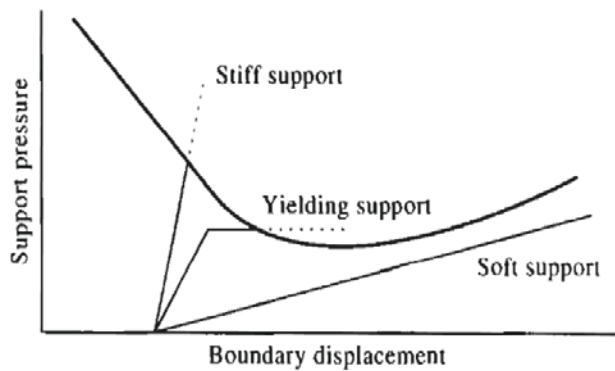


Figure 10. Rock bolts used at Leeville Underground mine.



Ground response curves and available support lines

Figure 11. Interaction of ground response curve and support lines.

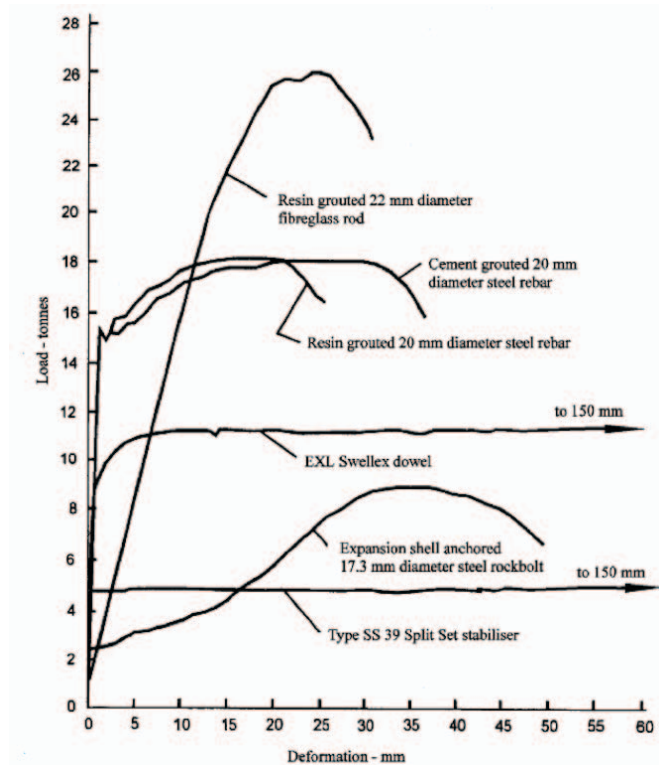


Figure 12. Pull test results for rock bolts (Stillborg, 1994).

Pull test result of standard Swellex bolt
(Anchored length: 2ft)

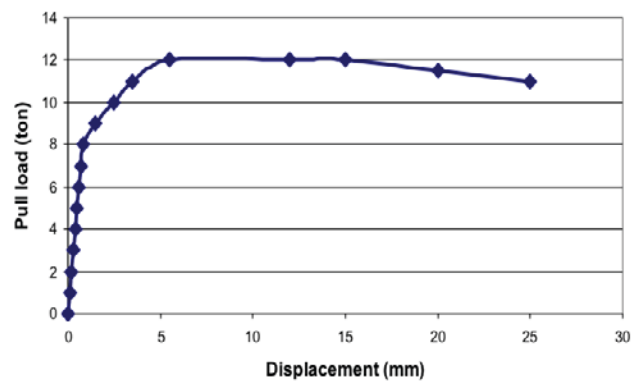


Figure 13. Field pull test for the Swellex bolt.

- Stillborg method I. Based on the rock arch theory, the length of rock bolt is determined for the moderately jointed rock by the span of drift:

$$L = 1.4 + 1.84B \quad (4)$$

- Stillborg method II. For the heavily jointed rock mass, the length of rock bolt is determined by the following:

$$L = 1.6 + \sqrt{1 + 0.0012B^2} \quad (5)$$

- Hoek and Brown et al. use an empirical rule for the length of rock bolts (Bieniawski, Z. T., 1992, and Lowrie, 2002). For

spans less than 6.1 m (20 ft.), the bolt length is one-half of the span. Spans from 6.1 m (20 ft.) to 18.3 m (60 ft.) interpolate between 3 m (10 ft.) and 4.5 m (15 ft.) lengths, respectively. For excavations higher than 18.3 m (60 ft.), sidewall bolts are one-fifth of the wall height.

- Other empirical rules for the length of rock bolts (De la Vergne, 2003; Lowrie, 2002). The length of rock bolts should be one half of the span if the span is less than 6.1 m (20 ft.). The length of the rock bolts should be one third of the span if the span is larger than 6.1 (20 ft.).
- Lang method. Lang and Bischoff (Bieniawski, Z. T., 1992), collected a set of practical data from many mines and obtained the relationship between the length of rock bolts and the span of drifts, shown in Equation (6).

$$L = B^{\frac{2}{3}} \quad (6)$$

The summaries of bolt length results based on above methods are shown in Figure 14. Those results indicate that the four methods, Hoek and Brown, Stillborg I, modified Barton, and Lang, have very good agreement. Two of the methods, Stillborg II and Barton, are too aggressive. The method, rock length of 1/2 span when span less than 6.1 m (20 ft) and 1/3 span when the span larger than 12.2 m (40 ft), is too conservative when the span is larger than 12.2 m (40 ft).

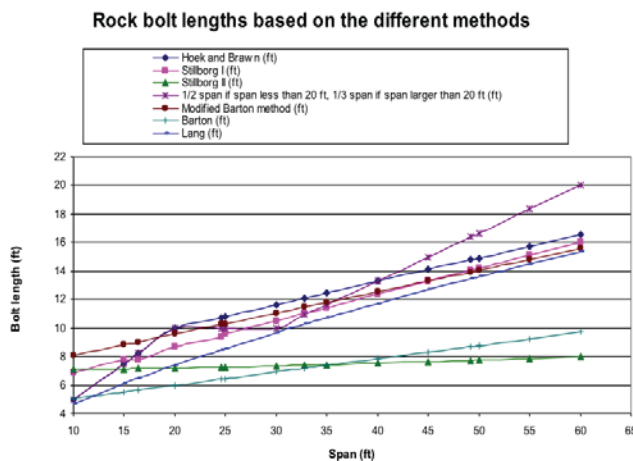


Figure 14. Rock bolt lengths determined from different methods.

These summaries – have they all been tested in the mine, or are the conclusions based from the authors experience? It just seems a general conclusion without facts backing them up?

The results of bolt lengths from the four methods, Hoek and Brown, Stillborg I, modified Barton, and Lang, are a good reference frame to determine the bolt length at Leeville mine. Based on those results, a new equation to determine the bolt length at Leeville mine is proposed as follows:

$$L = 1.5B^{0.58} \quad (7)$$

The bolt length results calculated by the proposed new method and other methods are listed in Table 4 need to modify Table 4 with (ft.) in new method and Currently used column headings and Figure 15. Bolt lengths currently being used at Leeville mine are

listed in Table 4. To guarantee that the bolt length at the mine is adequate, a numerical method is used to verify the new method. In the numerical approach, the minimum stress method is used to determine the length of the rock bolt. The approach is based on the in-situ stress and rock condition and the characteristics of the drift structures at the mine. Examine^{2D} (Rocscience, 2013) is applied to calculate the minimum stress in the back of the drift with different spans. The point where the minimum stress will be equal to zero in the back of the drift can be selected as the point of the rock failure, as shown in Figure 16. The weight of the failed rock should be supported by rock bolts. Using this approach, the safety factor can be obtained from the new method, as shown in Table .The material strength of standard bolt is 12 T, and the Super bolt is 24 T. This also was proved by ground control monitoring system at Leeville Underground mine in which a SMART Cable was installed to measure the maximum load.

Table 4. Bolt lengths determined by new method.

Span (ft.)	New Method	Currently Used at Leeville	Safety Factor	Comment
15	7.2	8	>3	Standard
20	8.5	12	>2	Standard
25	9.7	12	1.3	Standard
30	10.8	12	1.5	Super
35	11.8	12	1.4	Super
40	12.7	14	1.2	Super

Comparison of Bolt length by new method and other methods

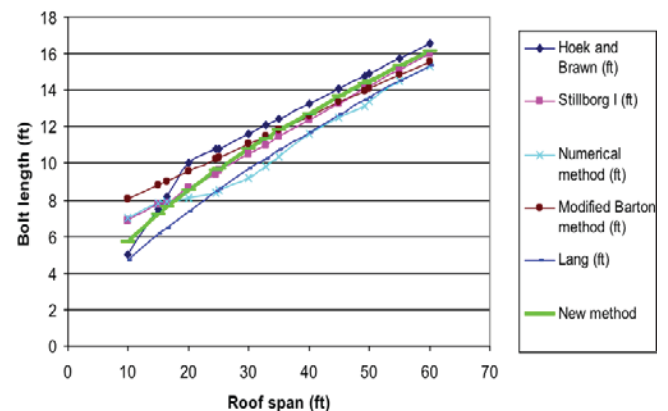


Figure 15. Comparison of bolt lengths by the new method and other methods.

QUALITY CONTROL FOR THE GROUND SUPPORT SYSTEM

The Quality Control (QC) system includes material quality control and installation quality control. The material quality control is conducted by manufacturers. An annual ASTM F432 certification (ASTM, 2004) is provided by producers for the rock bolt products. The installation quality control for the ground support system is conducted quarterly on the site. The site QCs includes the pull test for the rock bolts and Schmidt hammer test for the shotcrete.

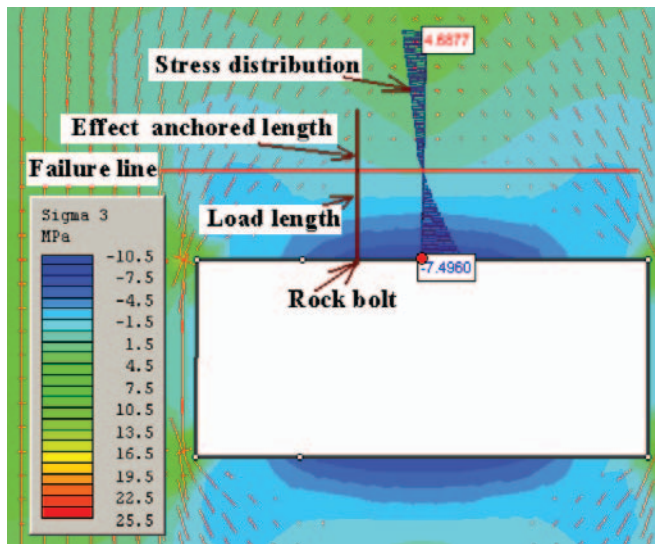


Figure 16. Using a numerical method to determine the length of rock bolts.

Two types of pull test methods are used. One is the 2-ft. long anchor test (also called the sleeve test) and the other is the non-destruction test. For the sleeve test, a 6-ft. long sleeve steel tube is used to confine the top portion of the 8-ft. long Omega bolt and only 2 ft. long bottom portion of the bolt is anchored in the hole so that the anchorage capacity per feet can be determined. For non-destruction test, when the load capacity for the bolt is satisfied (maximum 12 T), the test is passed.

The standard Omega bolt yield strength is about 22,480 lbs. The bolt yield strength is usually 80% of its ultimate strength. Therefore, the maximum load for the pull test is 12 T. The test procedure is summarized as follows:

- For the sleeve test, drill the hole with 1-1/2 inches in diameter. Insert the 8-ft. standard Omega bolt with 6-ft. steel sleeve into the hole, which is the steel tube with 25 mm internal diameter and 33 mm external diameter. In this case, only the 2-ft. bolt is anchored at the hole bottom. Pump water to 300 bars and hold in 6 second and perform the pull test.
- For non-destruction test, drill an 8-ft small hole with the diameter of 1 1/2 inches, and then install the bolt into the hole. Pump water to 300 bars and hold in 6 second and perform the pull test.

For the pull test with the sleeve, the anchorage of the bolt should be 4 to 6 T per ft. For the non-destruction pull test, the anchorage capacity of the bolt should be the maximum load, 12 T without slip.

Two shotcrete samples are taken per day, and two backfill samples are taken per shift. Twenty eight days of compressive strength tests are conducted for samples of the shotcrete and backfill to determine if they met the design requirements.

APPLICATION AND RESULTS

Leeville underground mine started developing in 2002 and operating in 2006. The ground support system has been applied for almost ten years, and the backfill system has been used for

seven years. So far, the most of stopes in Zone 1 and Zone 4 are gone. The good results have been achieved by ground support and backfill systems. The backfilled ribs can hold stable for whilst the secondary and tertiary stopes are mined out. Even though some of the secondary and tertiary stopes are double wide (40 ft) and double high (over 100 ft) and their ribs are still kept at the remain in a stable condition.

The large span structures, such as shop, passing bay, ore break chamber, etc., are shown in Figure 17. These large span structures, which were designed using the this ground support system, have kept in good shape for many years in such a poor ground condition.



Figure 17. 35-ft wide Passing Bay.

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Assessing Coal Bumps From Excess Energy in Finite Difference Models

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ABSTRACT

A numerical investigation is made into unstable failures of coal pillars (i.e. coal bumps) using the finite difference software FLAC3D. A static energy balance is first derived to calculate the excess energy released as a consequence of unstable failure in rock. Two- and three-dimensional mining layouts are then assessed on their bump-potential based on the magnitudes of excess energy released during simulated pillar failures. A three-dimensional pillar model is developed to represent tributary area loading of an infinite room-and-pillar layout. Pillar strength and brittleness characteristics are calibrated using a Mohr-Coulomb strain-softening constitutive law to simulate width-to-height ratio coal pillars ranging from 1:1 to 5:1. Width-to-height ratio pillars 1:1 to 3:1 release large magnitudes of excess energy as the pillars fail, representing massive collapse of room-and-pillar layouts. The larger pillars maintain their residual strength during failure; however significant excess energy is still released during temporary reductions in pillar strength. A two-dimensional model is then constructed of a gateroad pillar failing due to elastic rebound of the surrounding rockmass. Soft loading conditions initiate unstable failure for all pillar widths, while the total magnitudes of excess energy increase with the size of pillar being modeled. The results from these tests indicate that excess energy may be used as a direct assessment of the bump potential of simulated mine layouts.

INTRODUCTION

Coal bumps and rockbursts are low frequency, high impact unstable failures that pose a serious threat to miners and to the integrity of mine workings. Current numerical modeling software offers capabilities that can be utilized to detect and quantify coal bumps in coal mine layouts. This paper presents a methodology that was developed for assessing the magnitude of unstable failures in simulated FLAC3D mine models.

EXCESS ENERGY BALANCE

During unstable failure, more potential energy is released from a rock system than is stored as strain energy and is consumed through plastic deformation. The resulting excess energy is not accounted for statically and is instead converted into kinetic energy through the dynamic motion of rock within the system. The total

kinetic energy released by an unstable failure may thereby be determined from the excess energy of the otherwise static system. This concept is demonstrated for a simple uniaxial compressive strength (UCS) test shown in Figure 1 where a stiff loading system elicits a stable failure while a soft loading system provides excess energy that is then converted into kinetic energy as the brittle specimen is caused to fail in an unstable manner.

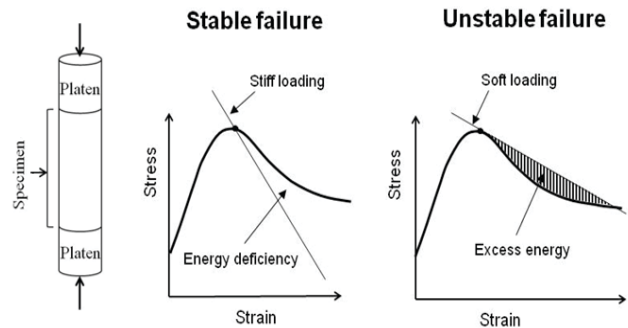


Figure 1. Idealized representation of unstable failure and excess energy in UCS test case.

This concept of excess energy was extended to the more complex loading conditions found in underground mines by calculating the static contributions of energy for a continuum numerical method. The calculation of excess energy, E_e , is shown below where W_b is the total boundary work applied to the system, W_g is the contribution of gravitational potential energy, U is the elastic strain energy stored in the rock, and W_r is the work performed through plastic deformations of the rock. The energy balance is based on Salamon's previous work (1984) and was implemented in FLAC3D v4.01 to determine the total excess energy released during simulated unstable failures.

$$E_e = (W_b + W_g) - (U + W_r)$$

A routine was written using the FISH programming language within FLAC3D to calculate boundary work and changes in

gravitational potential energy. The built-in energy tracking module in FLAC3D was then used to calculate the elastic strain energy and the work performed by plastic deformations in the model (Itasca, 2012).

Figure 2 shows the stress-strain results of stable and unstable failures of FLAC3D simulated laboratory triaxial compressive strength tests. Each confinement test was repeated using both stiff and soft loading platens to produce stable and unstable failures, respectively. When the failure is unstable, the specimen stress response follows the elastic rebound of the testing machine as represented by the dashed lines in the plot. If the failure is stable, the specimen compression follows the characteristic post-peak behavior of the specimen. The area between the dotted and solid line in the post-peak regime is proportional to the excess energy released in the case of unstable failures.

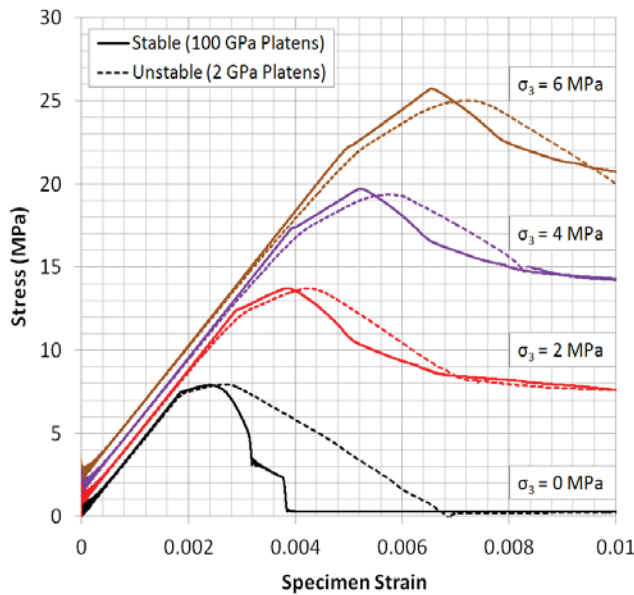


Figure 2. Unstable and stable failures of triaxial compressive strength tests at different confinements.

To demonstrate the proposed approach for determining the excess energy in Figure 2, a set of uniaxial compressive strength tests were run in FLAC3D using the material properties listed in Table 1. First, a pair of stiff 100 GPa platens was used to fail a brittle rock specimen in a stable manner. The results of the energy balance in this case are shown in Figure 3 where the calculated excess energy remained low throughout the test. Next, a pair of soft 2 GPa platens was used to cause unstable failure of the specimen, and the resulting energy histories are shown in Figure 4. In this unstable case, the excess energy increased during the specimen failure and matched expected values of energy released from the unstable system.

THREE-DIMENSIONAL TRIBUTARY AREA MODELS

As shown in the previous section, available constitutive models simulate the stress-strain response of brittle rock while approximating the elastic and plastic energy stored or consumed during failure. Mine models may be constructed using these techniques to allow for unstable failure conditions to arise within

Table 1. FLAC3D inputs for uniaxial and triaxial compressive strength test specimen.

Specimen Geometry	Cylinder, 2m (H) x 1m (D)
Mesh Type/Resolution	Radial, 0.1m (vertical), 0.1m (radial)
Elastic Properties	4 GPa (E); 0.2 (ν)
Cohesion vs. Plastic Millistrain	2.41 MPa, 0.5; 0.5 MPa, 9; 0.25 MPa, 15.5
Friction Angle vs. Plastic Millistrain	0°, 0; 15°, 1; 15°, 7; 5°, 12
Dilation Angle vs. Plastic Millistrain	0°, 0; 15°, 1; 15°, 7; 5°, 12

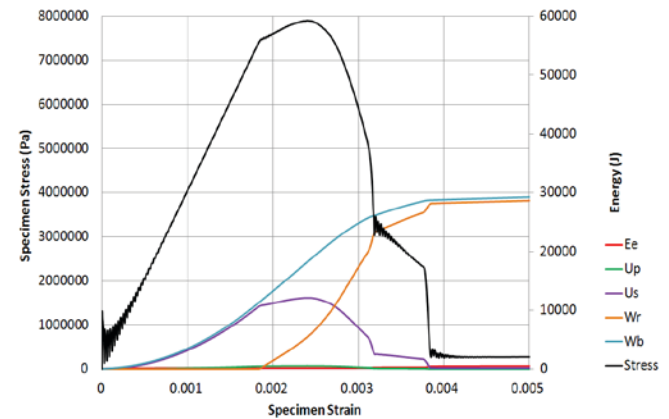


Figure 3. Energy histories of stable UCS test with 100 GPa platens.

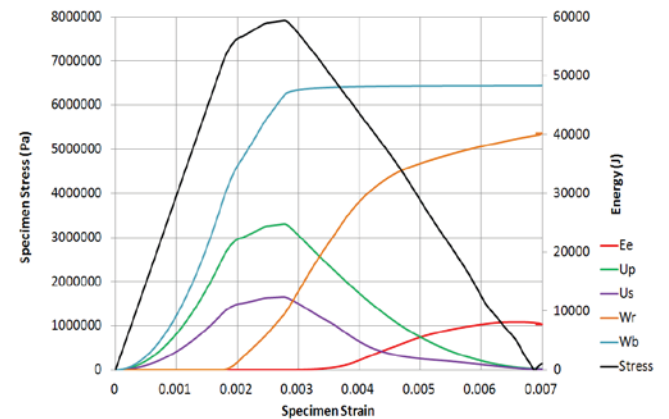


Figure 4. Energy histories of unstable UCS test with 2 GPa platens.

simulated volumes of brittle rock. The explicit representation of failure given by FLAC3D allows for static energy calculations to be performed even while the system is unstable. The total excess energy may therefore be determined during simulated rock failure to provide an assessment of the bump potential of a given mine layout.

The calculation of excess energy was implemented within a quarter-symmetry model of a pillar which represents an infinite room-and-pillar layout. Pillar geometries with width-to-height

ratios from 1:1 to 5:1 were tested under these conditions to determine the excess energy released in each case. A coal seam height of 2.4 m and an entry width of 6 m was assumed for the tests. The resulting model geometry is shown in Figure 5.

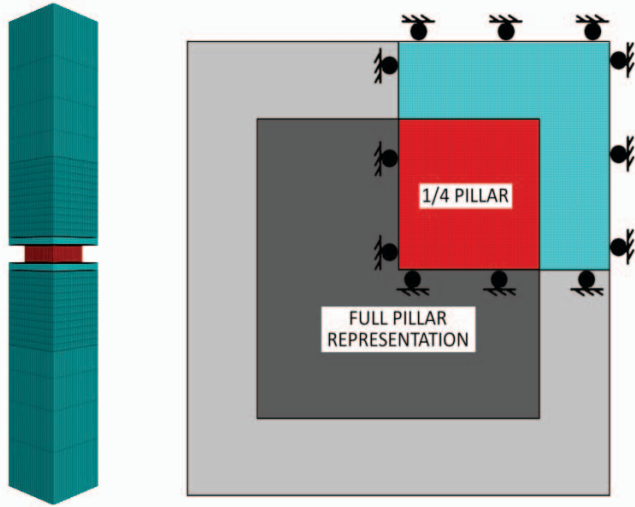


Figure 5. Quarter-symmetry pillar model to represent an infinite room-and-pillar mining layout.

A calibration procedure was applied to the simulated coal pillars to determine appropriate inputs for cohesion, friction angle, and dilation angle in the Mohr-Coulomb strain-softening model (MCSS) found in Table 2. The ultimate strengths of the pillars were compared against the Bieniawski (1968) and Salamon-Munro (1967) pillar strength formulas for a 6.2 MPa cubic strength of coal as shown in Figure 6. The post-failure modulus of the 2:1 and smaller pillars was also assessed against Wagner's classic in situ pillar tests (Wagner, 1974). The calibrated pillar responses shown in Figure 7 were found to be within acceptable ranges for all pillar widths that were tested.

Table 2. FLAC3D inputs for coal pillar models.

Mesh Type/Resolution	Cubic zones, 0.1m ³
Elastic Properties	2.7 GPa (E); 0.12 (ν)
Cohesion vs. Plastic Millistrain	1.45 MPa, 0; 1.45 MPa, 0.5; 0.2 MPa, 21.3
Friction Angle vs. Plastic Millistrain	23°, 0; 30°, 0.5; 30°, 1
Dilation Angle vs. Plastic Millistrain	2°, 0; 10°, 0.5; 10°, 21.3

Gravity was applied to the system and a pressure boundary was established at the top of the model to represent a tributary area load on the pillar. The pressure boundary was increased at a rate of 1 kPa per 100 steps until the pillar was caused to fail. Excess energy was calculated during these tests to assess the magnitudes of energy released due to unstable collapse of the room-and-pillar layouts.

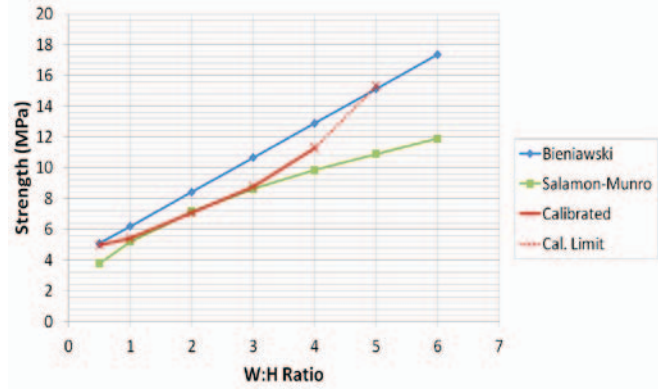


Figure 6. Ultimate pillar strengths of calibrated coal pillars compared against classic coal pillar strength formulas.

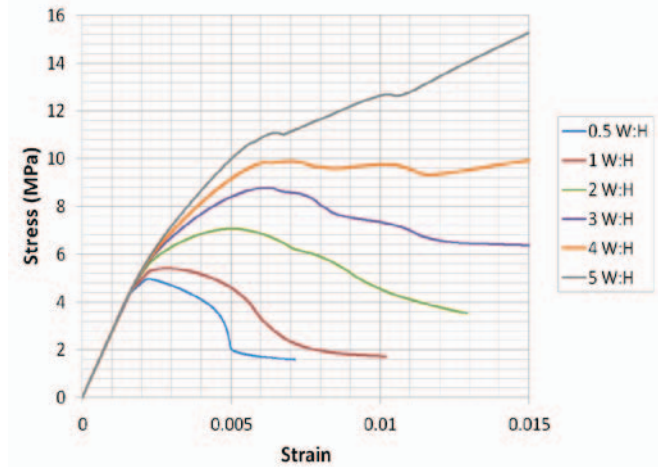


Figure 7. Stress vs. strain histories of the calibrated FLAC3D pillar models.

THREE-DIMENSIONAL MODEL RESULTS

The role of gravitational potential energy was studied in the expression of unstable failure in the quarter-symmetry pillar models. The 1:1 to 3:1 pillars failed in a completely unstable manner that resulted in the unchecked downward displacement of the overburden. An example of the excess energy history is provided for the smallest 1:1 pillar in Figure 8.

The 4:1 and 5:1 pillars were wide enough to maintain sufficient residual strength during failure to resist total collapse of the roof, however the formation of large shear failures in the wide pillars still resulted in temporary reductions in strength and subsequent losses of stability. This concept is demonstrated for the 4:1 pillar in Figure 9 in which a small decrease in strength led to a significant increase in excess energy.

Figure 10 shows the excess energy histories for the different pillar geometries normalized over the total development area. Larger magnitudes of normalized excess energy were recorded for the 1:1 through 3:1 pillars. The 4:1 and 5:1 pillars also showed some increase in excess energy as a consequence of large shear failures within the pillars.

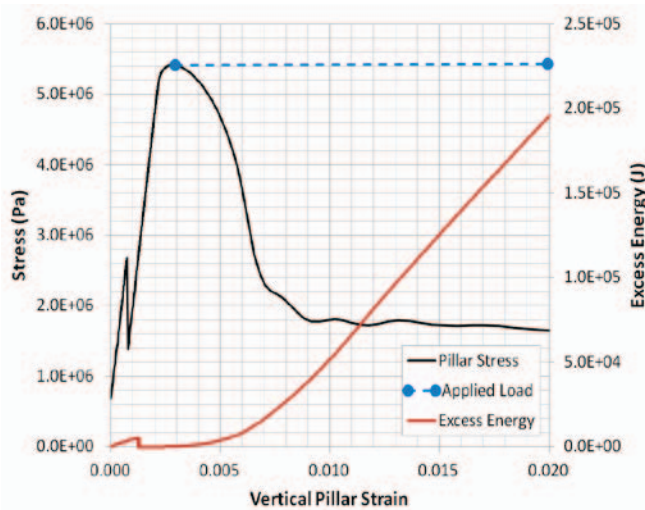


Figure 8. Excess energy during unstable collapse of 1:1 pillar.

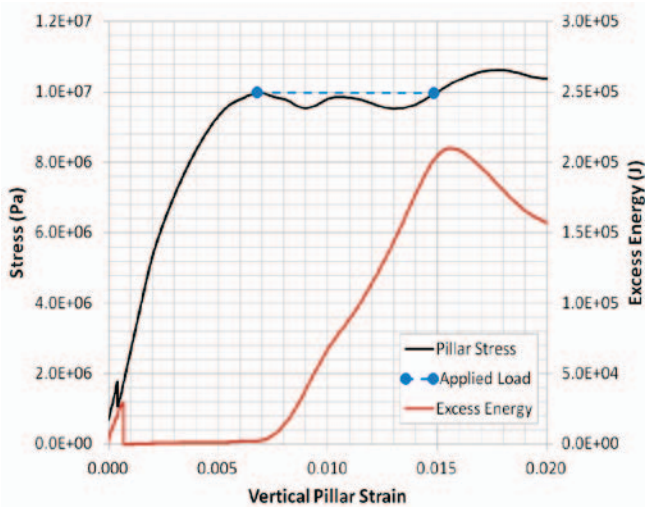


Figure 9. Excess energy during failure of 4:1 pillar.

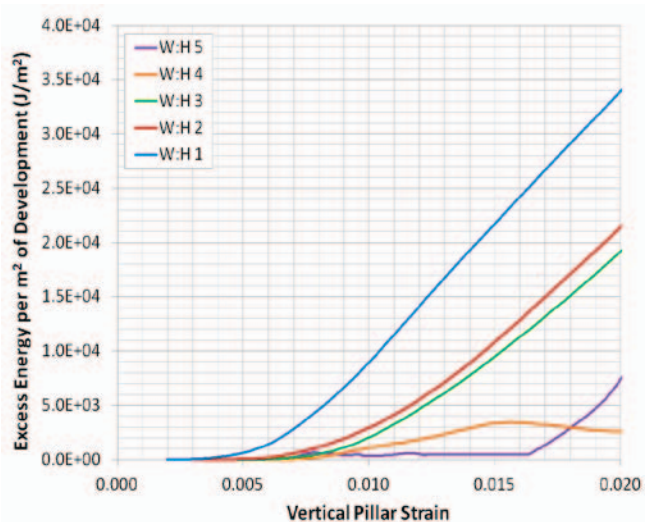


Figure 10. Excess energy released per square meter of development for different pillar geometries.

The calculation of excess energy was shown to be a direct assessment of the stability of different room-and-pillar mine layouts. The results of the tests revealed that smaller pillars of widths 1:1 to 3:1 did not provide sufficient residual strength to resist downward displacement of the roof, and a massive collapse was observed in these cases. The larger pillars led to more stable layouts; however, excess energy was still released due to temporary reductions in residual strength.

TWO-DIMENSIONAL GATEROAD PILLAR MODELS

To study the excess energy levels in the case of a gateroad pillars, a two-entry gateroad layout was assessed for pillar widths ranging from 1:1 to 5:1. The half-symmetry plane strain model shown in Figure 11 was constructed in FLAC3D to represent an elastic rockmass loading a single gateroad pillar with two 6-m entries to either side. The material properties of the pillar matched those found in the three-dimensional model listed in Table 2. Loads were increased in the system through a slow, constant velocity boundary at the top and bottom of the model. No gravitational loading was assumed for these simplified tests.

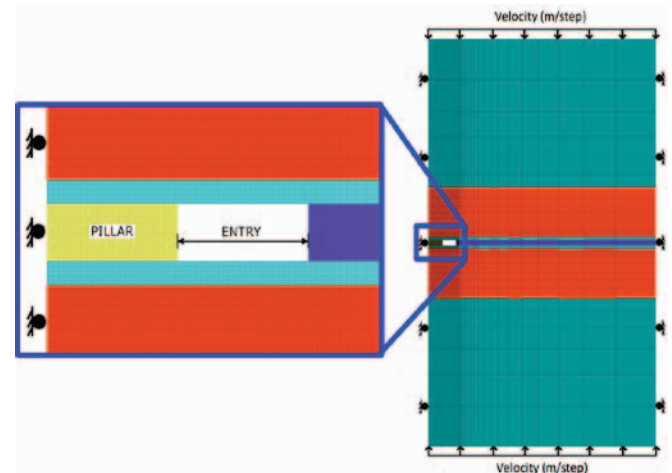


Figure 11. Two-dimensional plane strain model of gateroad pillar.

The elastic modulus of the rockmass was adjusted between tests to allow more or less strain energy to be stored in the loading system prior to the failure of the pillar. Young's moduli of 2, 5, and 20 GPa were assigned to induce unstable (2 GPa), quasi-stable (5 GPa), and stable (20 GPa) pillar failures. The stress-strain results of the 1:1 pillar are shown in Figure 12 where the 2 GPa rockmass led to an unstable failure and a deviation of the post-peak pillar response from expected values.

The results of the excess energy calculations are shown in Figure 13 for all test cases. Excess energy magnitudes increased with increasing widths in the unstable 2 GPa rockmass. These results revealed that more energy was released due to unstable failure conditions within the stronger, wide pillar cases.

The failure mode of the pillar played an important role in the stability of the simulated gateroads. Stress and strain measurements could be used for the smaller 1:1 and 2:1 pillars to directly identify the presence of unstable failure conditions however this approach became unsuitable for wider pillars which

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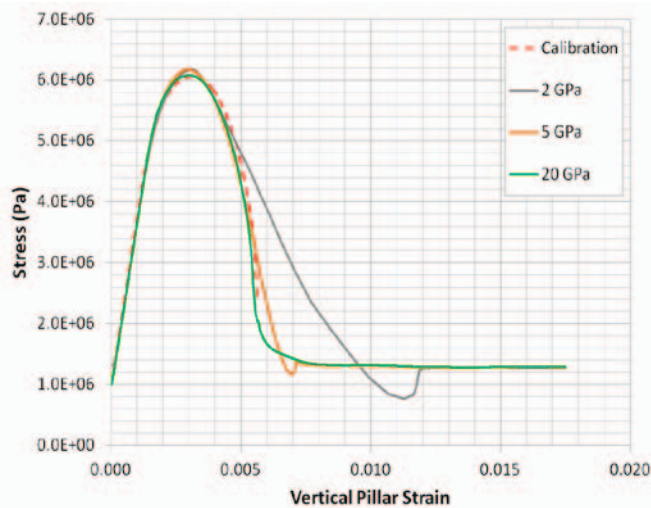


Figure 12. Failure of 1:1 pillar by unstable 2 GPa, quasi-stable 5 GPa, and stable 20 GPa rockmass conditions.

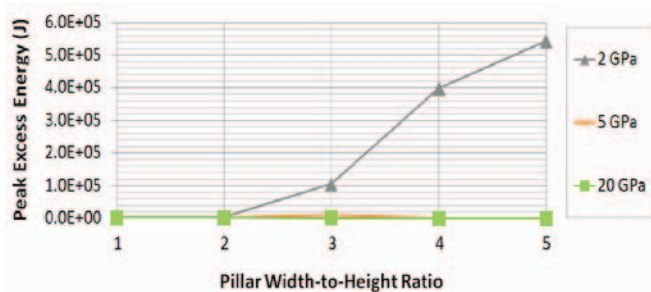


Figure 13. Excess energy released in gateroad models for increasing width-to-height ratio pillars.

experienced complex, multi-stage failure modes. The calculation of excess energy provided the only direct assessment of unstable failure and bump-potential for the wider pillar geometries of 3:1 and greater. A more thorough discussion of the changing failure mode of pillars at different width-to-height ratios is provided in Garvey's analysis (2013).

CONCLUSIONS

The excess energy calculation provided a rational method for analyzing the bump potential of room-and-pillar and gateroad layouts. During tributary area loading of the three-dimensional models, the smallest pillar geometries released the highest magnitudes of excess energy as is consistent with the scenario of a massive collapse of undersized pillars. In the two-dimensional gateroad, the excess energy increased with increasing pillar widths for the unstable, soft rockmass conditions.

Additional testing of the energy balance is required to calibrate the plastic work consumed during unstable pillar failure to match the approximate values of in situ pillar failures. Future backanalysis studies should also be conducted to validate the magnitudes of excess energy in the FLAC3D models against measures of the total kinetic energy released in historical cases of coal bumps.

Three-Dimensional Ground Stability Study Around a Mine Tunnel in a Highly Discontinuous Rock Mass under Significant in Situ Stress

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ABSTRACT

To exploit an underground mine effectively and safely, it is important to have a good understanding of the geotechnical behavior of the rockmass surrounding the underground mine excavation. The aim of this study was to investigate, at the three-dimensional level, the geotechnical behavior around a tunnel excavated in a metal mine in China. A three-dimensional numerical model was built by using the 3DEC software package to simulate a tunnel excavation under a high in situ stress condition and by relying on the information available on lithology, geological structures, in situ stress measurements, physical and mechanical properties of intact rock, discontinuities, and interfaces between different rocks. The authors also investigated the effect of the discontinuity network, possible intact rock and discontinuity parameter variability, representation of rock masses as discontinuum or equivalent continuum material and rock support system on the deformation, and stability around the tunnel. A very good agreement was obtained between the collected field deformation monitoring results and the results of the conducted numerical stress analyses with appropriate geomechanical property values.

Keywords: rock tunnel; geotechnical behavior; discontinuities; three-dimensional stress analysis; discontinuum modeling; equivalent continuum modeling

INTRODUCTION

The Chengchao Iron Mine is a large underground metal mine located in China. The ore-body runs several kilometers approximately in the N 65° W – S 65° E direction with plus or minus 15° variation. The width of the ore-body in the horizontal direction is in the hundreds of meters and thickness of the ore-body in the vertical direction is in the tens of meters. The ore-body exists at a number of different levels. The mining method used is sublevel caving without sill pillars. To extract and transport the ore, for economic reasons, roadway tunnels are excavated approximately in the length direction of the ore and perpendicular to that. The maximum horizontal principal stress direction is approximately N 75° W. This means some of the roadway tunnels in the mine need to be excavated approximately perpendicular to the maximum horizontal principal stress direction. With respect to the effect of

in situ stress on stability, this is the worst direction to excavate a tunnel. Even though this direction is avoided in civil engineering applications, in mining engineering, extraction of ore sometimes requires excavation of tunnels in this direction. Therefore, for the considered mine, it is important to study stability of tunnels excavated in the direction perpendicular to the maximum horizontal principal stress direction. This paper deals with such a study and is a summarized version of a journal article by Wang et al. (2012).

The rock mass consists of four different rock types: Magnetite, Granite, Diorite, and Marble. The main factors that influence the stability, safety, and efficient exploitation in Chengchao Iron Mine are as follows:

- Complex ore body structure in magnetite rock approximately along the maximum horizontal principal stress direction
- Existence of 2–3 fracture sets in each rock type and wide spread direction of these fracture sets
- High average fracture density (between 4 and 22 fractures/m)
- Variability and uncertainty associated with estimation of geomechanical parameters of intact rock and discontinuities.

Excavation of an underground tunnel in a mine leads to deformation and stress redistribution. A good understanding of the geotechnical behavior through a stress analysis around a tunnel helps one to design a proper support system and ensure that exploitation of the mine is effective and safe. In underground excavations, discontinuities play a significant role on the deformation and stability of the rock mass around a tunnel or cavern. Out of the methods available, discontinuum stress analysis methods have the best capability of capturing the directional effects arising on deformability and stability from various orientations of discontinuity sets. In addition, they have the capability of simulating large displacements and rotations that can occur with discontinuous rock masses. Also, they can be used to perform equivalent continuum stress analysis. For a literature review on application of numerical methods to investigate tunnel stability in jointed rock masses, refer to Wang et al. (2012), which shares the topic dealt in this paper.

Due to the above reasons, a three-dimensional numerical model is built by using the three dimensional Distinct Element Code (3DEC) software package (Itasca, 2007) to investigate the

geotechnical behavior around a tunnel excavated perpendicular to the maximum horizontal principal stress direction under a high in situ stress condition. The numerical model was built using information available on lithology, geological structures, in situ stress measurements, physical and mechanical properties of intact rock, discontinuities, and interfaces between different rocks. To study the effect of the discontinuity network on deformability and stability around the tunnel, stress analyses were conducted representing the rock masses that exist around the tunnel as (a) discontinuum material, as well as (b) equivalent continuum material. Studies were also conducted with and without rock support to evaluate the effectiveness of a rock support system on the deformation and stability around the tunnel. To examine the accuracy of the numerical results, the numerical analyses results are compared with the deformation monitoring data obtained from the field.

DEVELOPED NUMERICAL MODEL AND CONDUCTED ANALYSES

Setting up of the numerical model

The length $\times$ width $\times$ height chosen for the numerical model is 40 m $\times$ 20 m $\times$ 40 m. The center of the tunnel is located at a depth of 480 m. Figure 1 shows the simplified stratification of the numerical model with different lithologies. It is constructed based on the geologic profiles obtained in the field. The point of $x=0$, $y=0$ $z=0$ is at the center of the cube shown in Figure 1. The physical and mechanical properties selected to represent different lithologies and to perform stress-deformation analysis are shown in Table 1. 3DEC has several built-in material behavior models. For this study, the Mohr–Coulomb model was used.

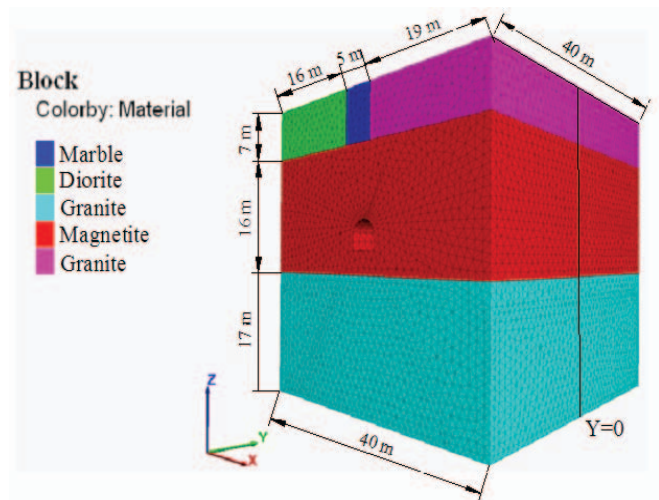


Figure 1. The built numerical model with different lithologies.

The values selected to represent discontinuity mechanical properties in different rock types in the numerical model are shown in Table 3. Discontinuities are considered to be open and smooth with no filling material. Accordingly, the cohesion and tensile strength are set to zero for the discontinuities. In the numerical stress analyses performed in this study, discontinuity interfaces are included between every two adjacent materials. Table 4 shows the mechanical property values used to represent the discontinuity interfaces between different types of rocks. Note that these interfaces are not weakness planes; they reflect the material properties to capture the transition between the adjacent materials.

Table 1. Physical and mechanical properties of different lithologies.

Lithology	Density (kg/m ³)	Bulk modulus, K (GPa)	Shear modulus, G (GPa)	Cohesion, C (MPa)	Internal friction angle, Φ (phi) (deg.)	Tensile strength (MPa)
Diorite	2630	26.49	19.05	3.75	25.9	2.25
Marble	2660	35.42	20.24	3.50	26.1	2.00
Granite	2780	36.07	24.84	3.63	27.3	1.25
Magnetite	4120	44.70	23.05	3.85	27.5	2.63

The joint systems that exist in different lithologies are very complex, with short spacing, high density, and multiple directions (see Table 2). When the number of joints is very high, it takes a very long time to get a calculation completed using 3DEC. Kulatilake et al. (1993) has shown that the first invariant of fracture tensor is a combined measure of fracture size and intensity. Therefore, an equivalent fracture system can be obtained by keeping the same orientations and increasing the fracture size and decreasing the intensity to keep the first invariant of fracture tensor the same. Note that decrease of fracture intensity is equivalent to increase of fracture spacing. Therefore, in this study for each fracture set, the fracture size was increased to make them persistent in the studied domain and accordingly, fracture spacing was increased to keep the first invariant of fracture tensor the same. The obtained modified mean joint area and mean spacing are also given in Table 2.

In situ stress measurements were not available exactly at a depth of 460 m, which is the top boundary chosen for the geomechanical model shown in Figure 1. However, three in situ stress measurements were available in the depth range of 335 m to 495 m (Table 5). These measurements were obtained by applying the overcoring technique using KX-81 triaxial hollow inclusion stress gauge. The in situ stress results given for depths 425 m and 495 m were used with linear interpolation to estimate the in situ stresses that should be applied at a depth of 460 m. Note that in the 3DEC analyses, the maximum and minimum in situ stress directions are set up in X and Y directions, respectively. The vertical stress applied at the top boundary (at a depth of 460 m) of the model is 13.96 MPa (zz-stress in 3DEC). Based on the estimation of $\sigma_1 = 1.60 \gamma H$, a maximum horizontal principal stress of 22.34 MPa (xx-stress in 3DEC) was applied in the horizontal direction on the tunnel cross sectional plane at a depth of 460 m. Similarly, based

Table 2. Fracture geometry properties for each lithology.

Rock types	Groups	Dip direction (deg.)	Dip (deg.)	Original mean spacing (m)	Modified mean spacing (m)	Original mean trace length (m)	Modified mean joint area (m ²)
Diorite	A group	290	70	0.333	1.29	5.0	294
	B group	30	50	0.25	1.05	3.0	126
	C group	65	60	0.0625	0.63	1.6	204
Marble	A group	220	55	0.1	0.38	2.0	46
	B group	160	73	0.1666	0.74	1.7	45
Granite close to top boundary	A group	60	68	0.045	0.31	2.3	201
	B group	40	76	0.0769	0.37	3.0	150
Granite close to bottom boundary	A group	60	68	0.045	0.66	2.3	902
	B group	40	76	0.0769	0.82	3.0	750
Magnetite	A group	45	50	0.33	4.38	2.8	1085
	B group	130	72	0.125	1.32	3.0	787
	C group	290	34	0.25	4.62	1.5	602

Table 3. Mechanical property values of discontinuities of different rock types.

Lithology	Friction angle (phi) (deg.)	Cohesion, C (MPa)	Normal stiffness, JKN (GPa/m)	Shear stiffness, JKS (GPa/m)
Diorite	27	0	6.0	4.3
Marble	26	0	6.1	4.7
Granite	26	0	6.5	5.0
Magnetite	27	0	9.0	7.0

Table 4. Mechanical property values of discontinuity interfaces included in the numerical model.

Interface type	Friction angle (phi) (deg.)	Cohesion, C (MPa)	Tensile strength/Shear bond strength (MPa)	Normal stiffness, JKN (GPa/m)	Shear stiffness, JKS (GPa/m)
Diorite - Marble	26	0	0	70	38
Marble – Granite close to top boundary	26	0	0	70	38
Diorite, marble, granite close to top boundary –magnetite	27	0	0	80	46
Magnetite – Granite close to bottom boundary	26	0	0	50	30

on $\sigma_3 = 0.39 \gamma H$, a minimum horizontal principal stress of 5.44 MPa (yy-stress in 3DEC) was applied in the horizontal direction along the tunnel axis at a depth of 460 m. The velocity was set to zero in X and Y directions on both side boundaries of the whole model (equivalent to no horizontal movements in X and Y directions). On the bottom boundary, the velocity was set to zero in the vertical direction (equivalent to no vertical movement). In situ zz-stress, xx-stress, and yy-stress were increased from the top to the bottom of the model according to the gravitational loading and lateral earth pressure coefficients associated with the different strata in the model.

In the Chengchao Iron Mine, the fully grouted, 18-mm diameter bolts were used to support the tunnel shown in Figure 2. The bolt

lengths were 2.0 m in the roof and 1.8 m in the two walls of the tunnel. The spacing used perpendicular to the tunnel cross section as well as along the tunnel cross section is 1.0 m×1.0 m. Table 6 provides the parameter values used to represent the bolt structure in the numerical model.

Conducted numerical stress analyses

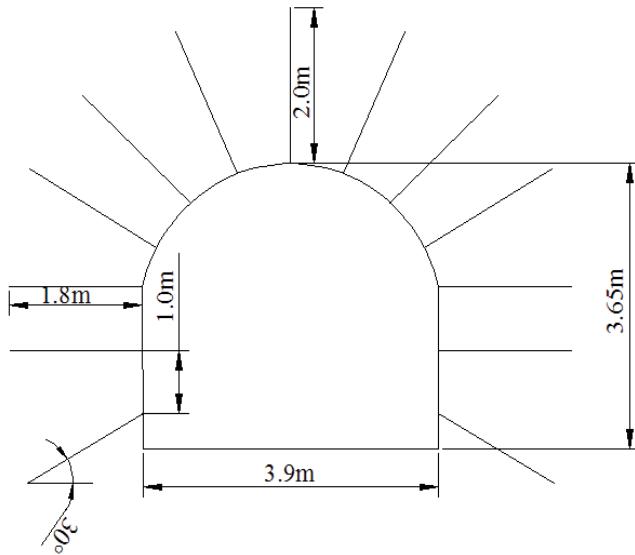
Table 7 shows the different numerical stress analyses performed for the study reported in this paper. Column 2 states whether only the intact rock was considered in the numerical model or if both the intact rock and joints were included in the numerical model. Column 4 states whether the combined effect of intact rock and joints is represented as equivalent continuum rock mass properties

Table 5. Available in-situ stress measurement results.

Site No.	Depth (m)	Vertical stress (Intermediate principal stress)	Maximum horizontal principal stress	Minimum horizontal principal stress
1	335	$\sigma_2 = \gamma H$	$\sigma_1 = 2.75 \gamma H$	$\sigma_3 = 0.46 \gamma H$
2	425	$\sigma_2 = \gamma H$	$\sigma_1 = 1.938 \gamma H$	$\sigma_3 = 0.336 \gamma H$
3	495	$\sigma_2 = \gamma H$	$\sigma_1 = 1.274 \gamma H$	$\sigma_3 = 0.443 \gamma H$

Table 6. Parameter values used to represent the bolt structure in the numerical model.

Parameter	Value
Young' modulus (GPa)	210
Cross-sectional area (m ²)	2.55×10^{-4}
Tensile yield capacity (MN)	240
Grout cohesive capacity per unit length (MN/m)	100
Grout stiffness per unit length (MN/m ²)	2×10^3


Figure 2. Diagram of the tunnel shape with bolt support on the roof and walls.

or not. If the numerical model has included only the intact rock or discontinuities are included explicitly (separately) (see column 2), column 4 is filled with "NO."

RESULTS FROM NUMERICAL STRESS ANALYSES

Effect of discontinuities on deformations around the tunnel

For Case 1, the in situ stresses can be calculated analytically. Therefore, this case is used to check whether the inputs are applied correctly to 3DEC to simulate the tunnel excavation. The obtained maximum displacements are given in Table 8. The computational time increases with the number of discontinuities present in the model. To reduce the computational time, the length of the model in the y direction was limited to 10 m. Out of the 12 joint sets, 11 of them have dip angles greater than or equal to 50°. The remaining

joint set, which has the dip angle of 34° has dip direction 290°. This means this joint set is dipping 5° north from the negative x direction (note that the negative x direction is N 75° W). Therefore, the discontinuity sets that have dip directions close to y or opposite to y direction have no low dip angles. The aforementioned imply that use of 10 m in the y direction is adequate to perform stress analysis with the discontinuities. Figure 3 shows the 3DEC model obtained with the discontinuities belonging to the different rock types. The discontinuity directions seen in this plot agree with the discontinuity set orientations given in Table 2.

Comparisons between Case 1 and Case 2 results given in Table 8 show the effect of joints. The tunnel is located in the magnetite rock mass. One discontinuity set of the magnetite rock mass has a dip direction 5° north from the negative x axis and has a shallow dip of 34°. The second discontinuity set of the magnetite rock mass has a dip direction 25° south from the positive x axis and has a steep dip of 72°. The third discontinuity set of the magnetite rock mass has a dip direction 60° north from the positive x axis and has a moderate dip of 50°. The latter two discontinuity sets can form wedges and slide towards positive x direction at the left tunnel wall. This may be the reason why the left tunnel wall x displacements are higher than the right wall x displacements. The high lateral stress ratio of 1.60 in the x direction has given rise to higher floor displacements compared to that of roof displacement.

Effect of geo-mechanical parameter variability on deformations around the tunnel

In this section, the values of all the intact rock and discontinuity geomechanical parameters have been either increased or decreased from the average (reference) values as a group and the effect of that on maximum tunnel displacements has been investigated. The increased set of values is termed as the stiff system and the decreased set of values is termed as the soft system. For intact rock parameters, a 20% increase or decrease (equivalent to the coefficient of variation of 0.2) have been used, and for joint and discontinuity interface parameters, a 30% increase or decrease (equivalent to the coefficient of variation of 0.3) have been applied (see Table 7: Cases 4–7). The aforementioned coefficient of variation values are considered to be reasonable values to apply for intact rock and discontinuity geomechanical parameters. For change in both the intact rock and discontinuity geomechanical parameters, 1.8–4.9 times differences have been obtained for the maximum displacements between the soft and stiff systems covering both the with and without rock support systems. This indicates the importance of selecting accurate geomechanical parameter values for tunnel behavior numerical modeling.

Table 7. Different cases of numerical stress analyses conducted.

	Joints included or not (Yes/No)	Rock support system included or not (W/WO)	Modeled as Equivalent Continuum? (Yes/No)	Description of values used for intact rock, discontinuities or rock mass
Case 1	NO	WO	NO	Average values for intact rock
Case 2	YES	WO	NO	Average values for intact rock and joints
Case 3	YES	W	NO	Average values for intact rock and joints
Case 4 (soft system)	YES	WO	NO	80% of intact rock average values; 70% of joint and interface average values
Case 5 (stiff system)	YES	WO	NO	120% of intact rock average values; 130% of joint and interface average values
Case 6	YES	W	NO	Rock mass parameters same as in case 3
Case 7	YES	W	NO	Rock mass parameters same as in case 4
Case 8	NO	WO	YES	50% of intact rock average
Case 9	NO	WO	YES	40% of intact rock average
Case 10	NO	WO	YES	30% of intact rock average
Case 11	NO	W	YES	50% of intact rock average
Case 12	NO	W	YES	40% of intact rock average
Case 13	NO	W	YES	30% of intact rock average
Note: With supports is abbreviated as W; without supports is abbreviated as WO				

Table 8. Comparison between different cases on maximum tunnel deformations.

Case/Parameter	Maximum x displacement on left wall (mm)	Maximum x displacement on right wall (mm)	Maximum z displacement on roof (mm)	Maximum z displacement on floor (mm)
(Case 1)	2.74	2.73	3.17	2.95
(Case 2)	12.0	10.5	7.5	10.0
(Case 3)	6.81	5.86	5.3	9.0
(Case 8)	8.96	9.52	8.43	10.19
(Case 11)	5.15	4.98	5.36	9.32
(Case 9)	12.42	13.17	11.05	14.74
(Case 12)	6.58	6.44	6.75	13.2
(Case 10)	17.99	19.73	15.22	22.86
(Case 13)	9.08	8.4	8.98	19.99

Research performed under equivalent continuum approach

In the presence of joints, rock mass mechanical properties show both the scale effects and anisotropic behavior. Due to the presence of joints, both the rock mass strength and deformability parameter values reduce significantly from the corresponding intact rock values. The amount of reduction depends on the fracture network and the difference between the intact rock and discontinuity mechanical property values. Several previous studies have been performed at the three-dimensional level by the research group of Kulatilake to find the Representative Elementary Volume (REV) and equivalent continuum mechanical properties of different real world rock masses. For a granitic gneiss rock, Kulatilake et al. (1993) have obtained a mean rock mass deformation modulus equal to about 30% of the intact rock Young's modulus. For the same rock, they have obtained a mean rock mass shear modulus

equal to about 40% of intact rock shear modulus. For Aspo diorite rock located at a depth of 485 m at Aspo Hard Rock Laboratory in Sweden, Kulatilake et al. (2004) estimated mean rock mass strength to be about 47% of the intact rock strength. For the same site, they found mean rock mass modulus to be 51% of the intact rock Young's modulus and mean rock mass Poisson's ratio to be 21% higher than the intact rock Poisson's ratio. In a recent study, for a limestone rock mass, Wu and Kulatilake (2012) have obtained a mean rock mass strength equal to about 45% of the intact rock strength. For the same rock mass, they have obtained a mean deformation modulus equal to about 50% of the intact rock Young's modulus. The mean shear and bulk moduli for the same rock mass have turned out to be about 40% and 30% of the intact rock shear modulus and bulk modulus, respectively. The mean Poisson's ratio for the rock mass turned out to be about 14% higher than the intact rock Poisson's ratio.

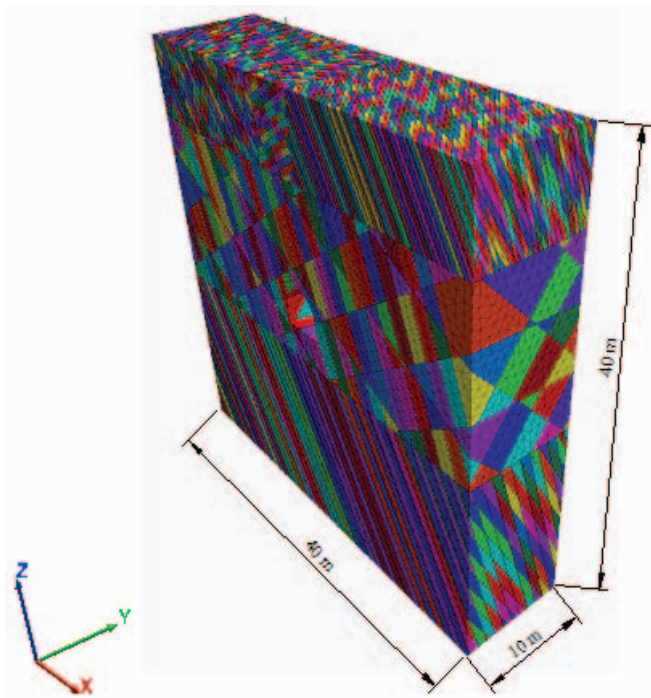


Figure 3. 3DEC model with discontinuities belonging to different rock types.

For the study described in this paper, no attempt was made to estimate REV and equivalent continuum mechanical properties as in the three previous studies stated above. Instead, it was decided to perform 3DEC calculations using different percentages (50% to 30% in steps of 10%) for the ratio of rock mass strength and deformability properties/intact rock strength and deformability properties. Because all these calculations attempt to simulate equivalent continuum behavior, joints were not included for any of the lithologies. Some of the calculations were conducted without rock support (Table 7: Cases 8–10), and the rest of the calculations were performed with rock support system (Table 7: Cases 11–13). The goal of these calculations was to see which percentage of equivalent continuum calculation would provide maximum deformations around the tunnel similar to the discontinuum approach calculations reported earlier. The results coming out of that then can be compared with the results stated in the previous paragraph on the equivalent continuum properties.

Table 8 shows, as expected intuitively, that the maximum deformations around the tunnel increase with decreasing equivalent rock mass parameters under, without, and with rock support systems, respectively.

Comparison of results between the discontinuum approach and equivalent continuum approach

Table 8 provides a comparison for the maximum displacements around the tunnel between the continuum and discontinuum analysis results. Note that in Table 8 the comparison should be made separately under without rock support condition and with rock support condition. For both under the without as well as with rock support conditions, the best comparison between the discontinuum analysis results and continuum analysis results is obtained for the continuum analysis cases that used 40%–50% of

the intact rock property values. This result agrees very well with the previous case study results obtained by Kulatilake et al. (1993; 2004) and Wu and Kulatilake (2012).

Effect of rock support system on deformation around the tunnel

Table 8 provides comparisons for maximum displacements around the tunnel between the no support and with rock support cases. Note that the rock supports have reduced the tunnel roof and wall deformations more compared to that of the floor. The main reason for this is that the rock supports were applied only on the roof and walls, but not on the floor. From Table 8, it is clear that the rock support system provides sufficient control on the deformation around the tunnel excavation and hence stability of the tunnel to provide satisfactory exploitation of magnetite resources.

FIELD DEFORMATION MEASUREMENT RESULTS AND COMPARISON WITH NUMERICAL PREDICTIONS

JSS30A-10 multi-point extensometers manufactured by Bsidia Instrument Company in China, each with a measuring range of 0.5–30 m with 0.1 mm precision, were used to monitor the deformation on the excavation surface as well as at interior points of the two walls of the tunnel. Along the tunnel axis direction (y-direction), monitoring was conducted at two locations (points 1 and 2), which were 40 m apart. Multi-point extensometers were installed in June 2007. All the deformation data were monitored under usual tunnel excavation/rock support sequence procedure of a tunnel. The total monitoring duration was 186 days before extracting the ore from the monitoring area. The results are shown in Figure 4. This figure shows that the deformations have stabilized after about 173 days. As expected intuitively, for each wall and each monitoring point, the highest deformation occurred at the excavation surface, and it reduced as the monitoring point goes further into the rock mass. Point 1 measurements compare quite well with point 2 measurements. Also, a good comparison can be seen between the right wall and left wall measurements.

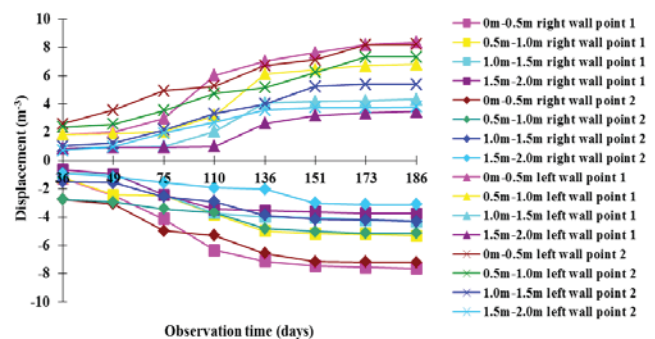


Figure 4. Deformation data from tunnel walls at a depth of 480 m from monitoring points 1 and 2.

Figure 5 shows a comparison between field deformation monitoring results and different numerical analyses results. Overall, the monitored field deformations compare very well with the numerical results shown in Figure 5. For the discontinuum analysis, the soft parameter values provide displacements slightly higher than the field values and the average values provide displacements slightly lower than the field values. The stiff parameter values

provide displacements lower than the field displacements. A perfect agreement can be obtained between the field values and numerical calculations if about 92% of the intact rock average values and about 87.5% of the discontinuity average values are used for the geomechanical parameters in performing discontinuum analysis. For the equivalent continuum analysis, 40% of the intact rock average values provide deformations slightly less than the field deformations and 30% of the intact rock average values provide deformations slightly higher than the field deformations. A perfect agreement can be obtained between the field values and numerical calculations if about 35% of the intact rock average values are used in the equivalent continuum analysis.

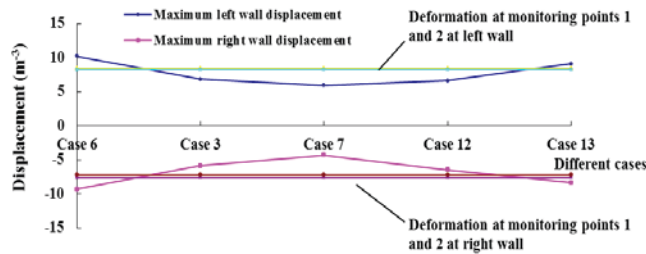


Figure 5. Deformation comparisons between field monitoring results and different numerical stress analyses results.

CONCLUSIONS

The following conclusions can be made from this study.

1. Under no rock support conditions, the maximum deformations around the tunnel increased by about 2.4 to 4.4 times by adding discontinuities to the geomechanical model which had only intact rock. The highest increase was observed on the walls of the tunnel. Note that in this application, the maximum in situ stress was acting perpendicular to the tunnel walls. The discontinuity sets have contributed to high wall deformations. The presence of discontinuities also leads to greater chance of instability around the tunnel excavation.
2. For change in both the intact rock and discontinuity geomechanical parameters, 1.8–4.9 times differences were obtained for the maximum displacements between the soft and stiff combined geomechanical property systems covering both the with and without rock support systems.
3. For both under the no support as well as with rock support conditions, the best comparison between the discontinuum analysis results and continuum analysis results was obtained for the continuum analysis cases that used 40%–50% of the intact rock property values. This result agrees very well with the previous equivalent continuum case study results obtained by Kulatilake et al. (1993; 2004) and Wu and Kulatilake (2012).
4. The applied rock support decreased the wall deformations by about 43%–51%, roof deformations by about 29%–39%, and floor deformations by about 9%–10%. Note that the rock supports were installed only to the ribs and the roof of the tunnel but not to the floor. This is the reason why floor deformations were reduced less compared to roof and wall deformations. The applied rock support increased the stability of the tunnel to use it for satisfactory exploitation of magnetite resources.
5. The monitored field deformations compared very well with the numerical results. For the discontinuum analysis, a perfect agreement can be obtained between the field monitored deformations and numerical stress analyses results if about 92% of the intact rock average values and about 87.5% of the discontinuity average values are used for the geomechanical parameters in performing the discontinuum analysis. For the equivalent continuum analysis, a perfect agreement can be obtained between the field monitored deformations and numerical stress analyses results if about 35% of the intact rock average values are used as the rock mass geomechanical parameters in the equivalent continuum analysis.

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A Comparison of Numerical Methods and Analytical Methods in Determination of Tunnel Walls Displacement – A Case Study

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ABSTRACT

Tunnel stability has an important role in the production process of an underground coal mine. There are various methods for analysing tunnel stability, such as numerical methods and analytical methods. In this paper, numerical methods (Phase2 software) have been used to determine tunnel wall displacement in a mining tunnel of the Parvade underground coal mine. The Ground Reaction Curve has also been drawn using analytical methods to determine tunnel wall displacement. The comparison of results from the numerical method and the analytical method show a noteworthy difference in the tunnel wall displacement. The displacement calculated by the numerical method shows a lower value than that of the analytical method because the numerical method is more suited to modeling the various coal and rock layers, as well as the shape of the excavations found in coal mines. The methodology used in this paper together with the results obtained from both methods can serve as useful tools for the coal mine design engineer when determining the ground support requirements.

INTRODUCTION

Excavation and ore extraction processes cause problems such as a displacement field generated in the orebody and the surrounding rock. This is one of the major engineering problems that engineers are faced with in underground mining (Singh, Singh, and Murthy, 2010). In the design process of an underground coal mine, the stability of tunnels is an important parameter that should be studied carefully because it has an important role in the production process of the mine. Instability and collapse of tunnels may cause different damages. Such damages not only result in an increase in costs, but also are dangerous towards the miners. Therefore, accurate analysis and determination of the displacement in the tunnel's roof and walls can help implement a suitable support system and therefore, make the tunnel more stable.

In this paper, the displacement fields in the roof and walls of tunnels in the Parvade coal mine have been studied. This mine is one of the main coal mines in Iran. Several collapses have occurred in this mine and instability of tunnel's roof and walls is one of the most important problems in this mine.

There are various methods for analysing tunnel stability and the determination of tunnel walls' displacement. Two of the main methods are analytical methods and numerical methods. These methods have been used widely in order to analyse the stability of tunnels during the design process of underground coal mines.

In this research, both methods have been used in order to determine the displacement fields in the case study. A comparison between the two methods was carried out that indicated that there is some difference between the results obtained from the two methods.

After comparing these methods, the suitable method can be chosen based on these results and the mine conditions for the correct calculation of tunnel's roof and walls displacement. The significance of this comes into light when one seeks a suitable implementation method and installation of the support system in the tunnel.

PARVADE UNDERGROUND COAL MINE

The Tabas coal region is one of the most comprehensive coal resources in Iran. Tabas underground mine is the main mine of the Iranian Coal Enterprises and is located in the Khorasan Province of Iran. The Parvade coalfield lies approximately 85km south of the city of Tabas in mid-eastern Iran. The total probable anthracite reserve in the Parvade region is approximately 1.2 billion tons. Furthermore, the minable reserve suitable for underground production is 28 million tons in the Parvade1 (Manteqi, Shahriar, and Torabi, 2012). Physical and mechanical properties of the rock mass used for numerical modelling are presented in Table1 (IRITEC, 2003; Manteqi, Shahriar, and Torabi, 2012).

ANALYTICAL METHODS: CONVERGENCE CONFINEMENT METHOD

Analytical methods are very useful in geomechanics because they provide results with very limited effort and highlight the most important variables that determine the solution to problems (Bobet, 2010). In this paper, we used analytical methods (convergence confinement method) to draw the Ground Reaction Curve in order to determine the tunnel walls' displacement in a mining tunnel of the Parvade underground coal mine. The convergence confinement

method has been described by other authors (e.g., Gesta et al., 1980; Duncan Fama, Fairhurst, and Carranza-Torres, 2002) and has been commonly used for the support system design in conventional tunnelling. The method achieves a 2D simplified approach for resolving 3D rock support interaction problems.

The main assumption of the convergence confinement method is that the support load required in order to stabilize the excavation decreases with inward tunnel displacement. As the boundary rock moves inward, tangential stresses increase, which results in both yielding of the rock mass and increased confining stresses on the surrounding (Carranza-Torres and Fairhurst, 2000).

GROUND REACTION CURVE

The Ground Reaction Curve (GRC) can be defined as a curve that describes the decrease of inner pressure and the increase of radial displacement of the tunnel's wall. The GRC analyses the behaviour of the rock mass surrounding the tunnel. Based on the applied pressure on the support system, the GRC determines the tunnel convergence or the displacement of tunnel walls. A typical GRC is shown in Figure 1.

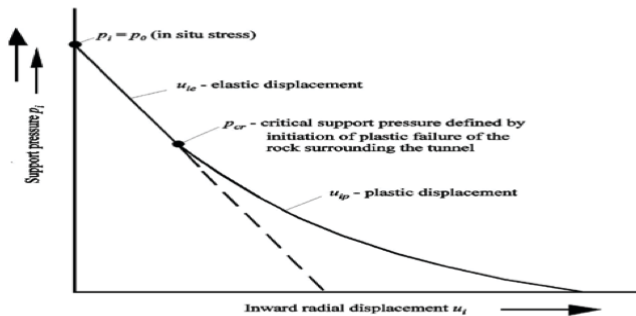


Figure 1. Typical ground reaction curve.

In general, two different types of methods are used to calculate the GRC in a tunnel: analytical methods, which are expressed in mathematical terms and numerical methods, where the differential equations of the problem have to be solved numerically. In this research, the Duncan-Fama analytical method for drawing the ground reaction curve has been used, as implemented in the 'RocSupport' software (RocScience Inc., 2000-2005).

DUNCAN-FAMA ANALYTICAL METHOD

This method is based on the Mohr-Coulomb failure criterion. Therefore, this method requires parameters such as modulus of elasticity (MPa), Poisson's ratio, internal friction angle, and rock mass compressive strength in order to draw the ground reaction curve (Carranza-Torres and Fairhurst, 1999; 2000). Furthermore, the tunnel was excavated in the coal seam and the weakest member of rocks based on the table 1 for drawing the ground reaction curve has been used. The ground reaction curve for Figure 2 has been drawn using the Duncan-Fama analytical method (RocScience Inc., 2000 – 2005). As evident from Figure 2, the maximum tunnel wall displacement calculated by the Duncan-Fama method is 164mm.

NUMERICAL MODELLING OPTIONS

Numerical methods are tools that enable engineers to evaluate, both qualitatively and quantitatively, the effects of geology on the design and the consequences of the design on geology. Numerical modelling applications are intended to provide mining and rock mechanics engineers a better understanding for solving problems related to the mine layout and roof support system design to enable consistent and techno-economic viable performance of mining structures throughout their planned life of operations (Singh, Singh, and Murthy, 2010). Numerical methods are promising and effective tools in understanding the rock mass response subjected to complex loading conditions.

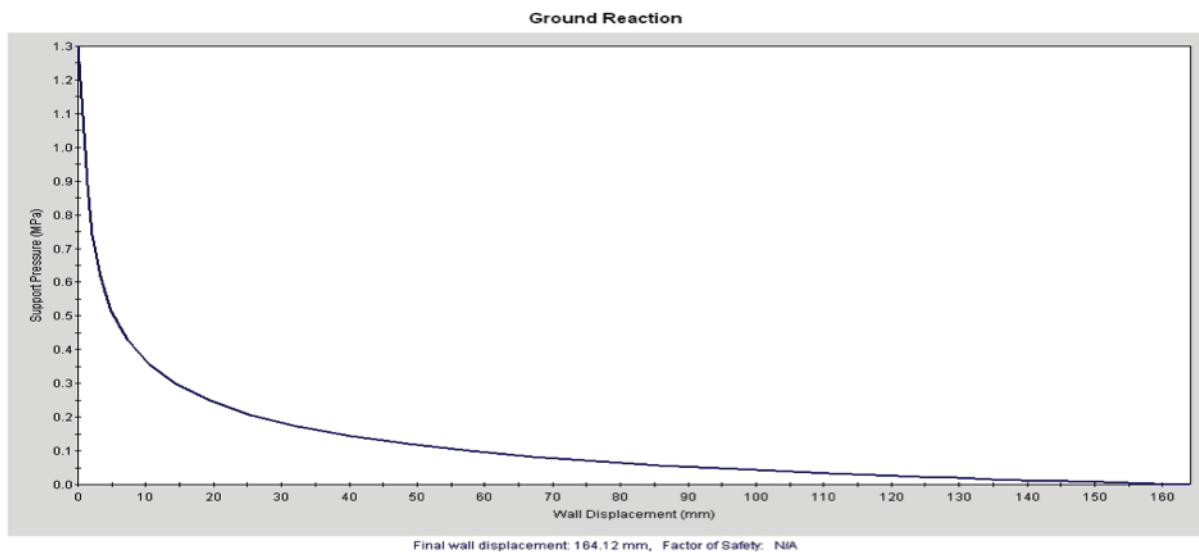


Figure 2. Ground reaction curve drawn using the Duncan-Fama method.

Table 1. The rock mass input parameters used in numerical modelling (Manteqi, Shahriar, and Torabi, 2012).

Rock Definition	Siltstone	Sandy Siltstone	Silty Mudstone	Coal	Mudstone	Sandstone
Definition code	1	2	3	4	5	6
Density (M/N/m ³)	0.0272	0.0271	0.0268	0.016	0.0263	0.027
Internal Friction angle(ϕ)	27.42	31.75	22.17	15.76	20.13	43.52
Cohesion c (MPa)	0.357	0.443	0.257	0.084	0.231	0.767
Modulus of elasticity E (MPa)	2238	2818	1778	749	1995	3548
Tensile strength (MPa)	0.012	0.007	0.005	0.002	0.013	0.017
Poisson's ratio ν	0.25	0.25	0.28	0.25	0.31	0.25
Bulk modulus ^a (K) (MPa)	1492	1878	1347	499	1750	2365
Shear modulus ^b (G) (MPa)	895	1127	695	299	761	1419
Uniaxial compressive strength (MPa)	0.273	0.287	0.114	0.015	0.165	1.01
^a (K) = $E/3(1-2\nu)$ ^b (G) = $E/2(1+\nu)$						

The numerical methods for rock mechanics problems are divided into two main groups: the continuum methods and the discrete methods. Although the choice of continuum or discrete methods mainly depends on the problem scale and fracture system geometry, the choice also depends on other problem-specific factors (Jing and Hudson, 2002).

For the continuum methods, there are three main approaches: the Finite Difference Method (FDM), the Finite Element Method (FEM), and the Boundary Element Method (BEM). The main discrete methods include the Discrete Element Methods (DEM) and the Discrete Fracture Network (DFN) methods (Jing and Hudson, 2002). It is important to be aware that the drawback of analytical methods is that they cannot represent complex in-situ conditions and geometry, and this is the main reason that the numerical methods are used. The numerical methods should be used carefully and with a complete understanding of the background of the problem that is being analysed in order to obtain accurate conclusions.

FINITE ELEMENT METHOD

The finite element method is perhaps the most widely applied numerical method across the science and engineering fields. Since its origin in the early 1960s, much finite element method development work has been specifically oriented towards rock mechanics problems (Jing and Hudson, 2002). This is because it was the first numerical method with enough flexibility for the treatment of material heterogeneity, non-linear deformability, complex boundary conditions, in-situ stresses, and gravity.

In this paper, we used the Phase2 software to determine the tunnel walls' displacement in this particular case study. Phase2 is a two-dimensional finite element program that calculates stresses and displacements around underground openings. It can be used

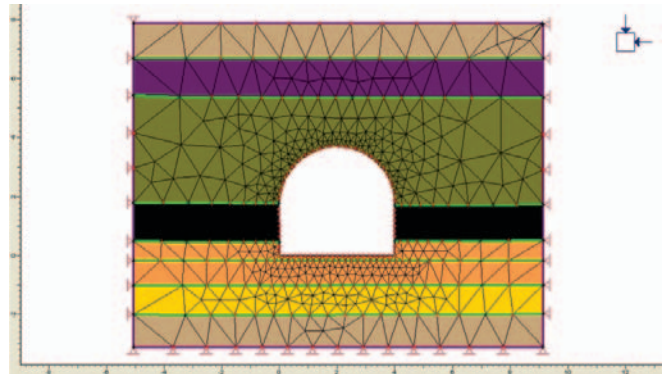


Figure 3. The created model in Phase2.

to solve a wide range of mining and civil engineering problems (Roccience, Inc., 2002).

MODELLING IN PHASE2

The first step of numerical analysis with the Phase2 software is modelling the underground opening in the computer. In this stage, we enter and edit the model boundaries, in situ stresses, and material properties and create the finite element meshes. The created model in Phase2 is shown in Figure 3.

The extracted results of the software are shown in Figure 4. As it can be seen in Figure 4, the horizontal displacement on the tunnel walls is high. Furthermore, Figure 4 shows the deformation vectors on the tunnel walls.

According to the extracted results from Phase2, the maximum tunnel wall displacement is 20mm (Figure 5).

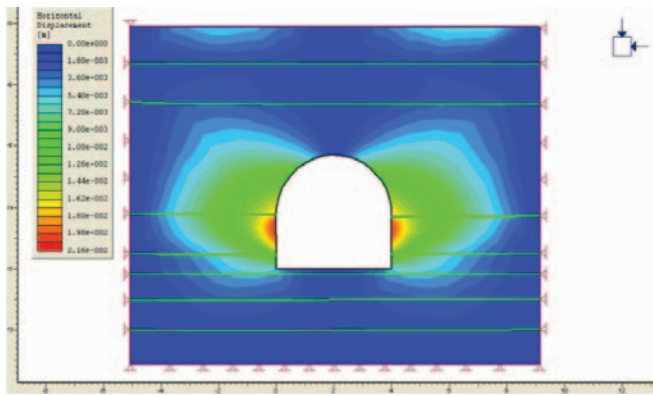


Figure 4. Horizontal displacement around the tunnel (Note: on the tunnel walls).

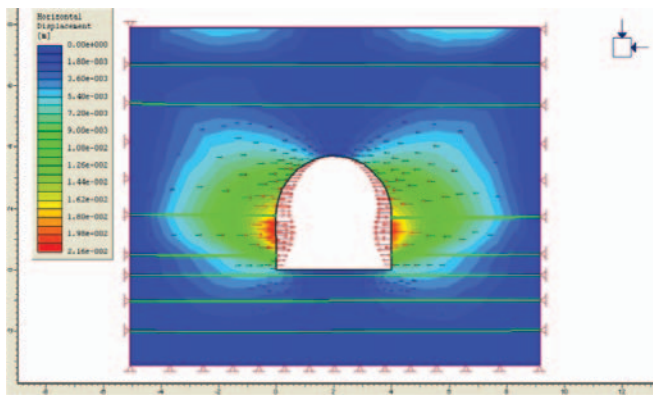


Figure 5. Maximum tunnel wall displacement.

The displacement of the tunnel walls calculated by the numerical method (Phase2 software) shows a lower value than that of analytical method (Duncan-Fama method).

CONCLUSIONS

In this paper, the displacement field was calculated for the mining tunnel's roof and walls of Parvade coal mine. It is necessary to determine the displacement in the tunnel's roof and walls after excavation so that suitable implementation and installation of support systems take place in the tunnel due to the existence of historical collapses in the Parvade mine. The displacement field was calculated using both analytical and numerical methods.

After comparing the results obtained from each method, it was found that the numerical method shows a lower value of displacement in the tunnel walls in comparison with the analytical method. There was some difference between these methods' results owing to the fact that their capabilities and assumptions are different to one another. The analytical method requires several simplifying assumptions, which contributes to large displacements being predicted. Firstly it assumes that the excavation is circular in a uniform stress field, which is seldom true in coal mining. In addition, the method can only consider a single material type. In this study, it was assumed the entire rock mass consists of coal, the weakest material. This simplifying assumption led to the very large predicted displacements. On the other hand, the finite element model was able to accurately simulate the various rock types, field

stress conditions and shape of the excavation, producing more realistic displacement results. After this comparison, the finite element method appears to be a more suitable method for stability analysis of tunnels in coal mines (based on these results, the mine observed conditions and engineering judgment). Therefore, special care has to be taken when using each of these methods since their assumptions and limitations may have a great effect on the final result. Furthermore, one of the objectives of this research was to show these different assumptions and limitations and their effect on the results.

Analytical solutions, often have limited application because they must be used within the range of assumptions made for their development. Such assumptions usually include elastic behaviour; homogeneous, isotropic material; time independent behaviour; quasi-static loading, and the ratio of horizontal stress to vertical stress being constant (it is often equal to 1). Rocks and soils may not be isotropic or homogeneous and the loading may not be static. Additionally, the geometry of the problem may be complex. In these cases, solutions can only be obtained numerically. One of the other limiting assumptions in drawing the ground reaction curve (and determining the tunnel walls' displacement) is the circular cross section of the tunnel. Numerical methods can widely be used in order to perform stability analysis in every underground excavation with different shapes and dimensions without the analytical method's limited assumptions.

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Panel-scale Modeling of a Deep Longwall Panel: the MULSIM Alternative

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ABSTRACT

Panel-scale modeling of stress changes induced by longwall coal mining is a challenging problem, particularly in deep mines of the western United States. Two common approaches to this problem are the empirical and boundary element modeling methods. One boundary element method, LaModel, was recently calibrated to field measurements at a mine in Colorado (Larson and Whyatt, 2012). However, the calibrated overburden properties were extreme, resulting in very low estimates of gob loading. Next, the empirical method was calibrated to field measurements and applied to the problem, but this approach clearly extrapolated this method well beyond the characteristics of underlying cases. This paper examines a third option, the elastic continuum boundary element program Mulsim. The program was expanded to handle LaModel-size meshes in a version dubbed Mulsim/Large. Properties for the elastic, homogeneous overburden model in Mulsim were estimated as a thickness-weighted average of typical published elastic properties for each major stratum. These properties, along with a commonly used pillar strength equation and coal properties, produced the measured load transfer distance and reasonable gob loading without further calibration. Given the number of input parameters required for all of these methods, it is not surprising that LaModel and the empirical approach could be adjusted further to incorporate similar gob loads. However, these extraordinary measures failed to bring gate road pillar loading in line with Mulsim results. Differences in pillar load estimates continued to be significant (as large as 65% in this study). These results demonstrate that, for this particular case, the choice of analysis method impacts results in ways that cannot be removed by calibration of input parameters. Finally, the natural fit between the Mulsim model and mine conditions suggests that approximating overburden at the particular site used for this research as an elastic continuum is superior to the alternatives examined.

INTRODUCTION

The success of underground mining rests, in part, on chain pillars and abutment areas safely supporting loads induced by mining. Overstressed regions may collapse or deform in a variety of ways, some of which are potentially hazardous. Thus, the Office of Mine Safety and Health Research (OMSHR) of the National Institute for Occupational Safety and Health (NIOSH)

has been investigating various methods of producing, refining, and confirming estimates of mining-induced loading of these areas. These methods are simple models that seek to represent dominant mechanisms (e.g., loading by deforming elastic beam or plate) of ground response to mining. While no model can include all the complexities of overburden over a longwall panel or district, it is important that principal assumptions are in line with dominant mechanisms at a particular site. Indeed, a model that is well suited for one site may be poorly suited for another. This investigation examines performance of three models of overburden behavior at a particular deep western longwall coal mine. It builds on a previous study that examined the consequences of modeling method for a hypothetical longwall mine operating in strong strata (Larson and Whyatt, 2009).

The first model is empirical—that is, it is not based in physics, but rather on a functional form that best fits the measured observations, using regression to determine the model parameters. It follows, then, that the empirical method may not reflect a particular physical model of overburden behavior.

The other two models are based on physical models. The first, LaModel, assumes overburden (as well as strata below the coal seam) to act like a stack of elastic plates with frictionless interfaces. The second model, Mulsim, assumes a homogeneous elastic continuum.

The empirical and LaModel methods were applied to modeling of load transfer distance at this site in a previous paper by Larson and Whyatt (2012). Results were not entirely satisfactory as the empirical approach was applied in conditions far removed from underlying cases, and the LaModel calibration called for extremely stiff overburden properties that precluded development of gob loading. These results led to a search for an alternative model that settled on the Mulsim program (Zipf, 1992a, 1992b), which was updated and recompiled for this study. The bulk of this paper concerns the application of Mulsim and an examination of how results are impacted by changing the analysis method.

METHODS AND CALIBRATION

The first step in the calibration process is to adjust rock mass stiffness to achieve load transfer distance (Heasley, 2008, 2009;

Heasley, et al., 2010). This step, covered by Larson and Whyatt (2012), is briefly reviewed here along with a description of these methods.

Empirical Method

The empirical approach used here combines two equations based on observations from a few cases. The first, proposed by Peng and Chiang (1984), defines load transfer distance (LTD) as a function of overburden depth (H) based on a small number of observations. This equation is

$$LTD = 9.3\sqrt{H} \quad (1)$$

The second equation describes the decay of mining-induced stress, σ , as a function of distance from the abutment rib, x , within the region between the edge of an area of gob and the LTD. This rule, proposed by Mark (1990), is based on a curve fit of a small group of pressure measurements. The magnitude of mining-induced stress carried in a two-dimensional slice within this region, L , depends on the angle, β , which governs partitioning of undermined overburden loading between abutments and gob. (See Figure 1, where L is represented by L_s or L_{ss} .)

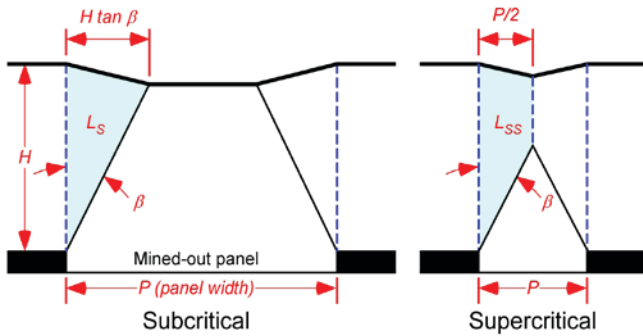


Figure 1. Conceptualizations of the side abutment load, patterned after King and Whittaker (1970). (Figure after Mark, 1990.).

$$\sigma = \frac{3L(\beta)}{(LTD)^3} [(LTD) - x]^2 \quad (2)$$

The part of the overpanel weight that is not derived from the volume in L is that which is transmitted through the gob. This proportion is determined by β , H , and P (the panel width).

These calculations are easily executed and adjusted in a fairly simple spreadsheet, which is the approach taken in this study. Lawson et al. (2013) explore this approach in considerable detail in a companion paper. This implementation provides ready access to calculations, easily illuminating which parameters (such as H , P , β , or LTD) dominate results, and providing opportunities to revise both equations and parameters to reflect local ground response characteristics. The speed and transparency of this approach are significant advantages. Larson and Whyatt (2012) applied these equations to a deep western coal mine, noting that the LTD was found to be four times that estimated by Equation 1. LTD is easily adjusted by simply inserting a fictitious depth in Equation 1, or by inserting LTD as a constant in Equation 2. However, Larson and

Whyatt (2012) also suggest that the extreme equivalent depth of 35,912 ft implied by this LTD is well removed from the underlying case depths ranging from approximately 380 ft to 875 ft (Peng and Chiang, 1984: p. 64). Pillar load estimates derived from this approach are examined later in this paper.

LaModel Calibration

LaModel, a code with a displacement discontinuity formulation in a laminated medium, has been very successful as an evaluation tool for coal mine layouts (Heasley, 1998, 2008; Heasley, 2011; Heasley, et al., 2010). It is a solution for a crack (filled with coal material) in a medium consisting of a series of plates with frictionless, cohesionless interfaces. One of the input parameters is the thickness of these even, repeated plates. The result of such formulation is that the laminated rock mass can be more flexible than a strictly elastic medium, thus enabling LaModel to simulate the surface effects of subsidence reasonably well, but at the cost of accuracy in load distribution (Heasley, 2008).

Heasley and his fellow researchers have provided specific guidance on model calibration (2008, 2009; Heasley, et al., 2010), and Heasley has also provided guidance on adjusting input parameters for site-specific conditions (2012). The basic steps of calibration are (1) calibrate rock mass stiffness to load transfer distance; (2) calibrate gob stiffness to reflect appropriate amount of load through the gob; and (3) calibrate rock mass strength. Larson and Whyatt (2012) demonstrated, with LaModel, a method of calibrating rock mass stiffness to load transfer distance, determined using a 20-psi increase with borehole pressure cells (BPCs) as a threshold. However, for reasons described below, they did not proceed with the rest of the calibration—that is, calibrating gob stiffness and coal strength—which are the last two steps of the calibration process outlined by Heasley.

Larson and Whyatt (2012) calibrated rock mass stiffness to load transfer distance with the LaModel program in a panel-scale model of the mine used in this study, applying the method recommended by Heasley and Heasley et al. (Heasley, 2008, 2009; Heasley, et al., 2010). The initial stage was successful in matching LTD but required an extreme set of overburden properties ($E = 37,461,800$ psi, $t = 443.31$ ft). Heasley's second step suggests adjusting properties to attain "reasonable" gob loading. While gob loads are difficult to measure, estimates of 20% to 60% are commonly used in pillar design, based on common geometries and calculations using ALPS standards (e.g., Mark, 1990). Unfortunately, the gob load resulting from overburden property calibration was very small. Attempts to increase gob loading through calibration of other model parameters were not successful. The best results obtained are listed in Table 1. Other parameter values resulted in even more unrealistic results. These results show a trade-off between realism in load transfer distance and gob loading. In short, it was not possible to accomplish calibration of gob modulus to both observed load transfer distance and reasonable gob loading at this site.

The Mulsim Alternative

Mulsim is an indirect boundary element method¹ (Crouch and Starfield, 1983). It is based on the analytical solution of a

¹ An indirect boundary element method refers to the fact that there is an intermediate step in the solution process—that is, the method first solves for the singularities that satisfy the specified boundary conditions, and then computes the rest of the boundary parameters in terms of these singular solutions.

Table 1. List of LaModel cases with resulting load transfer distance and percent loading through the gob.

Case	Coal modulus, psi	Final gob modulus, psi	¹ Load transfer distance, ft	² Percent of load through gob
1	370,000	6,000,000	409	9.94%
2	370,000	4,000,000	1,048	7.90%
3	250,000	4,000,000	566	10.02%
4	250,000	3,000,000	851	8.33%
5	150,000	2,250,000	761	9.96%
6	150,000	2,000,000	1,223	9.11%
7	50,000	660,000	635	9.98%
8	50,000	650,000	973	9.80%
9	50,000	500,000	1,788	6.69%
¹ Target load transfer distance was 1,764.2 ft. ² Percentage of load through gob according to empirical assumption = 23.1%.				

constant displacement discontinuity over a specific line segment in an infinite elastic medium. Mulsim differs from LaModel fundamentally in that the overburden equations derive from an elastic medium without laminations. The three-dimensional formulation can be traced to Sinha (1979), who drew on the work of Crouch (1976) and Salamon (1963, 1964a, 1964b). It was further developed by Beckett and Madrid (1986, 1988) as Mulsim/BM, which expanded the number of materials to 26, included gob elements and structural elements such as packwalls and cribs, and included fine grids for an area of interest. Zipf (1992a, 1992b) added nonlinear elements, multiple mining steps, and energy release and strain energy computations into a version he called Mulsim/NL. Maleki (see Maleki, et al., 2003) added the ability to include effects of topography and added other enhancements in a proprietary version of the code that he called MulsimTI. The authors received the source code of Mulsim/NL from Dr. Zipf and expanded it to handle larger problems (450 x 450 coarse elements and 2000 x 2000 fine elements within those coarse elements) and up to 52 in-seam materials for the purpose of this study, calling it Mulsim/Large. In this paper, the code is typically referred to as Mulsim for brevity. It is potentially a good predictor of mining-induced stress redistribution where the overburden has strong, stiff members, as is frequently the case in the western United States. (Larson and Whyatt, 2009).

Table 2 lists features of LaModel and the current NIOSH version of Mulsim. There are some important differences in capabilities that may influence which tool to use. One not listed is the comparative computational time required. In the size of problem modeled for this paper, Mulsim/Large took 400 to 500 times longer to complete computations. Computation time with Mulsim/Large can be reduced by minimizing the number of coarse blocks that become fine elements.

Constructing inputs and handling outputs for a large number of excavations for both LaModel and Mulsim can be a challenge. Heasley's LamPre 3.0 (2010) is a very useful tool, but it handles a

maximum of 20 mining steps, even though LaModel 3.0 can handle 200 mining steps. Some practical methods learned by the authors for constructing input files for a large number of mining steps for both LaModel and Mulsim are described in the Appendix.

APPLICATION OF MULSIM

An equivalent Mulsim model was constructed for this same deep western mine. LaModel inputs were converted to Mulsim format, maintaining as many details as possible, including meshing of the coal seam. The overburden model was the exception. Mulsim overburden properties were estimated as a thickness-weighted average of estimated elastic parameters of each member. A stratigraphic column representative of the area (Robeck, 2005; see his Figure 3) was idealized further as shown in Figure 2. Estimated properties for each type of strata are listed in Table 3. Properties are in line with ranges reported by several authors (Farmer, 1968; Vutukuri, Lama, and Saluja, 1974; Lama and Vutukuri, 1978a, 1978b, 1978c; Pariseau, 2012). The seam mined was the DU Seam. Drilling experience suggests this seam is a somewhat softer and weaker than other seams in the column. Initial models examining load transfer distance used an elastic coal model. Later models used the Mark-Bieniawski pillar model (Mark and Chase, 1997) with 900 psi coal, also in accord with LaModel runs. The gob modulus was determined with the default procedure in LamPre (Heasley, 2008, 2009; Heasley, et al., 2010), as it was for both programs.

Model-estimated load transfer distances were determined using the 20-psi-stress-increase threshold that Larson and Whyatt (2012) used with BPC measurements. The Mark-Bieniawski coal strength model produced an average load transfer distance estimate of 1,845.0 ft, which is 4.5% above the measured average, but this average ignores BPC locations affected by yielded elements in which there was no stress increase resulting from panel mining. The average load transfer distance determined with this model was, thus, artificially high at 1,845.0 ft, but just 4.5% above the measured average of 1,764.2 ft. Using an elastic model to minimize anomalies caused by yielding elements, initial model results, without further calibration, produced an average model-estimated load transfer distance that was 1,628.2ft, just 7.7% below the measured average. This load transfer distance result stands in stark contrast to initial LaModel results, which were roughly 75% below the measured average. The initial Mulsim model was used, unaltered, as a basis for subsequent analyses.

Models were also compared by the portion of undermined overburden weight carried by the gob. These results are summarized in Table 4. The Mulsim result has an effective β (for this panel width and depth) of 13.2°, which contrasts with the empirical database standard of 21°. However, this effective β is within the range reported by Tulu and Heasley (2012). While the correct proportion carried by the gob is difficult to know, values in the 20% to 60% range are credible. By comparison, the range in the table, with the exception of the calibrated LaModel result, are from 23.1% to 47.1%. A value of close to zero would require little or no caving, which, if true, would be evident through observation. With this exception, it is difficult to discount any of these results as unrealistic.

Table 2. Feature comparison of LaModel and Mulsim/Large.

Feature	LaModel	Mulsim/Large
Infinite elastic solid?	Yes	Yes
Laminations?	Yes	No
Effects of stress components other than vertical?	No	Yes
Number of seams possible to include	4	4
Dipping seam?	No	Yes
Coarse elements?	No	Yes
Symmetry capability at boundary?	Yes	No
Fixed capability at boundary?	Yes	Yes
Topography datum?	Yes	No
Maximum number of coarse elements in x and y directions	NA	450
Maximum number of fine elements in x and y directions	2,000	2,000
Maximum number of in-seam materials	52	52
Maximum number of excavations	200	Unlimited

Table 3. Properties of the principal stratigraphic units assumed for components in Figure 2 for interbedded units, proportions of each material were estimated to determine properties. Idealized members and their properties were used, in turn, to determine initial properties for input into LaModel and Mulsim.

Property	Sandstone	Siltstone	Shale	DU seam	Other seams
Young's modulus, million psi	4.000	2.000	1.500	0.3700	0.4000
Poisson's ratio	0.2300	0.2500	0.3500	0.3500	0.3300
Density, lb/ft ³	162.0	162.0	162.0	80.00	85.00
Unconfined compressive strength, psi	1,400	8,000	4,000	900.0	1200
Friction angle, degrees	35.00	30.00	30.00	35.00	35.00
Dilation angle, degrees	5.000	5.000	5.000	5.000	5.000
Tensile strength, psi	1,400	800.0	400.0	90.00	120.0

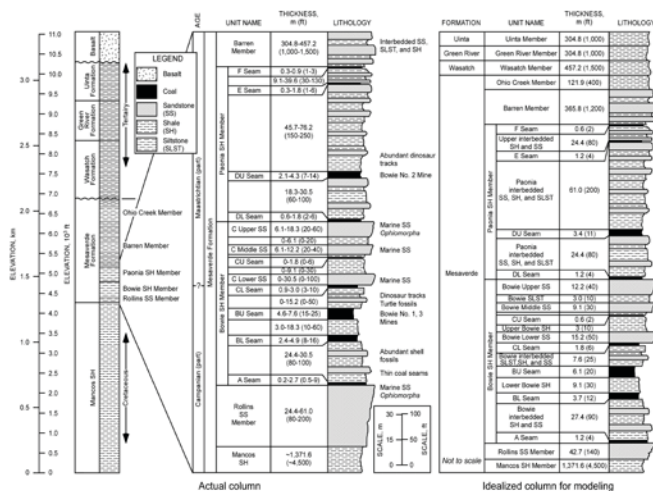


Figure 2. Generalized stratigraphic column in the area of the case study. Idealized column for case study was based on this column. (Figure after Robeck, 2005.).

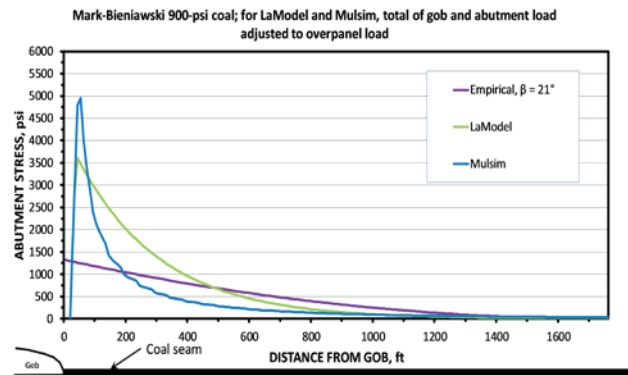


Figure 3. Abutment stress (stress increase above the pre-mining stress on the abutment) profile of case study with no gate roads. Total of abutment load and gob load = overpanel weight for all three methods. Gob load proportion is as reported in Table 4.

ABUTMENT STRESS DISTRIBUTION

The primary goal of all of these models is to estimate loading of abutments and gate road pillars surrounding a longwall panel. The solid abutment case is a good starting point as it is the simplest. The panel geometry in this case was simplified by neglecting gate

Table 4. Gob loading for various models.

Model	Gob load, % of overpanel weight ¹
Empirical	23.1
LaModel, default parameters	47.1
LaModel, calibrated rock mass stiffness	0.28
Mulsim, Mark-Bieniawski 900-psi	36.7

¹Total overburden weight for LaModel differed from the rest because it accounted for topography.

roads and crosscuts, which were replaced by solid coal. Results for a vertical cross-section from mid-panel (i.e., halfway down the length of the panel) are plotted in Figure 3. Such plots are a convenient way to consider loading of pillars of various widths. The area under each of these curves is affected by various levels of gob loading. Recall that the calibrated LaModel result has virtually no gob load.

A better comparison might be made by adjusting results to reflect identical amounts of gob loading. Then, these results will vary in the distribution of loading, not the amount. Such a result is plotted in Figure 4, using the empirical estimate of gob loading as a standard. The largest impact is a reduction in near-gob abutment stress for LaModel. The curves in Figure 4 are very similar in shape to those in Figure 5, which was published by Heasley (2000, see his Figure 3). The single difference is that load transfer distance for the present case study is approximately nine times that in Heasley's figure. Notably, the peak stresses in the present study are also smaller than in Heasley's figure, due to the large difference in load transfer distance.

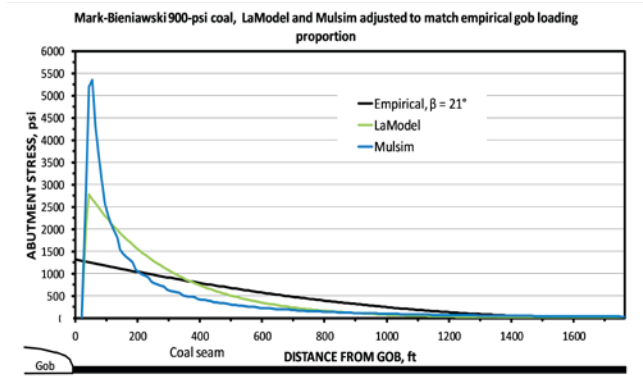


Figure 4. Abutment stress-increase profiles for case study with no gate roads. Loading for LaModel and Mulsim have been adjusted with respect to that in Figure 3 so that gob loading is equivalent to that of the empirical case with $\beta = 21^\circ$.

PILLAR LOAD ESTIMATES

Re-introduction of actual mine pillars in the mesh allows us to estimate loads on pillars at this mine, and assess how these estimates vary with method. The Mulsim result is taken to be the standard for this comparison, as it appears to best describe ground response. Thus, the areas under the curves in Figure 6

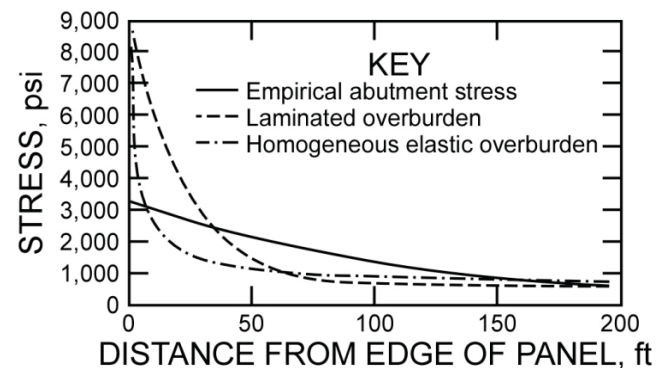


Figure 5. Comparison of the longwall abutment stress computed from the homogeneous elastic model, the laminated model, and the empirical formula. (Figure after Heasley, 2000.).

are somewhat less than those in Figure 4 of the previous section. Figure 6 shows the abutment stress profiles for all three methods after such adjustments. The relative lack of stress concentration around gate roads in the LaModel curve is a result of its very high overburden stiffness.

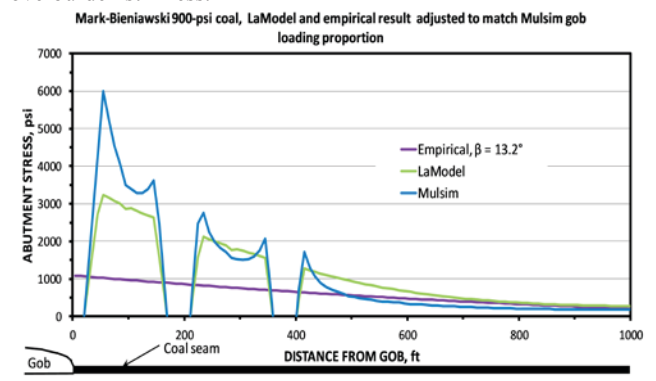


Figure 6. Abutment stress vs. distance from gob for empirical method, LaModel, and Mulsim modeling for the current case study. Empirical method and LaModel total loads remain unchanged, but the gob load proportion of the empirical calculation and LaModel have been adjusted to match that of Mulsim. The angle β matching that abutment load for the empirical curve is 13.2° .

Proportional total pillar load estimates for each method are listed in Table 5, again using the Mulsim result as the standard. While some pillar load estimates are within 5–10%, they are mixed with results that are off by 30% to 65%. Clearly, an error on the order of 50% in estimation of loading is significant.

DISCUSSION AND CONCLUSIONS

Estimation of pillar loads in longwall mines necessarily begins with a choice of analysis method. Larson and Whyatt (2009) showed that choice of method, including the empirical, LaModel, and Mulsim methods examined here, had a significant impact for a hypothetical deep coal mine in strong strata. Given the hypothetical nature of the case, that analysis was based on estimates of overburden properties. This analysis goes a step further—examining consequences in an actual deep longwall coal mine

Table 5. Load on pillars by method, normalized to that of Mulsim.

Pillar	Normalized load on pillar			
	Mulsim	LaModel with 0.28% gob load	LaModel with 36.7%gob load	Empirical with 36.7%gob load
Pillar 1	1.000	1.128	0.716	0.424
Pillar 2	1.000	1.655	1.051	0.777
Note: Pillar 1 is closest to the gob.				

from which observations and measurements of ground response to mining are available. Larson and Whyatt (2012) used this field information to calibrate the empirical method and LaModel to measurements of load transfer distance. This distance was four times that expected from an empirical method (Peng and Chiang, 1984). This result demonstrated that this site is dissimilar from cases underlying the empirical approach. It also required dramatic stiffening of overburden properties in the LaModel analysis—a stiffening that precluded development of gob loading. These results suggested that a different analysis method was needed.

This paper re-introduces the elastic boundary element method for analysis of mining-induced stress around a longwall coal mine. This application is not new (Beckett and Madrid, 1986, 1988; Zipf, 1992a, 1992b), but has been scarce in recent literature. This study revived the Mulsim/NL program that implements this method. The program was expanded to handle LaModel-size meshes in a version dubbed Mulsim/Large. Fortunately, the expanded version of Mulsim used here is well suited to run in parallel with LaModel. An appendix to this paper describes how LaModel input files can be modified for use in such parallel analyses.

Properties for the elastic, homogeneous overburden model in Mulsim were estimated as a thickness-weighted average of typical published elastic properties for each major stratum. These properties, along with a commonly used pillar strength equation and coal properties, produced the measured load transfer distance and reasonable gob loading without further calibration. As such, this method appears to be well suited for deep western coal mines like the case examined here.

Given the number of input parameters required for all of these methods, it is not surprising that LaModel and the empirical approach could be adjusted further to incorporate similar gob loads. This was accomplished by scaling abutment stresses to reflect a desired partitioning of undermined overburden weight to gob and abutment. However, this extraordinary measure failed to synchronize gate road pillar loading. These adjusted results differed from the Mulsim estimates significantly (as large as 65% in this study). These results demonstrate that, for this particular case, the choice of analysis method impacts results in ways that cannot be removed by calibration of input parameters and adjustment of gob loading. That is, the initial choice of analysis method is constraining the possible range of results at this site.

More generally, these results suggest that the analyst should consider the strong simplifying assumptions underlying boundary element programs like LaModel and Mulsim and, for empirical

methods, ground characteristics at underlying case studies. Gross mismatches between assumptions and site ground response are likely to be revealed during calibration when ground properties are assigned extraordinary values. An obvious need for additional adjustment of gob loading or other results outside the analysis method also suggests a mismatch. In such cases, we recommend that the analyst consider alternative methods of estimating the distribution of mining-induced stress.

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APPENDIX: PRACTICAL METHODS FOR CONSTRUCTING INPUT FILES FOR LAMODEL AND MULSIM

Constructing a complicated sequence of excavations for LaModel or Mulsim can be challenging. Heasley's LamPre 3.0 (2010) uses a utility that interfaces with AutoCAD to create a model grid based on excavations indicated in the AutoCAD mine layout map. However, the user still needs to create the excavation sequences. It is not practical to have an AutoCAD map for all excavations desired, especially on a panel scale when many excavations are necessary to track load transfer distance. The authors constructed the excavations another way by superimposing the LaModel grid, consisting of drawn lines between elements, onto the AutoCAD map of the mine layout and constructing the mining layout on a series of sheets of a spreadsheet. Such a grid enabled the authors to determine the basic mine layout with codes for gob, excavation, and elastic material for the remaining coal. Excavation sequences were easily constructed on other mining steps by copying the material codes from the previous mining step to a new sheet and moving the face with copy and paste techniques. While LamPre has

such copy and paste capabilities, the authors found this function to be easier to perform with Microsoft Excel, even though it ultimately required copying to a text editor and using the “Find and Replace” feature to remove all of the tabs. The material codes for each mining step were then imported into a single excavation LaModel input file to take advantage of Heasley’s utility in LamPre to impose yield material codes. This was done separately for each mining step because LamPre has a limit of handling 20 mining steps. These sets of material codes with yield rings were then imported back into appropriate sheets of the spreadsheet to keep a record of the mining-step sequence.

Because Mulsim does not have the option of symmetry on model boundaries, it is absolutely necessary to add coarse elements around the area of interest. In this case, the authors added rings of 20 such elements. Moreover, the fine-element grid used in the LaModel mesh was large enough that Mulsim computational time was prohibitively excessive. To reduce computation time, the authors took the 640 x 1370 LaModel grid for each excavation and reduced it to 410 x 980, taking it from 66 to 475 in the x direction and 181 to 1160 in the y direction. Outside of this fine-mesh boundary, a Visual Basic macro was used to convert each of the 5 x 5 fine groups into an average material strength for a replacement coarse element. This was best accomplished using the Mark-Bieniawski strength criterion for interpolation because it is linear. A coarse material code closest to the average strength of the 5 x 5 unit was selected. Another macro wrote the fine mesh and coarse mesh to separate .txt files, in the format required by Mulsim.

Because there was no pre- or post-processor available for Mulsim, material codes were plotted as contour plots, using the “Set as Categorical” feature in the analysis and graphing software OriginPro² to assign material codes to contour numbers. This capability permitted another visual check of the placement of material codes.

LaModel took just under 27 hours to complete all mining steps. However, one mining step involving gob in Mulsim required 24 to 38 hours on state-of-the-art workstations and up to 48 hours on blade servers made available to the authors.³ As a test, the results of a sequence of 10 mining steps were compared to the results of individual mining steps that each started from the pre-mining state. The differences were insignificant, owing to the monotonic nature of the mining sequence. Therefore, 185 mining steps of the mining sequence were divided into separate models that could be run in parallel. All mining steps for a model were run and completed within four days, with two sets of parallel models with 18 processors having a total of 104 cores. Assembly of the Mulsim input files was done quickly with the aid of DOS batch files.

² Mention of any specific product name does not imply endorsement by NIOSH.

³ These time ranges reflect the condition of running one model per core—thus each core on a workstation ran one model at the same time.

Influence of Mine Layout, Mining Method, and Geology on Down-Dip Coal Barrier Performance at the Solar No. 7 Mine

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ABSTRACT

The Solar No. 7 and Solar No. 10 mines were designed to prevent mine waters from directly discharging to the surface. In January of 2004, a discharge occurred as the Solar No. 7 mine was in its final stage of flooding through a barrier averaging 456-ft in width over a 1,600-ft length (No. 1). Another barrier (No. 2) was designed to protect four municipal water wells operating 1,800-ft away and producing from within or slightly below the Upper Kittanning coalbed from the adjacent Solar No. 7 and No. 10 mine pools. These two barriers were selected for this study because of varied mine layouts, mining methods, and geology factors affecting their performance. Analysis was aided by the collection and geo-referencing of 6-month mining maps from the Pennsylvania Department of Environmental Protection (PA DEP). A structure contour map of the Upper Kittanning coalbed was constructed from mine map elevation points. These data and 2-ft surface contour maps were used to measure the precise location of the coal outcrop. The actual discharge point was as little as 425-ft away from a thin-pillar section and 475-ft away from a pillar recovery section. The discharge point was found to be located slightly above the coal, indicating that the drainage path was at least partially through the strata directly above the Upper Kittanning coalbed. Hydraulic conductivities and potential discharge pathways are analyzed and discussed. The possible factors influencing the performance of the down-dip barriers at Solar No. 7 and Solar No. 10 are: a) the width of the barrier, b) the applied hydraulic gradient, c) the type of mining adjacent to the barrier, d) the overburden, e) the pattern and characteristics of fractures, and f) the quality of waters contained within the mine pools. This study, part of a larger effort funded by the Appalachian Research Initiative for Environmental Sciences (ARIES), is tasked to determine the key factors responsible for the successful design of down-dip coal barriers.

INTRODUCTION

Most legacy drift mines (mines operated prior to the 1970s) entered the coal reserve from outcrop and developed up-dip. This was the preferred mining method because water entering the mine would move away from the working faces, minimizing problems during production (Figure 1a). Unfortunately, this mining technique can negatively impact the quality of water moving through it. When mine water comes in contact with strata rich in iron,

aluminum, manganese and other metals, it can produce a range of pH and sulfate-rich drainage.

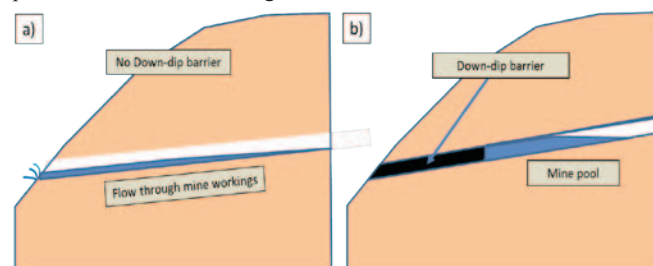


Figure 1. a) Typical scenarios for legacy underground coal mines where mining developed from the drift area and moved up-dip allowing water to move away from the working faces. b) Post-SMCRA, where mining established a down-dip coal barrier to contain the mine pool and eliminate drainage.

Section 516(b)(12) [30 USC 1266(b)(12)] of the Surface Mining Control and Reclamation Act (SMCRA) of 1977 changed this way of mining, requiring all new mines with acid- or iron-producing coal seams to locate openings to prevent a gravity discharge of water from the mine. This statute has successfully led to the development of mine pools that are contained by a down dip barrier that prevents mine waters from discharging to the surface (Figure 1b). In general, these mine pools contain reduced oxygen levels, which lowers the potential for pyrite and other minerals to produce waters rich in metals and sulfates. Currently, mine layout designs focus primarily on the size and hydraulic characteristics of the un-mined coalbed between the up-dip mine pool and areas where down-dip surface discharge is possible. This paper focuses on a detailed analysis of the mine layout, mining method, and geology of the Solar No. 7 and No.10 mines in Somerset County, Pennsylvania, where coal barriers have been both successful and ineffective in preventing discharges.

BACKGROUND

The Solar No.7 and Solar No. 10 room-and-pillar underground coal mines are located in northern Somerset County, Pennsylvania, near U.S. Route 30 (Figure 2). The Upper Kittanning coalbed in this area is 4 to 5-ft thick and overlain by shales and sandstones.

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The coalbed is underlain by claystones and the Johnstown limestone. Thick sandstone overlies the main entries near the Solar No. 7 portals (Figure 3). The Solar No. 7 mine was opened in 1975 by Lunar Mining, Inc. followed by Solar No. 10 in 1976 by Solar Fuel Company Inc. Both mines were purchased and sold to a number of different companies during their working lives. The Solar No. 7 mine was operated by Genesis Inc. at the time of its closing in 2002. Currently, Rosebud Mining Co. owns the mine property. The Solar No. 10 was owned by Solar Fuel Company Inc. at the time of its closing in 2002 and is the current owner of the mine property.

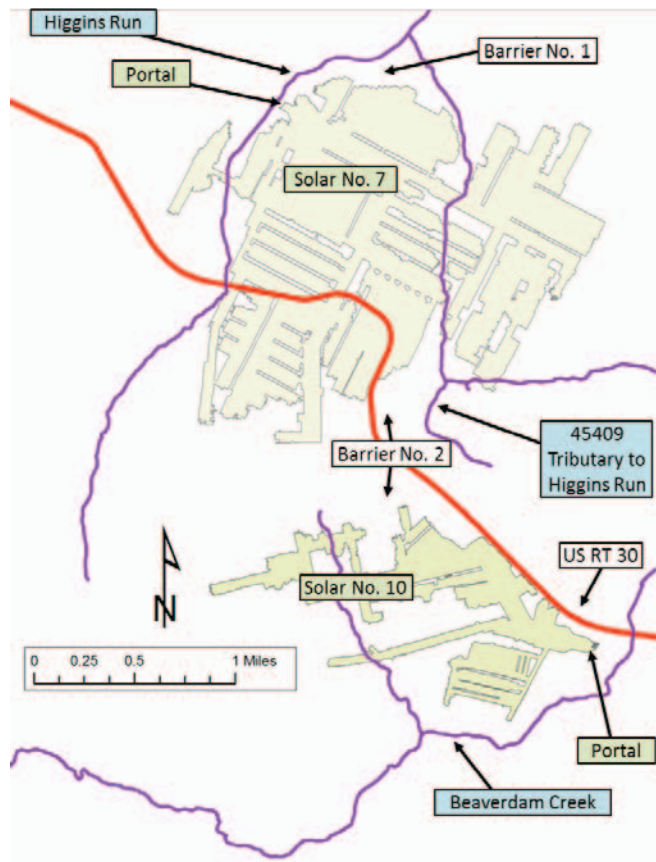


Figure 2. a) Locations of the Solar No. 7 and Solar No. 10 mines. The positions of the major streams, U.S. Route 30, mine portals, and down-dip barriers (No. 1 and No. 2) are also shown.

MINING METHOD

Solar No. 7 and Solar No. 10 are room-and-pillar mines. Development panels were sometimes subjected to either full or partial pillar recovery. Figure 4 shows the mine layout of the Solar No. 7 mine with the outlines of full and partial pillar recovery areas. Pillars within full extraction sections are recovered, leaving only remnants of coal acting as a temporary roof support. The immediate roof in these areas was allowed to fail, filling the sections with broken rock and forming gob. In areas where partial pillar recovery occurred, portions of the pillars were left in place. Lastly, a significant amount of the production sections in the southeastern portion of the mine were developed using thin pillars (Figure 4). Most of the pillars in these sections are less than 30-

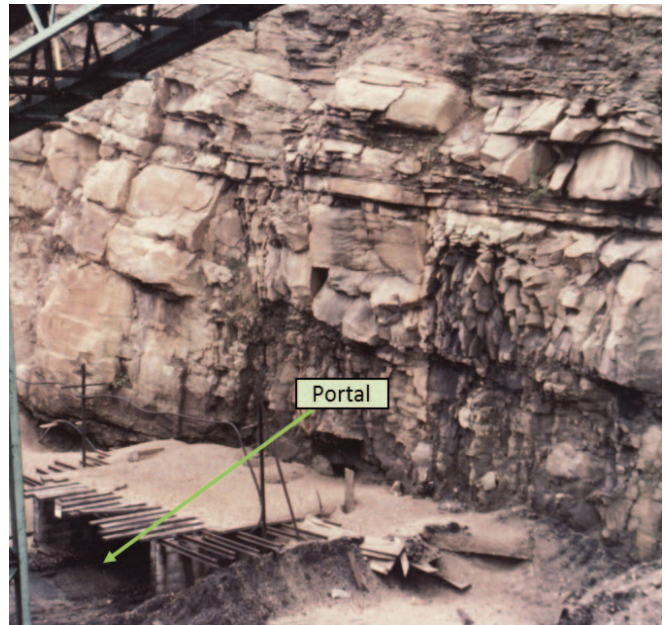


Figure 3. Photograph of the Solar No. 7 portal area showing the thick, massive sandstone roof rock.

ft wide and their long-term strength characteristics are unknown. Similar mining methods were used at the Solar No. 10 mine.

GEOLOGY

In a report by Iannacchione, et. al. (1981), the stratigraphy above the Upper Kittanning coalbed was analyzed using data from over 70 exploration core holes that covered the Solar No. 7 and Solar No. 10 mine properties. Three basic roof-rock types were identified: a thick sandstone (Freeport sandstone); a thick, dark-gray laminated shale; and a lateral transition zone between the sandstone and shale units. The transition zone is generally comprised of slickensided shale and a few thin sandstone stringers.

The Upper Kittanning coalbed at the Solar No. 7 and Solar No. 10 mines lies within the Somerset Syncline where the structure contour elevations are approximately 1,600-ft and gently rise 2° to 4° to the Boswell Dome in the northwest and the Negro Mountain Anticline in the southeast. A detailed structure contour map was constructed from the hundreds of survey contained on the mine's official maps (Figure 5). The locations of Barrier No. 1, along Higgins Run, and Barrier No. 2, protecting the Stoystown municipal water supply wells, are also shown.

ACCURATELY DETERMINING THE UPPER KITTANNING COALBED OUTCROP

A precise understanding of the structure contours for the Upper Kittanning coalbed is needed to accurately determine the coal outcrop along the Higgins Run down-dip barrier (Figure 6). The authors developed information on three different Upper Kittanning coalbed outcrop lines:

- Dashed line: Outcrop line drawn on the mine's 6-month mining map.

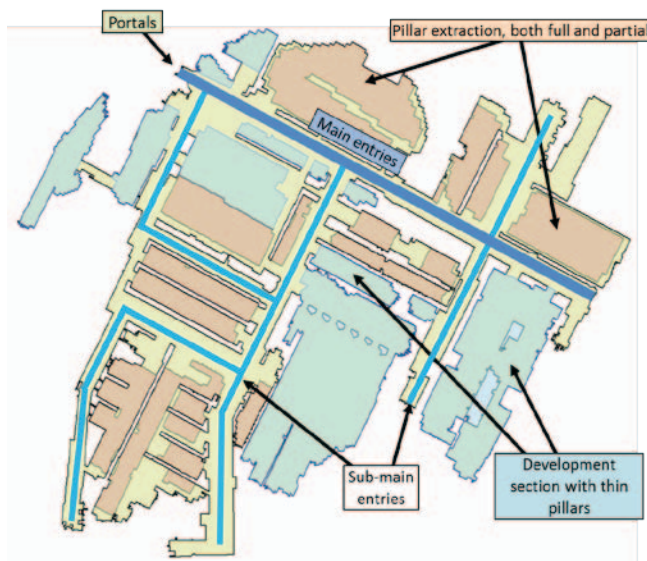


Figure 4. Mining methods used at the Solar No. 7 mine included multiple entry room-and-pillar main and sub-main entries with attached production panels.

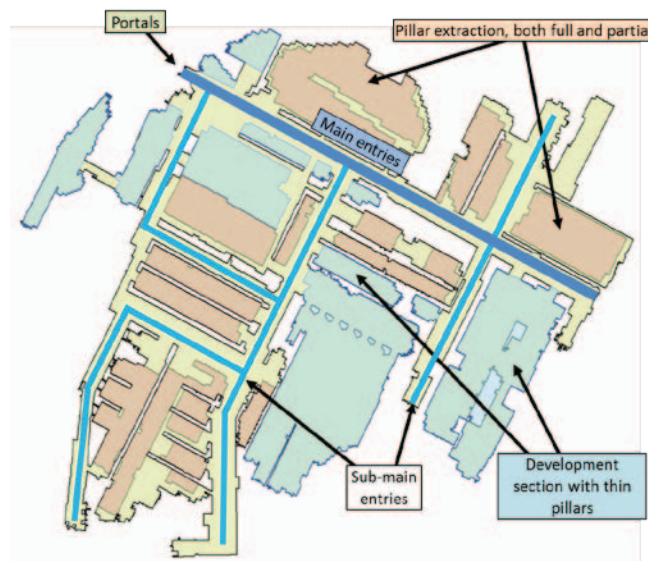


Figure 6. Three examples of the Upper Kittanning coalbed outcrop lines along Higgins Run, each dependent on a unique structure contour map.

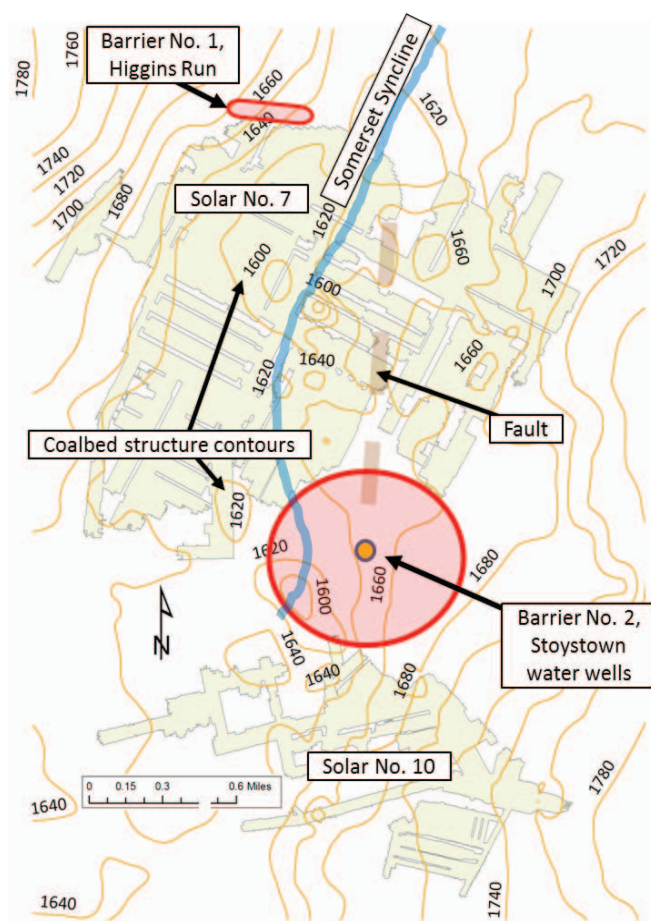


Figure 5. Upper Kittanning coalbed structure contour map constructed from numerous in-mine survey points. Also shown are the location of the down-dip barriers studied in this report. (No. 1 and No. 2).

- Dots: Outcrop line drawn from intersection of the structure contours from the Pennsylvania (PA) Geological Survey data with their equivalent surface lines (from the LIDAR 2-ft surface contour maps provided by the PA Geological Survey).
- Solid line: Outcrop line drawn from the intersection of the structure contours from the in-mine survey points with their equivalent surface lines (again from LIDAR 2-ft surface contours).

These three outcrop lines each present a slightly different view of the barrier size along Higgins Run. The most accurate outcrop line comes from the in-mine survey data and is used throughout the remainder of the report. The authors wanted to present this analysis to illustrate the complexity of establishing the position of the coal outcrop. This position can affect the barrier width estimates.

DISCHARGE ALONG HIGGINS RUN (BARRIER NO. 1)

In the winter of 2004, PA DEP agents first noted a ‘yellow’ discharge indicative of iron rich waters from the hillside along Higgins Run (Figure 7a). The surface discharge occurred when the mine pool reached its highest elevation of 1,681-ft. The discharge occurred in a relatively narrow zone ranging between 10 and 40-ft in width. The length of the down-dip barrier from the 1,680-ft contour (maximum pool elevation) to the eastern end of the projected Upper Kittanning coalbed outcrop is measured to be approximately 1,400-ft. The average width of the barrier was calculated from 16 line segments, and it averages 456-ft (Figure 7). The elevation at the discharge point is approximately 1,660-ft, producing a hydraulic head of approximately 20-ft. The estimated discharge rate was 30 to 75-gpm.

All of the reported discharges occurred within a narrow area (Figure 7b). It is interesting to note that a wetlands, approximately 100-yards to the west of the discharge point, did not show any signs of contamination from mine water (Figure 7c). At this point in time, the mine operator began to pump and treat water from

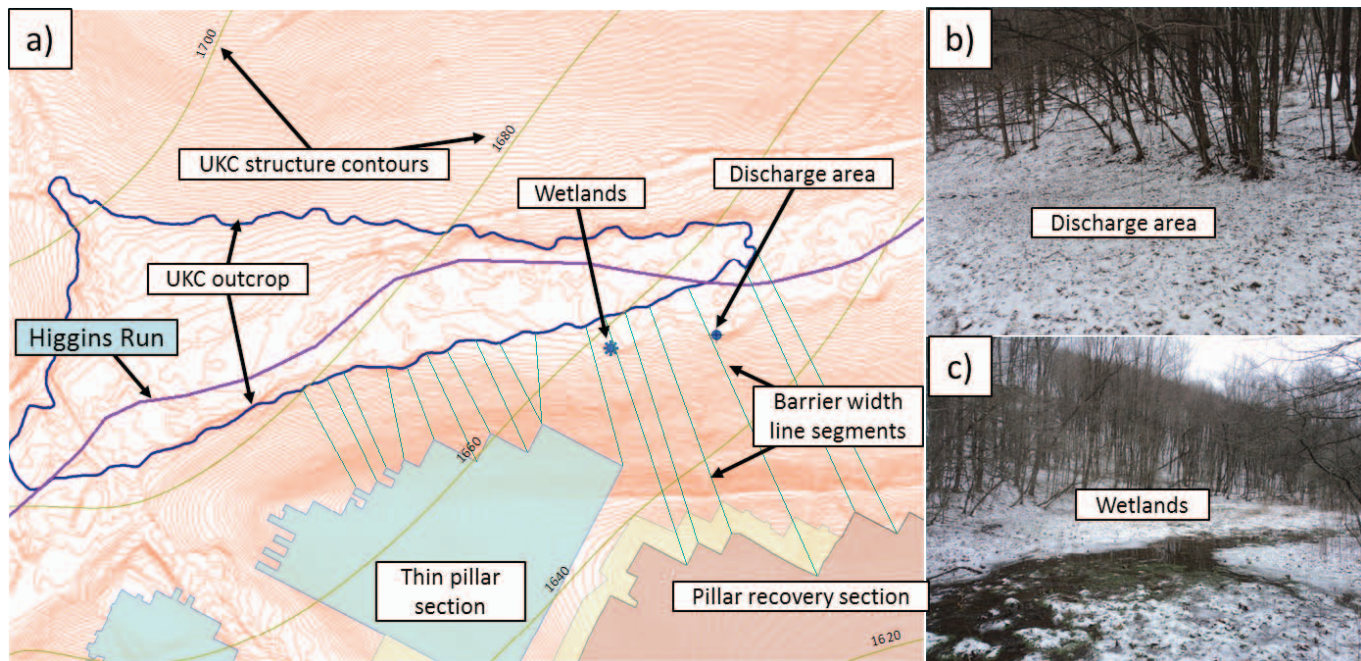


Figure 7. a) Plan view showing the mine layout, Upper Kittanning coalbed outcrop line and structure contours, Higgins Run, discharge area, and adjacent wetlands. b) Photograph of the discharge area. c) Photograph of the adjacent wetlands.

the mine pool. Over the next several months the mine pool was lowered to approximately 1,655-ft. Discharge conditions ceased at that elevation. On February 26, 2004 raw discharge from the Solar No. 7 borehole indicated the water pooled in the mine had become significantly impaired (Table 1).

Table 1. Samples of raw water from a borehole into the Solar No. 7 on February 26, 2004.

Parameter	Values
Flow, gpm	1,000
Iron, mg/l	190
Suspended Solids, mg/l	74
Manganese, mg/l	4.08
Aluminum, mg/l	0.46
Sulfates, mg/l	560
Specific Conductance, umho	1,753
Alkalinity, mg/l	59.3
Acidity, mg/l	138.3
Field pH (S.U.)	6.5
Laboratory pH (S.U.)	5.85

HYDRAULIC CONDUCTIVITY OF THE HIGGINS RUN BARRIER

As previously stated, down-dip coal barriers are important in preventing the discharge of mine water from an abandoned coal mine. The low hydraulic conductivity of the coal barrier restricts the flow between the mine pool and potential discharge points. The distance between these points can play a major role in the

effectiveness of the barrier to retard the mine pool water from reaching the surface through the coalbed. By modeling a coal barrier's hydraulic properties, accurate predictions regarding mine pool discharge are possible. Darcy's Law utilizes the principal properties of an isotropic system to provide an accurate "barrier K" value,

$$Q = K_h b L_b \left(\frac{dH}{w_b} \right) \rightarrow K_h = \frac{Q}{t L_b \left(\frac{dH}{w_b} \right)}$$

where

Q = flow out of the aquifer (ft³/d),

K_h = horizontal hydraulic conductivity (ft²/d),

w_b = width of barrier (ft),

L_b = length of the barrier (ft),

dH = difference in hydraulic head, discharge point to the top of the mine pool (ft), and

t = thickness of the barrier (ft).

The 'barrier K' value for coal in two bituminous coal mines of northern West Virginia was estimated using this method (McCoy et al., 2006). The McCoy study assumes that all of the water seeping through the barrier is homogeneous and is not flowing through any open and dominant fractures. An important assumption is that

Table 2. Range of properties used to estimate the hydraulic conductivity of the portion of the Higgins Run coal barrier that discharged during 2004.

Barrier name	Coal thickness (t), ft	Barrier width (w_b), ft	Barrier length (L_p), ft	Hydraulic head, ft	Flow rate, gpm	K_h , ft/d
Higgins Run	4.5	456	1,400	21	30–75	20–50

vertical infiltration through the overburden and into the coal barrier is negligible. This means that the actual K_h value would be lower due to mass balance. In studies by McCoy, et al (2006) and Islam, et al. (2013), barrier K values were estimated to range from 0.03 to 14-ft²/d. The wide range of values could be due to varying site conditions.

The authors used the data collected during this investigation to estimate the hydraulic conductivity for the Higgins Run barrier (Table 2). The ‘barrier K’ values attained in this study were calculated under the assumption that groundwater travels strictly through an unaltered coalbed (no fractures). The minimum and maximum values of hydraulic conductivity were found to be 20 and 50-ft/d. These values are much larger than those discussed in the McCoy study, indicating that a fracture flow path is possible.

FRACTURE FLOW ABOVE THE HIGGINS RUN BARRIER

An estimated discharge rate of 30 to 75-gpm occurred at one location (Figure 7) along the 1,400-ft barrier (Barrier No.1). Potential causes for the high hydraulic conductivity values could be due to the presence of fractures and bedding planes surrounding the coal barrier. These features are mainly geologic in origin, although full extraction mining can deform, or even induce, their occurrence. Iannacchione, et al. (1981) found that joints, coal cleat, and slickensided planes all had prominent clustering of N 17° W, N 72° W, N 48° E, N 62° E, and N 74° E. The latter three orientations are sub-parallel to the shortest distance from the yield pillar and full extraction pillar sections to the discharge point along Higgins Run.

Open fractures are more common in shallow rocks because the weight of the overburden in deeper rocks can potentially close the fractures (Pigati et al., 2006). The highest concentration of weathered fractures and bedding planes are typically found near the outcrop. The down-dip barrier along Higgins Run is very close to the surface, ranging in depth from zero to 100-ft. In this area, weathered fractures are possible and their orientation is likely to be sub-parallel to the width of the barrier.

The most general case to consider is a single weathered fracture extending vertically from the coalbed to the surface (range of 10 to 100-ft) and connecting the Solar No. 7 mine pool with the discharge point (Figure 7) along a N 60° E trendline. This case can be modeled using an approach developed by Romm (1966).

$$Q = \frac{\rho_w g b^2}{12\mu} (bh_f) \frac{\partial H}{\partial w_f}$$

Where:

Q = volumetric flow rate (ft³/d) [448.8 gpm = 1 ft³/s]

h_f = saturated fracture height (ft)

w_f = length of the fracture through the barrier (ft)

dH = difference in hydraulic head, discharge point to the top of the mine pool (ft)

b = fracture aperture (ft)

ρ_w = density of water (1.94-slugs/ft³)

g = gravitational constant (32.17-ft/s²)

μ = viscosity of water

$\frac{\partial H}{\partial w_f}$ = hydraulic gradient (We assume 21-ft/500-ft)

Under these conditions, the 30 to 75 gpm discharge along Higgins Run could be produced from a single water filled fracture with a height (h_f) of 21-ft and an estimated aperture (w_f) ranging from 0.08 to 0.11-in.

STOYSTOWN WATER SUPPLY BARRIER

The Solar No. 7 and Solar No. 10 mines are adjacent to the Stoystown municipal water supply. Stoystown, Pennsylvania, has four water wells producing directly from the Upper Kittanning coalbed and the underlying Johnstown limestone and Kittanning sandstone (Figure 8). These water wells were drilled in a valley and average approximately 200-ft in depth. The overburden above Barrier No. 2 ranges from slightly less than 200-ft to greater than 400-ft. Under these conditions, fractures should be relatively narrow (low aperture width). A potential source of this water reservoir is thought to be a fault that crosses the Solar No. 7 mine from north-to-south (Iannacchione, et al., 1981) and intersects the area containing the Stoystown water wells (Figure 5). This fault was observed underground to contain numerous open fractures and bedding planes (Figure 9).

The no-mining coal reserve had a radius of 1,800-ft and, in this location, constitutes a down-dip barrier (Barrier No. 2). Most of themine sections around the perimeter of this reserve are filled with water. The elevation of the Upper Kittanning coalbed at the water wells is approximately 1,645-ft. If the Solar No. 7 mine pool was at 1,655-ft, only a 10-ft hydraulic head would have been present. When the Solar No. 10 mine is flooded to its portal, an elevation of approximately 1,765-ft is achieved. This would produce a maximum hydraulic head of approximately 120-ft. It should be

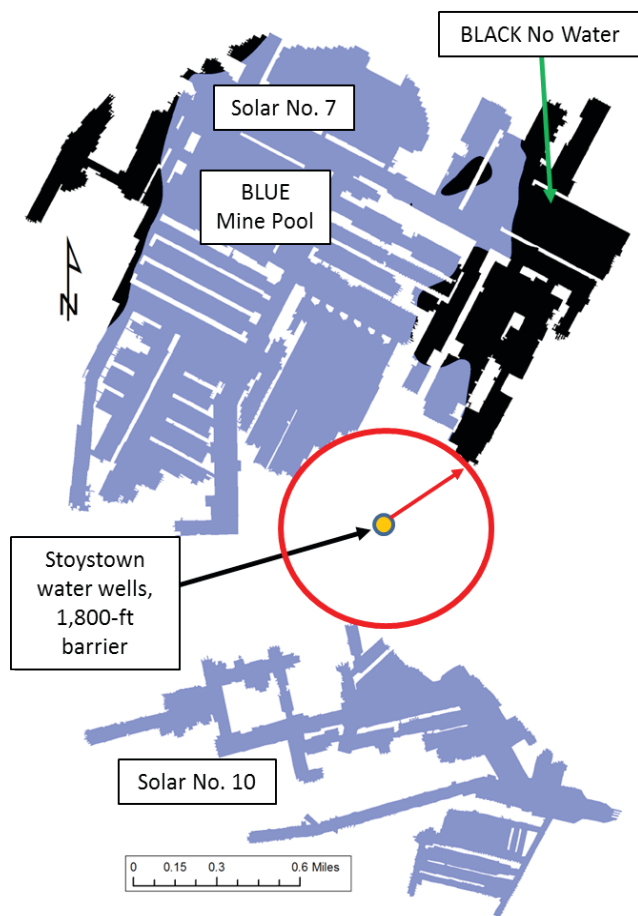


Figure 8. Location of the Stoystown municipal water wells with the 1,800-ft coal barrier. The water level for the Solar No. 7 mine pool is 1,655-ft and 1,765-ft for the Solar No. 10 mine pool. Also note the location of the fault that could be partially responsible for the productivity of these wells.



Figure 9. Photographs of faults observed within the Solar No. 7 mine. These faults are projected to intercept the Stoystown water supply wells.

noted that mining sections within the Solar No. 7 and Solar No. 10 mines near this Stoystown coal barrier were developed with a combination of thin pillars and full pillar extraction mining methods (Figure 4).

Mining by Solar No. 7 on the north side of Barrier No. 2 has seemingly done little to affect the quality of water located in the strata associated with Upper Kittanning coalbed and produced from the Stoystown water supply wells. If it had affected this

water supply, elevated levels of iron, manganese, sulfates and other products found in the Solar No. 7 mine pool would be present. One reason for this could be the relatively low hydraulic head (~10-ft) produced when the mine pool is maintained at 1,655-ft.

The influence of mining by Solar No. 10 on the south side of Barrier No. 2 on the Stoystown water supply is more difficult to determine. It is more likely that Solar No. 10 mine pool water would impact the water supply since a 120-ft hydraulic head is being applied to this side of the barrier. There seems to be two possibilities for non-impairment of the Stoystown water wells through Barrier No. 2 from the Solar No. 10 mine:

1. The down-dip barrier is large enough that it retarded the amount of flow due to resistance of flow in the coal (Darcy's law), or
2. Flow through the barrier is occurring under the 120-ft hydraulic head, but the water quality in the Solar No. 10 mine pool is not impaired.

This second point was explored in more detail by the authors. The Solar No. 10 discharge is generally low (0 to 60 gpm) and contains low concentrations of iron, manganese, and sulfates. This discharge is located at the reclaimed portal and is most likely caused by a leaking mine seal or water flow through the strata surrounding the seals. Table 3 shows the parameters of raw water collected from the Solar No. 10 mine. From these samples, it is indicative that the water is net alkaline and contains significantly less iron, manganese, and sulfate than the Solar No. 7 mine pool. These data suggest that it is not possible to definitively determine the effectiveness of Barrier No. 2.

CONCLUSION

The discharge at the Solar No. 7 mine occurred in a relatively narrow zone within a down-dip barrier along Higgins Run (Barrier No.1). The average width of the barrier is 456-ft and the length is 1,400-ft. The hydraulic head on this barrier at the maximum mine pool was calculated to be approximately 21-ft. The discharge to Higgins Run occurred in an area where pillar recovery and thin pillars were used. The discharge was relatively low, ranging from 30 to 75-gpm. The hydraulic conductivity of the Higgins Run coal barrier is calculated to be between 20 and 50-ft/d. This relatively high value may be influenced by intense fracturing, some with large apertures due to weathering near the surface outcrop or adjacent to the pillar recovery area. Using a method derived from Romm (1966), this same discharge could be produced from a single water filled fracture with a height (hf) of 21-ft and an estimated aperture (w_f) ranging from 0.08 to 0.11-in. Both of these conditions are possible. However, at this point, neither can be proven. The relevant point is that each condition would require a different design consideration. Does the barrier need to be wider or do weathered fractures need to be identified and sealed to prevent a discharge under these specific conditions?

In the second case study, an 1,800-ft no-mining coal barrier (Barrier No. 2) may have successfully protected four highly productive municipal water wells. The hydraulic head on this barrier ranged from 10-ft on the north to as much as 120-ft on the south. The mine pool on the north side of the barrier contains impaired water that is high in iron, manganese, and sulfates; while the mine pool on the south side consist of non-impaired water. It is possible that the hydraulic conductivity of the coal barrier is

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Table 3. Samples of raw water from the CWPD monitoring point at the Solar No. 10 mine from March 2011 to November of 2012.

Parameter	Monitoring dates							
Date sampled	29-Mar-11	10-May-11	16-Aug-11	24-Oct-11	21-Feb-12	13-Jun-12	28-Sep-12	7-Nov-12
Flow, gpm	0.072	0.072	0.072	60	50	50	50	50
Iron, mg/l	2.95	3.19	9.41	4.83	5.5	5.3	10.5	0.75
Suspended Solids, mg/l	5	<5	21	9	15	6	15	<5
Manganese, mg/l	0.57	0.59	0.68	0.55	0.48	0.64	0.6	0.1
Aluminum, mg/l	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	<0.1	0.1
Sulfates, mg/l	94	99	98	96	79	80	77	56
Specific Conductance, umho	698	633	647	657	621	682	622	289
Alkalinity, mg/l	153	147	164	163	160	162	159	43
Acidity, mg/l	-135	-135	-156	-151	-142	-132	-149	-33
Field pH (S.U.)	7.5	8	7.5	7.5	7	7.5	7.5	7.5
Laboratory pH (S.U.)	7.58	8.14	7.25	7.45	7.19	7.49	7.44	7.71

low enough to prevent measureable water infiltration. It is equally possible that flow through the barrier has occurred but has not contaminated the Stoystown water supply because the Solar No. 10 mine pool is relatively low in iron, manganese, and sulfates.

Down-dip coal barriers need to be designed to protect surface and groundwater resources (Barriers No. 1 and No. 2) from mine pool discharges. The ability of coal barriers to resist flow caused by hydraulic gradients from mine pools can be greatly affected by the potential presence of fractures close to outcrop. It is interesting to note that Kendorski, et al. (2007) stated that most empirical design methods for barrier width are a function of depth; yet, current barrier conductivity empirical design methods use only hydraulic head and coalbed thickness. In the future, when conditions are similar to those encountered in this study, the design should be as much focused on the potential flow pathways caused by fractures adjacent to the coalbed as the thickness of the coal barrier. Lastly, it may be advisable to restrict pillar recovery adjacent to the down-dip barrier.

ACKNOWLEDGEMENT

This study was sponsored by the Appalachian Research Initiative for Environmental Science (ARIES). ARIES is an industrial affiliates program at Virginia Tech, supported by members that include companies in the energy sector. The research under ARIES is conducted by independent researchers in accordance with the policies on scientific integrity of their institutions. The views, opinions and recommendations expressed herein are solely those of the authors and do not imply any endorsement by ARIES employees, other ARIES-affiliated researchers or industrial members. Information about ARIES can be found at <http://www.energy.vt.edu/ARIES>.

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The Interaction of Unique Geology and Retreat Room-and-Pillar Mining on Overburden Barrier Performance at the Grove No. 1 Mine

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ABSTRACT

This study, funded by the Appalachian Research Initiative for Environmental Science (ARIES), examines how mine layout, mining method, and geology interacted to allow water from the Grove No.1 mine pool to discharge into local surface waters. Federal and State regulations prohibit the discharge of acid or iron-producing waters from underground coal mines. Mining companies design barriers around their operations to prevent mine waters from discharging to the surface. The Grove No.1 represents one mine where these designs proved inadequate. Unique geologic conditions at the Grove No.1 mine pool, combined with retreat room-and-pillar mining, contributed to a discharge that required passive water treatment.

INTRODUCTION

The Grove No. 1 Mine (MSHA ID #3602398) was opened in March of 1969 and closed in 2001. There were multiple owners including GM&W Coal Co., Carbolith Extraction Inc., Grove Resources, and Lion Mining Co. The mine is located in Somerset County, Pennsylvania, not far from US Routes 30 and 219 (Figure 1). Grove No.1 mine underlies two watersheds, South Fork Bens Creek and Roaring Run. Both are part of the larger Conemaugh watershed. The Pennsylvania Department of Environmental Protection (PA DEP) has designated South Fork Bens Creek as an exceptional value stream where native brook trout are known to live. The mine was designed as a drift room-and-pillar operation and entered the reserve through portals near the Upper Kittanning coalbed outcrop.

The Upper Kittanning coalbed at the Grove No.1 mine has overburdens ranging from zero at the portals to over 1,000-ft. In this area, the coalbed strikes N70°E and dips an average 15-degrees to the southeast (vertical change of 1,000-ft over 1 mile) as it plunges from highs along the limbs of the Laurel Hill anticline into the bottom of the Johnstown syncline. Initially, mining occurred within the Roaring Run watershed (AREA 1, Figure 1). Sometime in the early 80s, mining began to enter the South Fork Bens Creek watershed (AREA 2 and AREA 3, Figure 1). During the entire active mining period, no discharges were reported into the South Fork Bens Creek watershed.

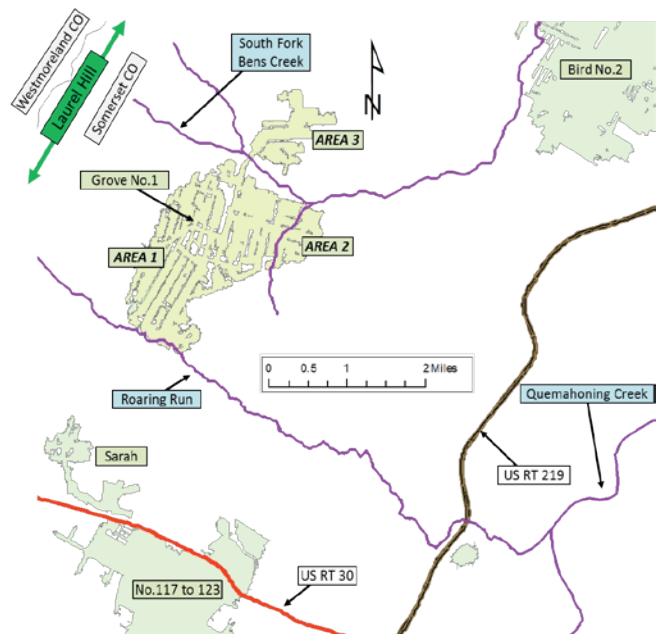


Figure 1. Location of the Grove No.1 adjacent to Laurel Hill Anticline in Somerset County, Pennsylvania and underlying South Fork Bens Creek and Roaring Run. Also shown are the mine portals and the general sequence of mining [numbered areas; AREA 1 being the first, AREA 3 being the last to be mined].

BACKGROUND TO THE DISCHARGE

The Grove No.1 mine closed in 2001. Shortly after, the water level of the mine pool reached an elevation of 1,450-ft (Blue area, Figure 2a). The pool level was controlled by a treatment facility operated by Lion Mining along South Fork Bens Creek. The mine petitioned the PA DEP to allow the mine pool to reach the 1,700-ft level. The petition was approved, and, by June 21, 2001, the pool rose to a reported elevation of 1,700-ft (Purple area, Figure 2a). This elevation was later to be found in error. As shown in Figure 2, the pool elevation is controlled by the Upper Kittanning structure contours (elevation above sea level). In July of 2001, a discharge was reported along South Fork Bens Creek (Figure 2a). A

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photograph was taken during the ensuing investigation by Federal and State officials (Figure 2b) capturing the iron-rich discharge upwelling in the bottom of South Fork Bens Creek.

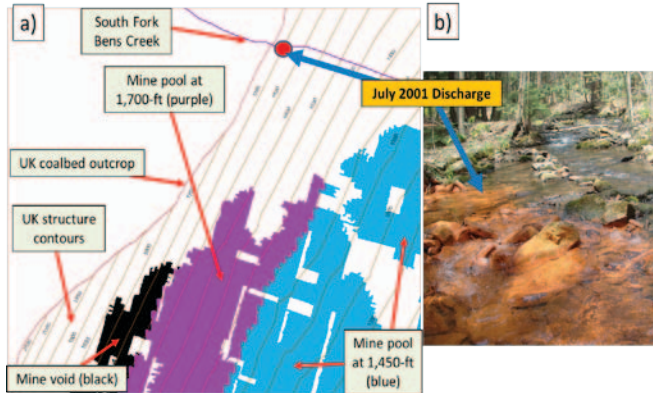


Figure 2. a) Two important mine pool elevations are shown during 2001 (i.e., 1,450-ft in BLUE and 1,700-ft in PURPLE). Also shown is the portion of the mine not filled with water (BLACK). These pool elevations are controlled by the dip of the Upper Kittanning Coalbed (see UK structure contours). b) Photograph of the discharge (June 2001).

The South Fork Bens Creek discharge was not expected for several reasons:

- The Upper Kittanning coalbed outcrop was at 1,730-ft or approximately 30-ft above the elevation of the mine pool.
- The distance from the closest mine workings to the upper reaches of South Fork Bens Creek was over 2,000-ft. (The authors don't know of any other discharge that traveled this far along the strike of the coalbed.)

The PA DEP conducted an investigation to explain and identify the cause of the South Fork Bens Creek mine drainage discharge (Matyus, Kernic, and Korich, 2002). In this report, the authors found that the mine pool had risen to a maximum elevation of 1,806-ft, producing a hydraulic head difference of an average 78-ft between the top of the pool and the seep elevations (multiple seeps were identified in the PA DEP investigation). The report concluded the source of the South Fork Bens Creek discharge was the Grove No.1 mine pool.

The University of Pittsburgh collected information from multiple sources to help analyze the unique geology and complicated mine layouts at the Grove No.1 Mine. This report will help to document how geology and mining layouts can interact to adversely affect the performance of barriers meant to prevent coal mine discharges.

UNIQUE GEOLOGY

The structure of the Upper Kittanning coalbed in the area of the Grove No.1 mine is, indeed, unique. The Laurel Hill Anticline has produced dramatic dips to rock formations along most of its 80-mile length (i.e., West Virginia border in the south to the middle of Indiana County in the north). In many locations, the flank of the Laurel Hill anticline dips at an average rate of 4° (Flint, 1965). However, in some areas, the flank of the Laurel Hill anticline can

dip upwards of 60°, especially when faulting is present (Puglio and Iannacchione, 1979). The Upper Kittanning coalbed structure contours for the Grove No.1 mine were collected from official maps submitted by Lion Mining, Inc. and digitized within ArcGIS. A 3-D perspective of the surface topography and structure contours of the Upper Kittanning coalbed are shown in Figure 3. The dips of the Upper Kittanning coalbed within the Grove No.1 mine range from 5 to 30°.

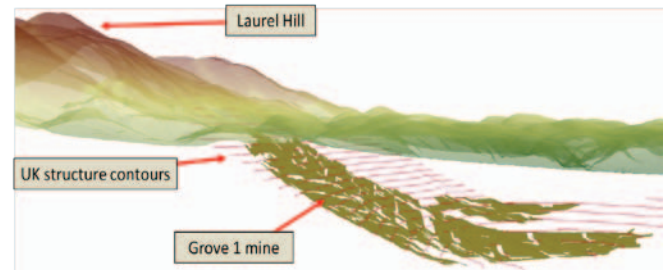


Figure 3. 3-D view of surface topography and Upper Kittanning coalbed structure contours in the vicinity of the Grove No. 1 mine.

Formations that are draped over significant geologic features, such as the Laurel Hill Anticline, will often experience lateral adjustments producing prominent bedding features. Movement is most likely to occur within the softer units, such as the coalbeds, and can be observed as bedding plane faults. One of the authors had an opportunity to perform geological mapping within the Grove No.1 mine in the early to mid-1980s. The results from that mapping exercise were never published, and the files were destroyed when the US Bureau of Mines closed in 1995. Figure 4 is a cartoon that depicts the occurrence of slickensided bedding plane faults and the offsets observed within the clastic dykes that were penetrated by the faults. The bedding plane faults, along which the offsets were observed, were highly slickensided and occurred within the coalbed and the overlying shale members. It is possible that they also occurred within the underlying rock units. These structural features were found throughout this portion of the mine listed as AREA 1 in Figure 1.

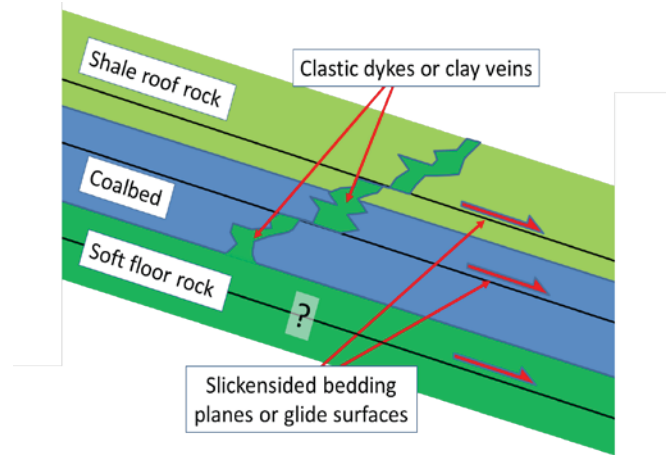


Figure 4. Cartoon of clastic dyke offset along prominent slickensided bedding plane surfaces.

ROOM-AND-PILLAR MINE LAYOUT WITH PILLAR RECOVERY

The significant dips of the Upper Kittanning coalbed affected the room-and-pillar mining method used at Grove No.1 (Figure 5a). The mine had three different mine layout styles:

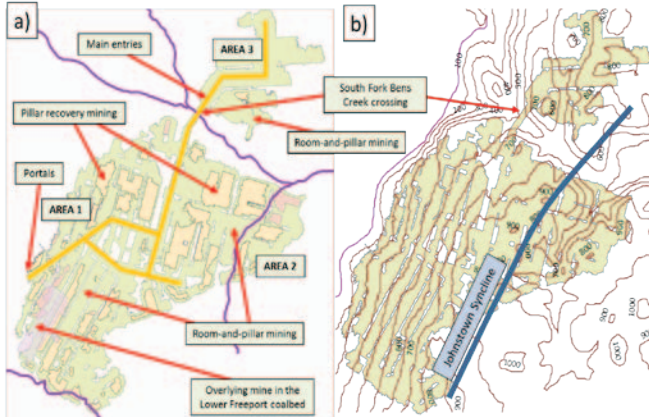


Figure 5. a) Characteristics of the Grove No.1 mine layout, and b) Overburden map.

- **AREA 1:** Main entries entered the reserve at the portals and quickly drove to the bottom of the Johnstown Syncline. Significant dips require transporting workers and supplies along a roof track-mounted monorail. Track haulage was not feasible in the main entry of AREA 1. Production sections were designed to optimize mine developments with level headings. Overburden ranged from approximately 100-ft to almost 900-ft.
- **AREA 2:** The main entries entered AREA 2, near the bottom of the Johnstown Syncline, where dips averaged 5°. Here, main entry headings are oriented along the coalbed strike, allowing for track haulage. The developments were mainly south of the main entries. The most striking condition in this area is the significant overburden relative to other areas ranging from a minimum of 600-ft to slightly more than 1,000-ft. Most production panels were eventually subjected to some form of pillar recovery (full or partial). Some full extraction panels lie under South Fork Bens Creek.
- **AREA 3:** The main entries entered AREA 3 across South Fork Bens Creek to reach additional reserves on the northeast side. The mining method was dominated by room-and-pillar developments. Dips were, on average 5°, and overburdens ranged from approximately 500-ft under South Fork Bens Creek to more than 800-ft under the hill tops. Crossing South Fork Bens Creek was a concern to the PA DEP. They required the mine to minimize coal extract under South Fork Bens Creek. A detailed mining plan was submitted and approved by the PA DEP.

As mentioned previously, the US Bureau of Mines had conducted an investigation at this site. The investigation was focused on determining the cause of the difficult mining conditions encountered while mining in areas of high dip (AREA 1). Of particular concern were the unstable roof conditions in both development and retreating production panels. There was also a small mine in the Upper Freeport coalbed overlying Grove No.1

in this location (Figure 5a). Figure 6 is a cartoon submitted to help explain the reasons for the difficult ground conditions. These conditions were most unstable in the development sections down-dip from an adjacent up-dip pillar recovery section. It is likely that the significant dip of the Upper Kittanning coalbeds (Figure 3) and the presence of laterally persistent slickensided bedding plane faults (Figure 4) facilitated the transfer of overburden load, down-dip from the retreat mining section to the adjacent development section (Figure 6). The eccentric nature of the load applied to the roof, floor, and pillars of the development section most likely overcame the design strength of the pillars and intrinsic roof support.

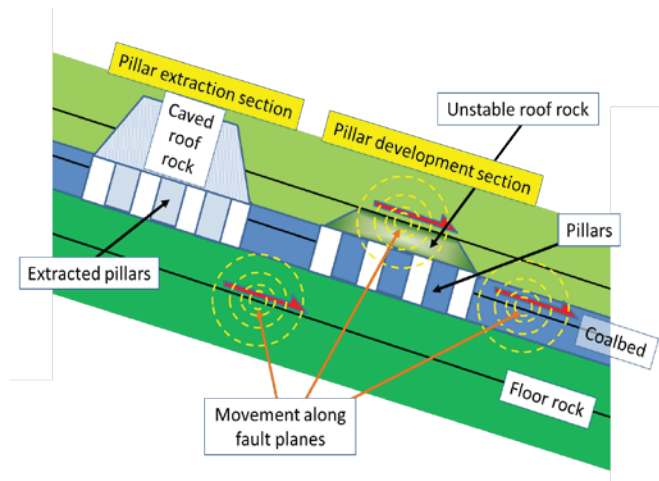


Figure 6. Cartoon of the geologic and mining layouts in adjacent development sections as the down-dip section is advanced while the up-dip section is retreated.

SEISMIC ACTIVITY

Grove No.1 mine has had a history of “alleged” seismicity. The first mention of this is on February 3, 1982, when a magnitude 2.6 event was located near Jennerstown, PA, by the US Geological Survey (Scharnberger, 2003). Jennerstown is only a few miles from the Grove No.1 mine, and this area, according to the Scharnberger report, has not previously registered an earthquake.

In October and November of 1988, Wu, Kravitz, and Hoch (1988) reported a series of “tremors and vibrations” that rocked the surface above the Grove No.1 mine. The Mine Safety and Health Administration (MSHA) conducted an investigation of the reported tremors and vibrations and found that they had occurred 1.5 months before the writing of the memorandum. These events were thought to emanate from the No. 15 Left and between the No. 11 and 12 Left sections. Both of these areas were in the process of recovering pillars under relatively steep ridges (Figure 7). Also, the “safety barrier” (barrier pillar between two pillar recovery sections) had been removed during pillar recovery, something that is different from regular mining procedures.

Other events include the following:

- In 1988, the Forwardstown Area Concerned Citizens Coalition reported more than 150 tremors, quakes, and vibrations in the area of the mine.

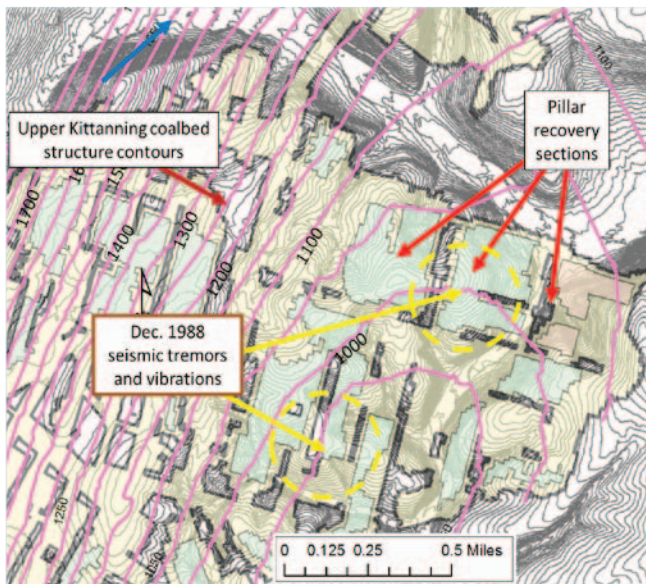


Figure 7. Grove No.1 mine layout showing the pillar recovery areas, Upper Kittanning coalbed structure contours, surface discharge zone, and December 1988 seismic tremors and vibrations.

- On May 14, 1991, an article in the Tribune Democrat stated “tremors felt...stopped after inspection (partial mining).”
- On January 4, 1994, Professor Shelton Alexander at Pennsylvania State University measured magnitudes of 3.1 and 3.2 emanating from the vicinity of the mine.
- On June 1, 1995, (Letter, 1995), the Forwardstown Area Concerned Citizens Coalition reported 44 tremors, quakes, and vibrations since January 4, 1994 (an 18-month period).

While several mechanisms are possible, it seems evident that full extraction mining reactivated movement along preexisting bedding plane faults resulting in significant seismic activity.

It seems the most telling information came from the Wu, Kravitz, and Hoch (1988) report. A direct connection was made between seismicity and pillar recovery activities (Figure 7). While microseismic events are common when full extraction mining takes place, it is uncommon for these events to fall into the range of earthquakes (i.e., Richter magnitudes greater than 2). Therefore, it seems reasonable to conclude that the significant Upper Kittanning coalbed dips, the presence of slickensided bedding plane faults, and the use of full extraction mining, all combined to produce significant damage to the strata adjacent to the Grove No.1 mine.

CALCULATING HYDRAULIC CONDUCTIVITY

Hydraulic conductivity of the rock media at the Grove No. 1 site was calculated using a water balance method along with Darcy’s Law. Water balance method is used to first determine infiltration into the mine given pumping rates and mine pool elevation data. The method simply states that inflow to the mine minus outflow from the mine must equal a change in the mine pool. The given data allows an average infiltration into the mine to be calculated (0.327-ft/day/ft² of mine area). Assuming this value is constant, infiltration and mine pool elevation data during the time of the discharge is used to determine the discharge flow

rate. Hydraulic conductivity can then be calculated using Darcy’s Law. Using this method, hydraulic conductivities are calculated to be 6.7 – 18.5-ft/day based on a 30-ft head difference and 2,100 – 3,200-ft length of travel. These values are considerably higher than hydraulic conductivities for coal found in the literature. The values do, however, correspond to the hydraulic conductivity for fractured rock.

PROBABLE CAUSE OF THE SOUTH FORK BENS CREEK DISCHARGE

The PA DEP investigation of the Ben Creek discharge identified seven seeps (Matyus et al., 2002). Chemical analysis from the seeps and the mine pool (BAMR Hole 2) showed similar water chemistry. The elevations for the seven seeps, two piezometers, and two drill holes are listed in Table 1. On May 27, 2002, the PA DEP measured a static mine pool water level of 1,805.9-ft. This would put the mine pool 74 to 83-ft above the seeps.

Table 1. Information on the seeps, piezometers, and drillholes near South Fork Bens Creek.

ID	Surface elevation, ft	Comment
Seep 1	1734.1	
Seep 2	1735	
Seep 3	1735.1	
Seep 4	1731.6	
Seep 5	1723.2	
Seep 6	1723.4	
Seep 7	1723.8	
PZ 1	1710.6	TOC 1711.3, deep aquifers, 1682
PZ 2	1709.8	TOC 1710.6, shallow fractured sandstone aquifer, 1703.1
Degas Hole Mains A	1891.6	TOC 1894.9
BAMR Hole 2	2009.3	Water samples confirmed association with seep

The most logical explanation for the discharge path between the Grove No.1 mine pool and seeps is that the water moved along the strike of the Upper Kittanning coalbed and associated roof and floor strata. This would produce water flow travel distances of 2,300 to 3,200-ft under a hydraulic head averaging 78-ft (Figure 8). Groundwater flow in this area is most likely dominated by open joints, fractures, and bedding planes. This fracture permeability is especially dominant in and near valleys such as South Fork Bens Creek where stress relief fractures are present (Johnson, 1980). The two processes of highly fractured rock close to the surface and 78-ft of hydraulic head probably provided the conduit and the driving force to allow the flow of mine pool water to South Fork Bens Creek. However, it is also possible that significant seismicity at the mine provided sufficient energy to enhance the fracture network hydraulic conductivity. The seismic energy often results

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in displacements along pre-existing fractures and bedding planes and can also result in greater water transmissivity.

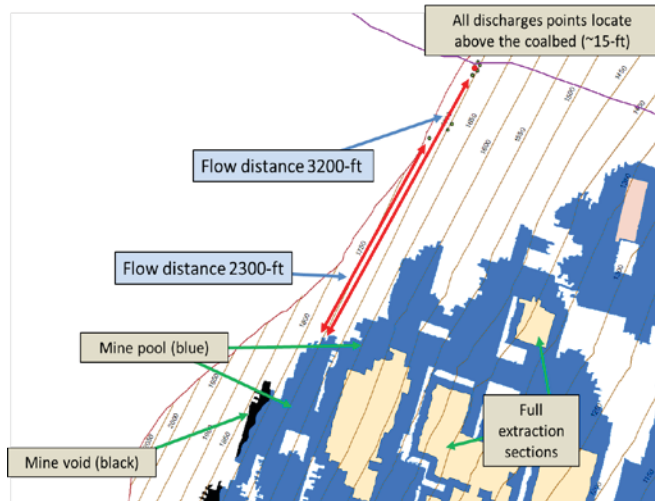


Figure 8. Location of the mine pool at its maximum recorded height of 1805.9-ft and the projected path of water discharge from the mine to the seeps along South Fork Bens Creek.

Seismicity at the Grove No.1 mine is associated with displacements along pre-existing rock bedding planes and both shear and tensile failures within rock layer due to stress-redistribution and stress-relief fracturing (Wyrick and Borchers, 1981). Since natural and mining-induced fractures are common at this mine, it is appropriate to assume that the discharge is dominated by fracture flow. Using the cubic law developed by Romm, widths and apertures of fractures can be calculated from the discharge rate (Romm, 1966). The cubic law is expressed as

$$Q = [(\rho_w * g * b^2) / (12 * \mu)] * [(b * w) * (\delta h / \delta L)]$$

Where:

Q = discharge flow, ft³/s

ρ_w = density of water (1.94-slugs/ft³)

g = gravitational acceleration (32.17-ft/s²)

μ = viscosity (2.034x10⁻⁵-lbs/ft²)

b = aperture opening, ft

w = fracture width perpendicular to the flow direction, ft

$\delta h / \delta L$ = hydraulic gradient (0.0143 and 9.38x10⁻³)

The values for fracture width and aperture can be compared to determine the feasibility of mine discharge contributions. Table 2 gives fracture widths and apertures that can accommodate the mine discharge (60-gpm). By observation, a feasible fracture width and corresponding aperture would be a 12-in fracture with a 10.1-mm aperture. Other combinations are also possible. Given that fractures of this size are common near the surface and are capable of conveying the discharge volume, it can be concluded that

fracture flow can be a major contributor in the cause of the Grove No.1 discharge.

Table 2. Fracture widths and apertures using Romm Theory.

Fracture Width, in. (mm)	Fracture Aperture, in. (mm)
0.012 (0.3048)	3.98 (101)
0.12 (3.048)	1.85 (47.0)
1.2 (30.48)	0.858 (21.8)
3 (76.2)	0.632 (16.1)
6 (152.4)	0.502 (12.8)
12 (304.8)	0.398 (10.1)
24 (609.6)	0.316 (8.03)
36 (914.4)	0.276 (7.02)
48 (1219.2)	0.251 (6.38)
120 (3048)	0.185 (4.70)
1200 (30480)	0.0858 (2.18)

SUMMARY

The Grove No.1 mine was designed to prevent pooled waters from directly discharging to the surface. All of the mine workings were located below drainage, so the only threat for unwanted discharge was through the overlying strata. After the mine closed and flooded, a major discharge of iron-containing water occurred along South Fork Bens Creek. This was not expected since the only mining that occurred under South Fork Bens Creek was a main entry system used to access reserves to the northeast. These mains, especially under South Fork Bens Creek, were adequately designed and should not have damaged the overlying strata. Therefore, it is highly unlikely that mine waters could flow upward over 300-ft vertically to the surface.

At the time of mining, strata conditions within the Grove No.1 mine were considered to be challenging, especially when development sections were mined down-dip of a pillar recovery panel. The previously mentioned bedding plane slips undoubtedly aided in the down-dip movement of strata that was observed in several of these development sections. It is likely that a portion of the strata within and above the Upper Kittanning coalbed was damaged during both pillar recovery and associated seismic events, allowing water to flow horizontally several thousand feet from the edge of mining to South Fork Bens Creek. This mechanism is important to understand if adequate mine layouts are to be designed in the future that will provide a low risk of mine water discharge to surface waters.

CONCLUSION

The Key Factors responsible for the performance of these barriers was found to be the following:

- Geology: Dips averaging 15 degrees and the associated structures, such as bedding plane slips, were present within the Grove No.1 mine property prior to mining.
- Mine layout: The mine was designed to drive main entries down-dip towards the Johnstown syncline. Development

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sections were driven along the strike of the formation on both sides of the main entry. Once a development section was driven to its full length, the panel pillars were recovered.

- Strata control: The combination of high dips, bedding plane slips, and pillar recovery mining methods produced challenging strata control conditions that resulted in tremors and vibrations at the surface. These tremors were large enough to generate significant horizontal and vertical fractures in the overlying strata.
- Mine flooding: When the mine began to flood and fill with water, the mining induced failures provided pathways for the impaired waters to reach Ben's creek.
- Strata barrier design issues: In this case, the strata barrier designed to protect Ben's creek proved inadequate, mainly due to unique rock fracture conditions and bedding plane slippage. When these conditions are encountered in the future, additional thicknesses of strata barriers or restricting pillar recovery mining are recommended.

ACKNOWLEDGEMENT

This study was sponsored by the Appalachian Research Initiative for Environmental Science (ARIES). ARIES is an industrial affiliates program at Virginia Tech, supported by members that include companies in the energy sector. The research under ARIES is conducted by independent researchers in accordance with the policies on scientific integrity of their institutions. The views, opinions and recommendations expressed herein are solely those of the authors and do not imply any endorsement by ARIES employees, other ARIES-affiliated researchers or industrial members. Information about ARIES can be found at <http://www.energy.vt.edu/ARIES>.

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Stability Analyses and Remediation of Two Mined-Out Coal Seams Adjacent to a Building

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ABSTRACT

As part of the development of a shopping center in West Virginia, a significant bench had to be cut into a mountainside that was over 700 ft. high. As a result, a steeply stepped, over 400-ft.-high wall was created. The high wall exposed old abandoned room-and-pillar coal workings at the toe of the cut. Because of exposure to the elements, the mine openings began to severely deteriorate with significant potential of undermining of the rock cut face and causing a large rock fall(s). Exacerbating this wall problem was a set of fairly planar sub-parallel and sub-vertical joints creating, in essence, thin rock slabs. Also present were stress relief fractures that were observed in the exposed pillars.

Given the proximity of the project building to this high rock face, mitigation of the rock fall potential was deemed necessary. In order to prevent continued damage and deterioration of these exposed mine entries, the rubblized openings were filled with bulkhead grout and pressure injected with grout. The total amount of grout installed in these openings was about 3,945 CY. Also, a rock drape was attached to the entire face to mitigate small rock falls from natural causes.

There also existed another level of mining below the ground surface and under an existing building at a depth of about 160 ft. Under a portion of the building, the pillars in these old works were second mined, and the building seemed to have been already affected by limited subsidence above this area. Also, calculated pillar safety factors in this high extraction section of the mine were too low. Consequently, these old works were stabilized by saturation grouting. An indication of overall roof distress was the significant fracturing encountered in the bedrock overburden during the drilling of the injection holes and the volume of grout placed in these holes. The total grout installed in the high extraction section of the lower mine was about 6,660 CY.

This paper describes the investigation and the grouting techniques employed at the rock cut face and under the existing building to mitigate continued damage to avoid future subsidence problems at the face and the building premises.

INTRODUCTION

A subsidence engineering investigation was undertaken at an existing building in West Virginia. The concern was related to potential collapse of the abandoned room and pillar mining beneath the building and adjacent rock cut immediately to the east side of the building (see Figure 1). There had been two mined seams: Mine No. 15 (upper seam) and Mine No. 15 Jr. (lower seam) at the site. Both mines were operated in the 1940s, and later in 1967, the lower mine was reopened. There are no production records for either mine, and it is unknown for how long the mining operation lasted. A subsurface investigation was undertaken where it was discovered that the old works closest to the surface were apparently stripped out below the existing building during the site development. The deeper mine was encountered at a depth of about 160 ft. The borings, which intersected the mine, were found to be, for the most part, collapsed in places and open in others. Mine stability analyses indicated that, below the northern half of the site, very poor pillar support conditions were left, but the southern half of the site had very low risk of collapse. Hence, the focus was on developing an efficient and economical mitigation plan for the areas that had very poor pillar support. Based on the engineering investigation and the stability analyses, mine grouting was considered the best remedial measure to mitigate continued damage to avoid future subsidence related problems at the rock face and the building site.

PROJECT SITE CONDITIONS

The building is located in an open shopping mall in West Virginia. The elevation of the pre-existing ground surface sloped across the building ranged from about 1320 to 1080 ft. east to west (see Figure 1). The site was developed in the early 1990s by excavating a bench into the foothill at an elevation about 1,085 ft high. The excavation into the foothill has resulted in an almost 400 ft., steeply benched rock cut that is about 80 ft. east of the building (see Figure 1). Because this rock cut across the old works, which were abandoned in the 1940s, mine openings were exposed in the rock face. Exposure of these openings by the cut has resulted in deterioration over the years from freeze/thaw and wet/dry cycles. As the roof rock material broke down, cave-ins occurred that progressed upward through the rock strata. This upward progression of roof cave-in could result in undermining the upper bench of the rock cut, leading to localized instability and rock fall



Figure 1. Calculated pillar safety factor isopachs of Mine No. 15 JR below the northern half of the building at the time of second mining (left figure) and after the site development (right figure).

events. There was also evidence that the coal left to support the overburden (i.e., coal pillars) was compromised by the rock cut. This was seen in the form of stress relief fractures, indicating some of these pillars seemed to be in the process of yielding (see Figure 2). The rock cut exposed these coal pillars to weathering, but, more importantly, it cut some of the pillars in size. Calculations indicated that pillars in these old works would not have sufficient capacity in the long term. If successive pillars would fail, the overburden would drop and create a more massive instability in the rock face, possibly resulting in large boulder falls to toppling rock slabs, given a prominent joint set which could be seen in the rock cut face. This sub-vertical, planar joint set is oriented sub-parallel to the cut, thus creating the slabs of rock (see Figure 3).



Figure 2. Stress relief fractures indicating the pillars in the process of yielding.

SUBSURFACE INVESTIGATION AND GROUND CONDITIONS

The subsurface investigation was conducted at the project site to assess rock properties above to immediately below mine level for Mine No. 15 Jr. (No. 2 gas seam) and existing mine conditions at the site. Accordingly, 9 borings were drilled at the project site.

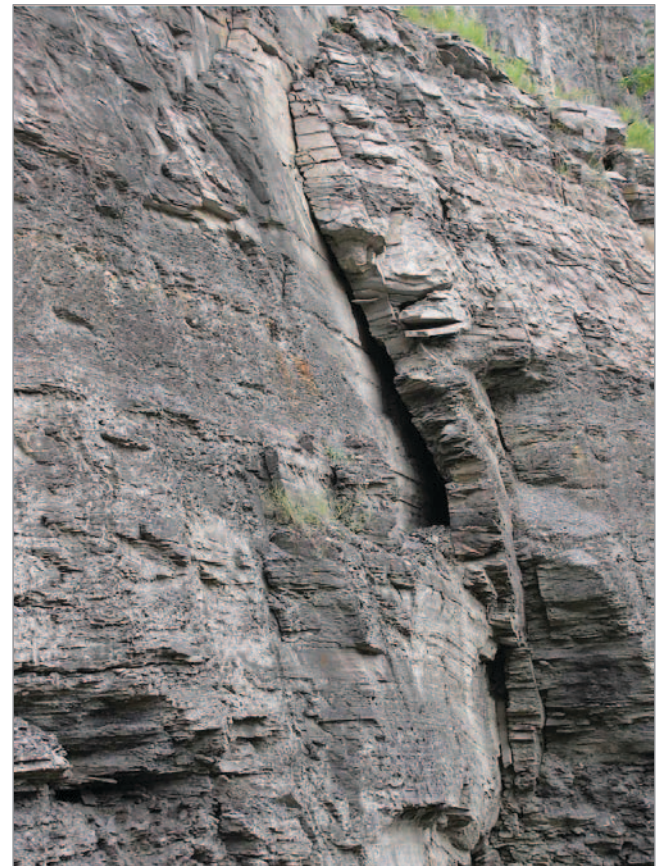


Figure 3. Planar sub-vertical joints sub-parallel to the rock cut creating rock slabs. Note: Bulge in rock slab which resulted from lack of support from mining opening at base of cut.

Of the 9 borings, 4 were drilled into the floor of the No. 2 gas seams to a depth ranging from 162 to 187 ft. In all these borings, continuous core was taken in the roof to the floor. The remaining 5 borings were drilled into the floor of the Upper Cedar Grove Coal, which is the upper mined seam. These holes were 30 to 34 ft. deep. Coring commenced in these shallow borings once bedrock was encountered. In addition to coring, the intersected mine voids and the sidewalls of all the investigative holes were inspected and recorded with a borehole camera. The observation from borehole camera revealed only some slightly open fractures in the overburden rock and these fractures were found to have groundwater sprinkling out of the borehole walls. The encountered mine void generally contained rubble. The mine appeared essentially dry and venting barely noticeable mine gases. No methane content in the mine gas was detected. Also, various rock mechanics tests were performed on the cored samples to establish the geotechnical properties of the mine roof and floor. All this information was used to analyze the mine stability and the site's susceptibility to subsidence.

The soil overburden materials (engineered fill or weathered/broken rock) on site ranged from 5 to 55 ft. thick. Based on the borings in the northern half of the site, the soil layer was about 5 to 15 ft. The borings to the south of the building generally had a greater thickness that ranged from 14 to 55 ft. This increase in soil thickness conforms to the regional structural dip of the coal seam, which was stripped during the site development. The fill/rock

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generally consisted of blast rock and residual soil. Since no soil sampling was done for this project, the exact interval of the contact between bedrock and fill has been unknown. It has been assumed that portions of the soil/fill section were, indeed, fractured bedrock according to drillers' comments. The roof rock was comprised of silty shale and sandstone and the floor rock of sandy shale grading to sandstone. Based on the deep mine borings conducted in the grout pad area, the encountered mine void interval was fairly limited in height from 2.8 to 5.5 ft. and was typically encountered from depth ranging from 155 to 170 ft. In some areas of the grout pad, a significant roof to floor closure existed. In these areas, less grout was expected to be injected because of rubblized conditions and reduced void heights.

MINING CONDITIONS

At the existing building site, there had been underground mining of coal. There had been two coal seams at the site that were mined using room and pillar techniques. The upper mine was called Mine No. 15 Upper Cedar Grove Seam, and the lower mine was Mine No. 15 Jr. (No. 2 gas seam). Both mines were operated in the 1940s. Later, in 1967, Mine No. 15 Jr. was reopened. There has been no production record for either mine, and it has been unknown for how long the mining operation was lasted. Based on the project data, Mine No. 15 was removed below the building in the 1990s during the site development. Hence, the effect of the Mine No. 15 below the building was not a concern. However, at about 160 ft. below the ground surface, Mine No. 15 Jr. was present. The mine map with the building and rock cut is shown in Figure 1. The map indicated that the production area had been second mined below the northern half of the building and to the northeast of the building beneath the rock cut. The northern and the southern halves of the building are separated by a 70-ft.-wide barrier pillar present below the middle of the building (see Figure 4). The room and pillar geometry in the production area below the northern part of the building and the rock cut is as follows:

First mined areas:

W_p = pillar width, 15 ft.; L_p = pillar length, 29 - 63 ft.; e = extraction ratio, 75%

Second mined areas:

W_p = 5 - 15 ft.; L_p = 15 - 26 ft.; e = extraction ratio, 88%

Below the southern half of the building, the mine map indicated that there was no second mining south of the 70-ft. wide barrier (see Figure 4). A total of 15 drill holes were done to confirm the presence of this barrier. The room and pillar geometry in the southern part of the building scales:

W_p = 35 ft.; L_p = 55 ft.; e = extraction ratio, 55 %

Barrier pillar (Below the middle of the building)

W_p = 70 ft.; L_p = 650 ft.

MINE STABILITY

There are three modes of mine stability that are most prominent, and they are roof collapse above the rooms, pillar crushing, and



Figure 4. Grout hole location in the northern side of the coal barrier pillar.

floor bearing failure. Following is the evaluation of the long-term stability considering these various failure modes of the subjacent mine work. This analysis provides an indication for the more critical mine stability conditions.

Overburden Roof Stability

Based on the project borings, there were two distinctive roof units present above Mine No. 15 Jr. The upper unit existed from about 30 to 120 ft. and could be over 90 ft. thick. This unit graded downward from fine-grained sandstone to a silty shale, essentially made up of sandstone in the upper ~65 ft. The lower unit was present from about 122 to 153 ft. and was found to be about 30 ft. thick. This unit graded with depth from a silty shale to a fine sandstone and then to a sandy shale.

With respect to surface subsidence, the controlling roof unit would be the upper sandstone. This very good quality rock would have the greatest spanning capacity because of its thickness and lighter overburden weight. Using a linear arch theory (Marino, 1988), and considering the average indirect tensile strength of 890 psi (or a uniaxial compressive strength of 9,000 psi), this roof unit has the potential to span hundreds of feet. Given this expected roof capacity, the second mining done beneath the site would not have been conducted to the point of pillar failure because of the safety issue related to the rapid progression of roof collapse as a result of the potential of the roof to carry overburden load to the adjacent pillar, causing them to fail.

Given the capacity of the overburden roof, this roof could easily span distances between the remnants pillars of up to 50 ft. Moreover, given that the laboratory tests indicated this rock to be durable, stoping, or a progressive upward collapse of the roof above a room or entry, was not likely.

Pillar Stability

The capacity of the underlying coal pillars was assessed by the use of Mark-Bieniawski (Mark, 1999) pillar strength relationship. The Mark-Bieniawski equation is as follows:

$$Sp = S1 (0.64 + 0.54 (Wp / Hp) - 0.18 (Wp^2 / (Lp * Hp))) \quad (1)$$

Where, Sp = pillar strength; Wp = pillar width; Lp = pillar length; Hp = pillar height (3.5 to 4.0 ft.) but potentially up to 5.0 ft.; and S1 = in-situ coal cube strength, 900 psi (Mark, 1990).

The vertical pressure exerted by the overburden on these pillars was determined from the tributary pressure, σ_{pt} , where

$$\sigma_{pt} = (1.1 * D) / (1 - e) \quad (2)$$

Where D = mine depth; and e = extraction ratio.

The safety factor (SF) against failure is then defined as

$$SF = Sp / \sigma_{pt} \quad (3)$$

Under the areas of the building and the rock cut, there were areas that were both first and second mined in Mine No. 15 Jr. In the first mined area, under the rock cut, the calculated SF was below 1.5. For the high extraction areas, the results from the SF calculations are provided in Figure 1 for the most critical conditions for when mined (assuming minimum Hp = 3.5 ft.) and after development (assuming maximum Hp = 5.0 ft.). As shown in Figure 1, there are no old work areas that meet SF of 2.5, and, in fact, areas with SF over 1 are in the minority. These results correlated fairly well with the mine void conditions encountered in deep borings. The mine void conditions were rubblized in many cases indicating distress; however, voids as high as 5.5 ft were encountered. From common mortar joint (CMJ) survey around the building, the CMJ variation was up to 0.38 ft with the most variation found around the northern half of the building above the high extraction old works. On the southern half of the building, the CMJ variation was within 0.08 ft. and within construction tolerances

For the barrier pillar, the abutment loads transferred to it from roof to floor convergence in the high extraction area to the north should be considered. The total load on the barrier consists of the tributary load and the abutment load on the north side of the barrier. The tributary or development load (Ld) is the load induced by the overburden directly above the barrier and is given by the equation

$$Ld = \gamma * Dc * Wb \quad (4)$$

Where, γ = average unit weight of overburden in pcf; Dc = soil and rock cover above the pillar in ft.; Wb = width of barrier in ft.

The abutment load that occurred as a portion of the weight of the overburden that had been supported by the adjacent panel pillars is transferred to the barrier upon yielding. The side abutment load could be determined by the equations proposed by King and Whittaker (1971) for supercritical and subcritical subsidence profiles. Assuming a supercritical subsidence profile (or a significantly wide collapse zone), the abutment load (Ls) is given by the equation

$$Ls = (\gamma/2) * (Dc)^2 * \tan(\eta) \quad (5)$$

Where, η = shear angle, determined to be 25° for the British conditions and was proposed to be 18° for Pittsburgh seam by Choi and McCain (1980). For the current study, a conservative shear angle of 20° was used. Therefore, the total loading on the barrier is given by the equation

$$LT = Ld + Ls \quad (6)$$

Where, LT = total load, kips/ft.; Ld = development load induced by the overburden directly above the pillar, kips/ft.; and Ls = side abutment load, kips/ft.

Using the above procedure resulted in a load along the barrier of 2,460 kips/ft. Using Equation (1), the capacity of the barrier was 71,930 kips/ft. Therefore, SF was over 10. In the area below the southern half of the building, the calculated SF for the pillars were over 3.0.

Floor Stability

The mine floor across the site consisted of durable sandy shale, which was about 1 to 3 ft grading into 8 to 18 ft. of fine grained, durable sandstone. Because only durable materials were encountered, floor softening was not considered (Marino and Osouli, 2012). The immediate sandy shale and sandstone were tested to have uniaxial compressive strength values of 5,540 psi and 6,600 to 11,000 psi (average: 8,780 psi), respectively. To assess the bearing capacity of these durable floor materials, the following equation from Vesic, 1975 was used:

$$qu = C1 * Nm \quad (7)$$

Where, qu = ultimate bearing capacity; C1 = cohesion of upper layer; Nm = modified bearing capacity factor based on the ratio of cohesion of upper and lower layers and the ratio of the footing width to the thickness of the upper layer.

Considering the above floor conditions and the maximum pillar pressure and the minimum pillar width, SF was still over 3.0. Consequently, floor failure was not likely.

Based on the above rock mechanics analyses, the critical stability condition was related to the pillar crushing. The factors of safety against pillar failure were calculated for the mine area in the northern half of the building and the adjacent rock cut, which were, in many cases, less than 1 and were in no locations within an acceptable range. Other areas to the south of the northern half of the subjacent old works only contained first mining and had sufficient pillar SF.

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MINE GROUTING

After the engineering investigation and the stability analysis were conducted, the most cost effective grouting methodology was determined. The grouting project was divided into two work units: Phase I and Phase II. Phase I refers to the completion of drilling and grouting of deep mine voids in the northern half of the building and also some areas of rock cut in the northern half. Phase II refers to the installation of bulkhead grout in exposed open cave-ins and drilling and grouting into upper mine voids in the rock cut. Phase I and Phase II were conducted simultaneously to meet the project deadline. Below is the discussion of mine remediation work carried out underneath the building and in the rock cut.

Mine Grouting Underneath the Existing Building

The first phase of mine remediation involved grouting of the northern portion of the deep mine workings beneath the existing building. The grouting was performed north of the east-west coal barrier depicted on the mine map (see Figure 4). The work also involved grouting some areas under the rock cut in the northern half of the project site. The objective of the grouting was to stabilize the mine and rock overburden to avoid roof to floor convergence in the specified areas beneath the building and the rock cut.

The onsite drilling for grouting commenced with the drilling and grouting of primary injection holes around the perimeter of the proposed grout area. This was followed by split-spaced secondary holes along the perimeter. With the need to grout beneath the existing building and the areas of the rock cut, most of the injection holes were drilled at an angle to intersect the mine void at the specified location. Once the perimeter grouting was completed, the primary stage of infill grouting was performed from west to east across the specified grout area. The infill area was again similarly grouted during the secondary stage. The secondary injection holes were split spaced the primary holes. Upon the completion of secondary stage of grouting, spot holes were done based on the observed grout performance at the grouting area. During the drilling process, borehole camera inspection was performed to assess the borehole and mine void conditions.

The entire caved interval was grout injected in stages starting 1 foot above the mine floor and then moving upwards. Up-hole open fractures in the rock overburden were also slush grout injected. Once the grout injection work was completed, the holes were stemmed with grout to the ground surface.

Three different types of grout mix were used in this phase of project: Mix A (30 sec efflux), Mix B (9.0 in. slump), and Mix C (6.0 in. slump). The mixes were designed targeting a 28-day compressive strength of 200 psi.

During the grouting process, each grout hole was injected with a minimum amount of water to "slick" the grout line from the pump to the tip of the injection pipe. Then Mix B was pumped into the grout hole for the perimeter grouting. Two criteria were set for the limitation of grout type for the perimeter grouting: if there is only nominal initial pressure build-up, Mix B is injected until (a) a certain volume of grout reaches at which point Mix C is injected to refusal with the possible use of additives if necessary, or (b) Mix B is refused off. Since the intent of injection into the infill holes

was to produce a significant amount of grout intakes, only a small amount of Mix C was used. Once the refusal of grout had reached, the injection pipe was moved up about 5 ft. and the grout injection was again attempted into the hole. This injection process was continued until the top of the roof caving was reached. The intent was to maximize grout injection pressure in order to achieve the most penetrative and intrusive compaction or filling as possibly feasible. Refusal was based on reaching header line pressures on the order of 200 psi. Once the mine grouting was completed, the up-hole open fractures were grout injected with Mix A. The amount of grouting of the open fractures was determined by the refusal of the hole to take the grout. Upon the completion of grout injection in the open fractures, the holes were stemmed with Mix A to the ground surface. A total of 27,500 lineal feet of drilling was required for the deep mine grouting. A quantity of 6,660 CY of grout was pumped into the mine voids.

Mine Grouting on Rock Cut

The second work of mine remediation involved the construction of bulkheads of grout on the face where open caved-in entries were present and grouting into the upper mine workings along the rock cut immediately to the east of the building. The objective of this specified grouting was to stabilize the mine and rock overburden to mitigate any roof to floor convergence in the outlined grout areas. The grout injection of the mine openings extended into the rock face 75 ft.

Prior to any remediation work on the rock cut, all loose or unstable material along the lower rock face was removed. Also, a wire-mesh drape was installed on the rock face for the control of rock falls from upper rock cut benches. In addition to providing support for the overhanging rock in the exposed open entries, the installed grout bulkhead would provide containment for the interior grouting of the mine openings in the area. In order to provide solid roof to floor contact to support the overhang and to best interlock bulkhead grout mass, all the caved material was removed from below the overhang area. Fallen roof material was excavated to intact mine floor level.

Two different types of grout mix were used in this phase of project: Mix F1 (6.0 in. slump) and Mix F2 (30 sec efflux). The mixes were designed targeting a 28-day compressive strength of 500 psi.

For bulkhead construction in the cave areas, a containment wall (formwork) was installed to a sufficient height, and Mix F1 was pumped behind the formed wall. The grout was installed in several lifts. The lift size varied from 2 to 3 ft. A 4-in. protection coating of shotcrete was applied to the exterior face of the bulkhead grout after the grout injection was completed.

In the rock face, the mined-out upper seam dipped from the north to the south. In the project area, the top of the upper seam on the rock cut was exposed at about 15 ft. above the toe of the rock cut north of the building and disappeared below the toe at about the middle of the building. Consequently, for drilling and grouting of the upper mine void to the north from the middle of the building, stone ramps were required in the drainage ditch that existed along the toe of the rock cut. Also, there were some areas on the rock cut where distressed coal pillars existed. Prior to grouting in the vicinity of these exposed, distressed pillars, convergence meters

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were installed on the pillars to monitor the lateral displacement of rock cut face while drilling and grouting into the mine voids. The convergence meters showed no lateral displacement of the rock cut face during drilling and grouting in the vicinity of the distressed areas.

The grout injection in the upper mine void began with drilling sub-horizontal injection holes parallel to the orientation of the mine openings. The inclination of the perimeter holes was positioned so that, at 75 ft., the initial injection point would be 1 ft. above or on the mine floor. Once grouting of this perimeter hole was completed, a second injection hole in the same entry was split-spaced between the perimeter hole and the rock face with a target initial injection point of again 1 ft. above or on the mine floor. This second hole was offset horizontally from the perimeter hole within the mine opening. With the completion of second injection holes, additional spot holes were drilled at a greater angle from a higher elevation, about 5 to 7 ft. above the previous holes on the rock face. These spot holes were required to verify if there were still any significant void below the roof that required grouting. Hence, for drilling and grouting these additional holes, higher stone ramps were required along the rock cut.

During the grouting process, each grout hole was injected with a minimum amount of water to “slick” the grout line from the pump to the tip of the injection pipe. Then Mix F2 was pumped into the grout hole. Once the refusal of grout had reached, the injection pipe was pulled out about 5 ft., and the grout injection was again attempted. Two criteria were set for the refusal of the grout: (a) Mix F2 is injected until the header pressure reaches 400 psi, or (b) Mix F2 is refused off. If, during the grouting process, the grout leaked out to the rock face through the open fractures, the grouting was stopped to let the grout set up for a day and again attempted the following day. This injection process was continued until the top of the roof caving was reached, and the rubblized section of mine roof was completely grouted. The intent was to maximize grout injection pressure in order to achieve the most penetrative and intrusive compaction or filling as possibly feasible. Once the grout injection work was completed, the holes were stemmed with the grout to the ground surface. A total of 4,600 lineal feet of drilling was required for the upper mine grouting, and a quantity of 2,285 CY of grout was pumped into the mine voids. A total of 1,660 CY of grout was required for the bulkhead construction.

SUMMARY AND CONCLUSIONS

A subsidence engineering investigation was conducted at an existing building in West Virginia. The concern was related to potential collapse of the abandoned room and pillar mining beneath the building and adjacent rock cut immediately to the east of the building. A subsurface investigation was undertaken,

where it was discovered that the old works closest to the surface were not present under the building and were stripped out to the existing over 400-ft.-high rock cut during the site development. The deeper mine was encountered at the site at a depth of around 160 ft. A 70-ft.-wide solid barrier pillar was present below the middle of the building. In the northern side of the barrier pillar, very poor pillar support conditions were left, but the southern side had very low risk of collapse. Hence, deep mine grouting was conducted in the northern half of the site. A quantity of 6,660 CY of grout was pumped into the deep mine voids. Along the rock cut to the north of the structure, there were two large open old mining caves. Bulkhead construction was required for the caves. A quantity of 1,660 CY of grout was required for the bulkhead construction. Upon completion of bulkhead construction, upper mine grouting was carried out in the rock cut to properly stabilize the mine and rock overburden to mitigate roof-to-floor convergence. A total of 2,285 CY of grout was pumped into the upper mine voids.

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Economic Impact of Ascending Mine–Waters in Germany

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ABSTRACT

Due to a political decision, German hard coal mining will come to an end in the year 2018. By then, there will have been more than 150 years of industrial grade mining activities. Located in the western part of Germany is one of the most densely populated areas of Europe. Multiple coal seams have embossed the earth's surface up to 20 meters (21.87 yards), as mining has subsided. The aftercare of hard coal mining will consist of three tasks: groundwater management, polder management, and groundwater treatment. The *Ewigkeitslasten* (cost in perpetuity) at the current groundwater level has been determined in the past. The actual cash value is calculated to be about 12.4 billion euros (approximately 16.5 billion dollars).

The cost of groundwater management is directly related to the groundwater level. Rising groundwater levels will reduce the cost of pumping water. Known effects of rising groundwater are uplift of the surface, emission of methane gas at the surface, higher risk of cave-ins, and contamination of drinking water horizons. Each of those processes produces cost in order to prevent negative consequences (e.g., uncontrolled methane emissions) or to handle reparations (e.g., damages caused by surface uplifts).

In the long run, it is the goal to minimize the *Ewigkeitslast*. As a first step, all cost-generating processes are specified, and a cost type based characterization is performed. The objective of this case study is to identify key parameters to determine the quality of a cost-generating process (proportional cost, fixed amount cost). Afterwards, it is intended to specify the quantitative impacts of each parameter and to set up an optimization model to minimize the overall cost. The quantitatively specified parameters will form the base out of which the optimization model is created.

INTRODUCTION

The area in the western part of Germany known as Ruhrgebiet is a seedbed for the German mining industry. Geologically, this region belongs to the European Coal Belt, which was formed during the Carboniferous period. The seams of the coal-bearing formations strike out to the surface in the southern part of the area and dip in northern direction.

Near-surface hard coal mining activities began as early as the 11th century. Afterwards, with the beginning of the industrialization in the 19th century, hard coal mining intensified and eventually reached its peak in the 1970s when the annual production was approximately 110 million tons from 140 mines and a workforce of about one million employees in the coal and steel industry. However, due to complicated geological conditions and a large overburden, the annual production declined steadily until today. In 2011, there were only five hard coal mines still operational, producing 12.1 million tons combined per annum. They were only competitive on the global market due to governmental subsidies. According to the current legal situation, these governmental subsidies will come to an end in 2018, which will then lead to the shutdown of the remaining German hard coal mines (statista.com, 2013; sendfeld.de, 2013; rag-deutsche-steinkohle.de, 2013).

The history of industrial grade mining activities in the Ruhrgebiet is about 200 years old. During that history, in some places, more than 15 seams were mined on top of one another with a combined thickness of approximately 25 m. Multiple coal seams have embossed the earth's surface up to 20 m, as mining has subsided. The remaining active hard coal mines are being operated in depths of up to 1,300 meters (about 1,422 yards) (rag-deutsche-steinkohle.de, 2013).

Due to the industrial development of the region, the population of the Ruhrgebiet increased to about 5.3 million, with a population density of 1,195 inhabitants per km², which is equivalent in both parameters to Atlanta, Georgia. The population density in the heart of the conurbation, under which most of the mining activities took place, amounts to 3,277 inhabitants per km² (more or less equivalent to Washington, D.C.) (ruhrgebiet-regionalkunde.de, 2013).

The shutdown of hard coal mining in 2018 is not synonymous with the cessation of all activities, as pumping groundwater will remain a task to be performed in perpetuity. In some parts of the Ruhrgebiet, the groundwater level has been lowered to approximately 1,300 m. The upcoming water accumulates to about 100 million m³ annually. An entire discontinuance of mine dewatering would inevitably lead to waterlogging and partially inundate the surface. Additionally, other aspects (e.g., the

protection of drinking water horizons) prohibit the entire shutdown of mine dewatering.

PROBLEM STATEMENT AND OBJECTIVE OF THE PAPER

The effects of mining activities, as well as the necessity of aftercare, were identified at an early stage. Therefore, a management process for the aftercare has already been developed. The actions, which have to be carried out subsequent to mining activities, are the so-called *Ewigkeitsaufgaben*, and they consist of three tasks: groundwater management, polder management, and groundwater treatment. All of these tasks have to be performed in perpetuity. According to current calculations, the cash value of costs in perpetuity is approximately 12.4 billion euros (equal to about 16.5 billion dollars) only for the Ruhrgebiet. The main cost factor is the groundwater management; it is directly linked to the groundwater level. At the current level, the cost for pumping 1 cubic meter (about 35 cubic feet) of water to the surface are about 1€ (\$1.33). The amount of water will remain almost constant, regardless of the mine water level. Nonetheless, the rise of the mine water level will result in a reduction in cost because the main cost factor is the power used to run the pumps. The power demand of pumping is correlated to the mine water level (Papst, 2006).

Rising groundwater has several technical effects, all of which result in specific cost developments. Some of the costs are associated with the mine water level. Consequently, the rise of the mine water level correlates positively with cost due to those technical effects and negatively with the pumping costs.

In this article, an approach will be presented that aims to minimize costs in perpetuity. Therefore, the effects of rising mine water, as well as the resulting costs, will be outlined. Based upon the definitions of the resulting costs, an optimization model may be set up to determine the minimal costs of the mine water rise and the corresponding mine water level.

EFFECTS OF ASCENDING MINE WATERS

The following paragraphs will explain the technical effects of the ascending mine water level.

Surface Uplift

A rise of mine water level into rock layers, which were kept dry during active mining, leads to a change in pressure of pore spaces. Particularly in the area of fractured zone above a created cavity, the change in pressure and the buoyancy created by mine waters induce an expansion of these rock layers. The expansion of rock layers and the buoyancy eventually account for an uplift of the surface (Sroka and Preusse, 2009) (Preusse, Kateloe, and Sroka, 2012).

According to the current state of knowledge, the measure of uplift is dependent on the following parameters:

- Geological circumstances
- Thickness of coal seams
- Depth of coal seams
- Dip of coal seams
- Excavation ratio of rock layers
- Backfill material

- Mine water level
- Velocity of the mine water rise

The temporal relation between the rise of the mine water and the surface uplift is not yet quantifiable. Considered qualitatively, the uplift of the surface is slightly delayed in time compared to the rise of the mine water. Several studies also show the correlation between the velocity of the mine water rise and the surface uplift. Empirical studies of flooded mines have shown that the amount of surface uplift is about 2 to 5% of the previous subsidence (Sroka and Preusse, 2009; Pöttgens, 1998; Sroka, Preusse, and Holzheim, 2006).

The components of the ground movement in the process of surface uplifts are identical to the components occurring during subsidence. The processes of uplift proceed mainly in a constant fashion and impact a broad surface area. At present, the angle of impact concerning these processes is presumed to be approximately 9° to 13.5°, compared to 45° to 50° in the process of subsidence (Sroka and Preusse, 2009).

Costs due to uplifts of the surface could accrue from the two following fields:

- Damage on infrastructure and other development due to mining activities
- Impairment of the surface drainage capabilities

In already flooded mining areas, damages on infrastructure and development always occurred in lines. In most instances, these lines coincided with terraces (i.e., major differences of uplift in a small area). Numerically, the differences of uplift that caused damage were in the range of 2 to 7 centimeters (.79 inch to 2.76 inches). In large part, the damage lines were in areas with geological fault zones striking out at the surface. It is assumed that damages on infrastructure and development due to surface uplift will mainly occur along geological fault zones in the Ruhrgebiet, as well (Baglikow, 2006; Fischer and Wildhagen, 2006).

Because of the uniformity of surface uplift and the broad impacted area, it is currently assumed that the surface drainage capabilities will not be impaired by ascending mine waters.

Firedamp

Gases are formed along with coal during the coalification. These gases are bound to the deposit and consist of methane (90 to 95 vol%), ethane (0.1 to 3 vol%), carbon dioxide (2 to 4 vol%), nitrogen (0 to 5 vol%), as well as traces of hydrogen and helium. However, even within a deposit, the composition of the firedamp can vary strongly (Meiners, 2001).

Driving the mine workings, as well as the excavation of coal, loosens and relaxes the overburden. The stored gas is released and accumulates in the ambient air. As a consequence of the rise of mine water, the firedamp is being pushed out of the rock body towards the surface. Even without the rise of mine water, methane flows towards the surface and discharges via outlets at the surface. Not taking into account the rise of mine water, the majority of outlets at the surface are known. However, if the mine water rises, the flow velocity of methane increases, leading to more outlets at the surface (Fischer and Wildhagen, 2006).

Methane becomes explosive at a concentration between 4.1 to 17.16 vol% in ambient air. Therefore, the potential risk of an explosion exists when methane accumulates in the surrounding air. The density of methane in gaseous state is 0.72 kg/m³ (0.0447 lb/ft³). It is exceeded by the average density of ambient air—approximately 1.3 kg/m³ (0.0806 lb/ft³). Thus, an explosive concentration of methane can only occur in closed areas such as buildings, mine workings, and zones around backfilled shafts (Meiners, 2001)(Bartknecht, 1993).

Near-Surface Mining

Due to the long tradition of hard coal mining in the Ruhrgebiet, there are numerous near-surface mine workings and cavities. In the recent past, there have been several sinkholes caused by hydrological influences. The appearance of these sinkholes stood in relation to one of the following situations:

- Remaining mine workings of near-surface mining activities
- Failure of backfilled shafts

Cause for the appearance of sinkholes related to the first situation is the lack of carrying capacity of the rock layers above mine workings. The thickness of primarily sustainable rock layers can decrease under the influence of water, which then leads to a decline of carrying capacities. Moreover, the influence of water on insufficiently secured shafts can result in sagging or complete failure of backfilled shafts (Grigo, Welz, and Heitfeld, 2007).

Both situations (described in the last paragraph) can occur under the influence of mine water and as a result of rise of the mine water level. The risk potential, however, is still present regardless of the mine water level. Nevertheless, in the long run, the danger of the appearance of sinkholes can even be reduced by a rise of the mine water level since a permanent contact to water prevents infrequent rinsing with infiltrated water. Also, this reduces the risk of erosion, suffosion, and colmatation (Grigo, Welz, and Heitfeld, 2007).

Shafts

There are approximately 5,300 surface openings and shafts in the Ruhrgebiet. To stabilize shafts and prevent the appearance of sinkholes, a classification of shafts according to the depth and the type of backfill process would be reasonable. Regarding the depth, it is distinguished between near-surface shafts (depth < 100 meters or 109.36 yards) and deep-mining shafts (depth > 100 meters or 109.36 yards). Regarding the type of backfill process, it is distinguished between non-backfilled shafts, non-long-lasting and instable backfilled shafts, and long-lasting and stable backfilled shafts. (Fischer and Wildhagen, 2006)

The previous section pointed out the influence of ascending mine water levels on near-surface shafts with non-long-lasting and instable backfilling, as well as the problems resulting from it. The same processes may also occur in deep-mining shafts if they are not backfilled stable and long-lasting. A shaft is considered to be stable and long-lasting if the safety of the public and also the re-use of the surrounding surface area are ensured. To achieve this, shafts are partially or completely backfilled with cohesive filling material; of the two, the process of complete backfilling is preferable to partial backfilling (Meier, 2007).

The contact with water may induce one or several of the following processes, which can potentially have negative effects on the backfilling material:

- Erosion—Detachment of rock particles and minerals and the resulting deformations of the affected material. This process can subsequently affect the stability of backfilling material.
- Suffosion—Washing-out of fine soil fractions. The pore size and hydraulic permeability increases which leads to an increase of the risk of occurring erosion.
- Colmatation—Fine-grained particles deposit in pore spaces, which reduces the hydraulic permeability of the soil. Waterlogging can then lead to a lack of stability of the shaft.
- Corrosion—Chemical modification and decomposition of the backfilling material, which negatively affects the stability of the shaft (Patt, 2001).

The amount of time that long-lasting and stable backfilled shafts are deemed secure is predicted to be approximately 100 years. Shafts that are backfilled differently at present will undergo further constructions in the years ahead in order to make them long-lasting and stable. In the context of this paper, the duration of the stability of these shafts is only of secondary importance. The order of shaft stabilization is based on a priority list, which is a result of assessment and consideration of a variety of influencing parameters regarding the conditions of the shafts; these include water level, use of the surface, and the previous form of stabilization (Arnsberg, 2012; Fischer and Hollmann, 2010).

Groundwater

The Ruhrgebiet is divided into three hydrologic regions, which differ from each other in the way of composition of mine water. Mine water in the southern region contains HCO₃ and SO₄. The central region is characterized by a high mineral content, and the northern region is characterized by the mine waters' richness in NaCl. Ingress of mine water into drinking water horizons would have fatal repercussions on the drinking water supply of the whole region and must be avoided. This is regulated by law in the Verschlechterungsverbot ("no deterioration act") and the Water Resources Act.

The majority of the fresh water reserves are located in the northern part of the Ruhrgebiet where impermeable water layers of marl are found underneath the water reservoir. In addition to the natural protective layer, the mining company decided to instate a safety margin between the intended mine water level and the drinking water horizon. This safety margin will be maintained in all three hydrologic regions of the area (Grigo, et al, 2007; Blachere, et al, 2006; Sroka and Preusse, 2007).

COST OF ASCENDING MINE WATERS

The basis for the optimization model to minimize the cost generated by ascending mine waters is the definition of all potential cost items. Furthermore, it is desirable to determine the respective correlations with the mine water level. The optimization itself requires the mathematical definition of cost functions. This process will not be part of this article. Another precondition for an optimization is the temporal comparability of cost. Therefore, in addition to the amount of cost, it is also necessary to know the

effective date when cost occurs. To guarantee the comparability, the actual cash value of each cost item at the date of comparison will be deduced from the date of occurrence.

This section will describe the quality of the cost of each process, as well as the correlation between cost and time, respectively, with regard to the mine water level.

Surface Uplifts

As discussed earlier, negative impact on the surface due to surface uplifts will most likely present itself in the form of mining damages. According to the current state of science, mining damages will most likely occur in lines. Those lines will coincide with terraces activated by the mine water rise.

The lower limit of uplift differences at terraces that cause mining damages is 2 cm (0.79 inch). The amount of surface uplift is about 5% of the beforehand occurred subsidence. The implication of those two observations is that mining damages due to surface uplift can only be caused in areas with subsidence of more than 40 cm (15.75 inches). This condition is fulfilled in the majority of the Ruhr area.

The correlation between the mine water level and surface uplifts and particularly the differences of surface uplifts in small areas is not yet determined conclusively. Nonetheless, the following assumptions can be made according to the state of science:

- Mining damages will occur in lines along terraces.
- The amount of uplift differences in a terrace depends on the mine water level.
- Mining damages may occur as soon as the lower limit of uplift difference is surpassed.
- The higher the amount of uplift differences, the higher the number and amplitude of mining damages.
- At a certain upper limit of uplift differences all potential mining damages will have occurred in maximum amplitude.

The combination of those assumptions results in a cost pattern due to surface uplift and mine water level as shown in Figure 1.

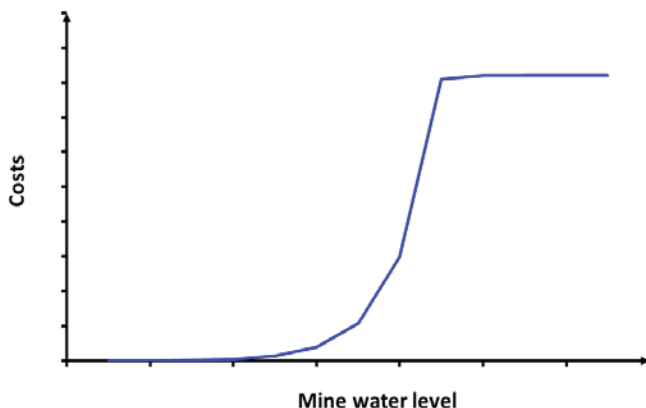


Figure 1. Costs due to surface uplifts.

The total amount of the cost depends on several factors. Among these are the density of development, number and length of terraces, and the average amount of mining damages. If the area-specific parameters are known, it is possible to determine the total amount of cost due to surface uplifts.

Firedamp

Cost due to mine gas is created by the necessity to prevent endangerment of the population. This means that the emergence of a hazard in a populated area requires immediate safeguards. Depending on the local conditions, there are three different safeguards:

- Sealing the gas leakage in an endangered building
- Passive degassing of a mining cavity
- Active degassing of an endangered building

The flow paths of the gas in the ground and the changes of those flow paths are unknown. Gas leakages will emerge at different instances in the future. Each emergence will result in the necessity to install safeguards in a certain percentage of the building surrounding the leakage. This means the costs due to mine gas can occur in uncertain intervals and in clusters. The graph of the accumulated cost over time is, therefore, cascaded, as shown in Figure 2.

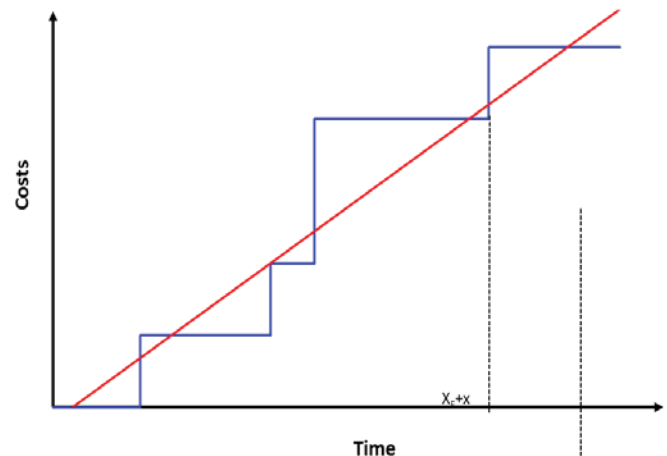


Figure 2. Correlation of time and costs due to firedamp; Grey: cascaded accumulated cost; Red: regression line.

The regression line in Figure 2 shows the accumulated cost as a liner function in dependence of the time. The point in time X_E is when the mine water level reaches its maximum. The last damage will occur at $X_E + x$. If the assumption is made that the mine water level will rise at a constant pace up to its maximum, the factor time may be exchanged with mine water level. The temporal link to the costs then transforms into a causal link. The maximum amount of cost is, therefore, directly correlated to the maximum mine water level, as shown in Figure 3.

The optimization model will require consideration of both links, the temporal and the causal, in order to work correctly. The reason

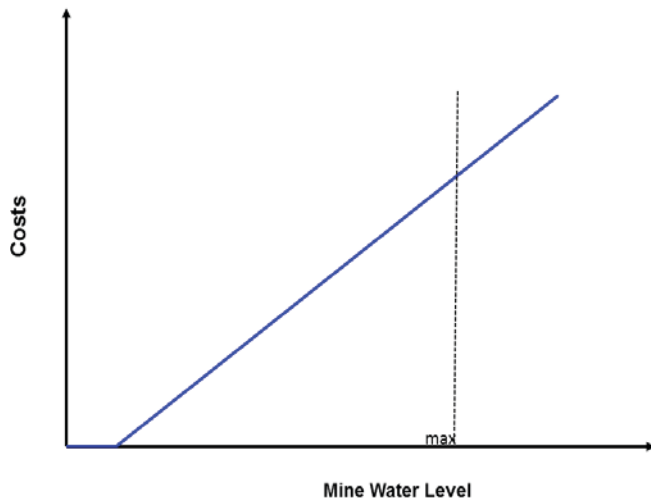


Figure 3. Correlation of mine water level and costs due to firedamp.

for this is the requirement to disregard the total amount of costs, and look at only the actual cash value.

Near-Surface Mining

Shallow mining is characterized by depths up to 100 meters (about 109 yards) below the surface. Because of the safety margin of the mine water level and the drinking water horizons described before, an interaction the mine water and the shallow mining is not to be expected; therefore, no cost is being generated.

Shafts

All non-long-lasting and stable backfilled shafts will be continuously reconstructed in the future regardless of the mine water level. Cost due to the mine water ascension will only occur because of the renewed necessity of reconstruction in the future. The amount of the cost depends on the amount of time the shafts are rated as secure. Consequently, the shafts have to be monitored permanently and reconstructed as soon as deficits emerge.

Assuming the reconstruction of all shafts is necessary about every 100 years, the current cash value of the costs can easily be calculated. The amount of the cost changes as the time period of safety changes. Assuming that the mine water shortens the period of safety, the costs will rise along with the mine water level. Reason being that, the higher the mine water level ascends, the more shafts and the larger parts of shafts are influenced. The resulting graph is shown in Figure 4. The partial gradients of the graph depend on the number of shafts, surpassing a certain length, respectively extending into the mine water.

Groundwater

As described previously, the mine water level will be kept to a safe distance from the level of drinking water. Hence there is no direct cost generated by the factor groundwater. Indirect cost may be generated by the restraint on the optimization the groundwater poses. Cost arises from the expenses for pumping water higher than necessary from an economical point of view.

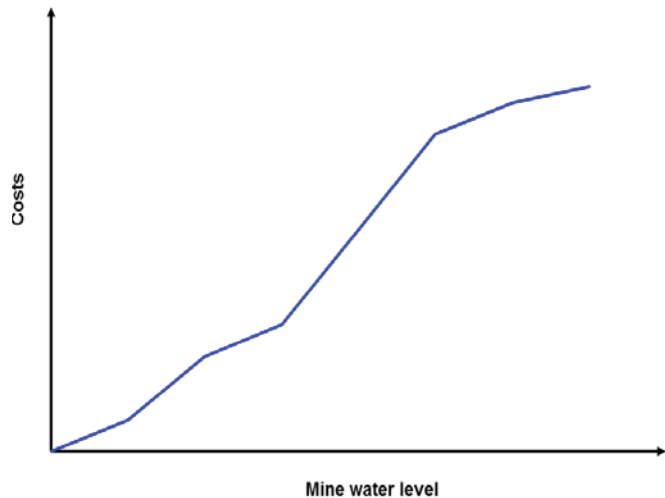


Figure 4. Costs due to shafts.

CONCLUSIONS AND OUTLOOK

After the shutdown of all mining activities in the Ruhrgebiet, the rock body will be flooded in order to save pumping costs. The rise of mine water will affect the rock body and the surface in several aspects. Those consequences may cause costs that can be correlated to the mine water level directly or indirectly.

As the basis for an optimization model with the goal of minimizing costs due to the rise of mine water, the correlations between the mine water level and functional costs have been examined. It has been pointed out that costs resulting from mining damage due to surface uplift, firedamp, and abandoned shafts are directly linked to the mine water level.

The concept of “no deterioration” regarding the freshwater and drinking water quality, as well as the restrictions on the rise of the mine water level, which is self-regulated by the mining company, limit the possible optimization. Therefore, the factor of groundwater is considered a potential expense factor. Because of the self-inflicted limitation of the maximum mine water level, the near-surface mine workings will not be influenced by mine water and are, therefore, not regarded to be relevant to cost.

In a follow-on project, the quantitative correlations of the cost components will be determined. On this basis, cost functions will be developed and combined in an optimization model. Such a model could theoretically be applied in any region. Nonetheless, the primary purpose of the model will be the determination of the economically ideal mine water level in the Ruhrgebiet.

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Recent Developments on Surface Ground Strain Calculations Due to Underground Mining in Appalachia

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ABSTRACT

The prediction of ground movements due to underground mining using the influence function method is a mature technology, widely used by researchers and planning engineers around the world. Surface strain, either tensile or compressive, is an important indicator of potential impacts on structures such as buildings, bodies of water, pipelines, railway and power lines, and tailings dams. Calculation of static or dynamic surface strains, for varying surface terrains and slopes, should be accurately predicted and be independent of the methodology applied for the calculation.

This paper discusses and compares the development of horizontal and ground strains on the surface due to underground mining for different surface topologies (sloping terrain, monitoring lines, etc.) and presents the steps needed to ensure accurate calculations. Examples are given to demonstrate how ground strains can be calculated for one- and two-dimensional gridded surfaces.

INTRODUCTION

When the ground surface is digitally represented as a Digital Terrain Model (DTM), it is either in raster form (a grid of squares with a mean value of elevation at the center of the grid cell) or as a vector-based Triangular Irregular Network (TIN). A surface grid can also be constructed starting from a digital TIN model or a digital elevation map. In all such cases, surface elevations are assigned to discrete coordinate pairs (points).

Different analytical models have been proposed to calculate vertical and horizontal displacements on the surface due to underground mining, e.g. SDPS (Agioutantis and Karmis 2013a), CISPM, (Peng, 2008) as well as a number of empirical models, e.g. by Holla and Barclay (2000). Most of the analytical models employ the influence function method for such calculations, which can be easily calibrated to calculate displacements for any mining geometry. Other indices, such as strain, may then be calculated by differentiating this function at specific surface points.

However, calculations do not usually take into account the influence of the surface terrain since the shape of the surface terrain is not available as a continuous mathematical function.

To account for the influence of the surface terrain, a continuous function representing surface topography should be available for projecting horizontal displacements prior to calculating strain. In general, small surface areas may be represented using continuous mathematical functions (Agioutantis and Karmis, 2013a), but this is of limited use since these representations only apply locally. As a result, different techniques should be employed for accurate ground strain estimations based on calculated values of surface subsidence and horizontal displacement due to underground mining.

Accurate calculation of surface (ground) strains is important, especially since it is one of the predominant deformation indices used for potential surface structure damage evaluation (Karmis, Agioutantis, and Jarosz, 1995). This paper discusses and compares the development of horizontal and ground strains on the surface due to underground mining for different surface topologies (sloping terrain, monitoring lines, etc.) and presents a methodology for calculating ground strains over one- and two-dimensional surface grids.

CALCULATION OF GROUND DEFORMATIONS USING INFLUENCE FUNCTIONS

Based on the influence function method, in the two- dimensional case, subsidence $S(x,s)$, at any point $P(s)$ due to an infinitesimal excavation, can be expressed by the following equation Karmis et al., 1990):

$$S(x, s) = \frac{1}{r} \int_{-\infty}^{+\infty} S_0(x) e^{\left(-\pi \frac{(x-s)^2}{r^2}\right)} dx \quad (1)$$

and for finite extraction limits (x_1, x_2)

$$S(x, s) = \frac{S_{max}}{r} \int_{x_1}^{x_2} e^{\left(-\pi \frac{(x-s)^2}{r^2}\right)} dx \quad (2)$$

where

s = the coordinate of the surface point

x = the coordinate of the infinitesimal excavated element

$So(x)$ = the convergence of the roof of the infinitesimal excavated element

$S(x,s)$ = the subsidence at the surface

$x1,x2$ = the extents of the excavation

$Smax$ = the maximum expected subsidence over the finite extraction (x_1,x_2)

r = the radius of principal influence $=h/\tan(\beta)$

h = the overburden depth

β = the angle of principal influence

Furthermore, for the case of a finite size rectangular parcel defined by the coordinates (x_1,y_1-x_2,y_2), equation (3) relates the horizontal displacement $U(x)$ to the first derivative of subsidence, and equation (4) relates the horizontal strain $E(x)$ to the horizontal displacement (see also Karmis et al., 1990):

$$U(x) = -Bs \frac{\partial S(x,y)}{\partial x} = Bs \frac{Smax}{r^2} \left[e^{\left(-\frac{x^2}{r^2}\right)} - e^{\left(-\frac{x_2^2}{r^2}\right)} \right] \times \int_{y_1}^{y_2} e^{\left(-\frac{y^2}{r^2}\right)} dy \quad (3)$$

$$E(x) = \frac{\partial U(x)}{\partial x} = -Bs \frac{\partial \left[\frac{\partial S(x,y)}{\partial x} \right]}{\partial x} \quad (4)$$

Where Bs is the strain coefficient in length units.

GROUND STRAIN CALCULATIONS ON A FLAT SURFACE FOR POINTS ON A PROFILE

Using this formulation, the strain at any point on the surface can be calculated as a function of mine geometry and overburden parameters. In fact, strain is calculated as the derivative at a single point of a continuous function, which represents the horizontal displacement at a point (see equations (3) and (4)). In such cases, ground strain is usually taken to be equal to the horizontal strain. In addition, it is common to calculate the axial or directional strain for points on a profile line.

The following is presented as a simple example of strain calculations (Figure 1). A transverse line is defined on a flat (horizontal) surface over a panel at an elevation of $z=500$ ft. Extraction thickness is assumed as $m=5$ ft, the supercritical subsidence factor is taken as $a=40\%$, and the edge effect is assumed as $d=0$ ft. Point spacing on the horizontal plane is 20ft. Using default parameters for the influence angle (β) and for the strain coefficient (Bs) as determined for the eastern Appalachian

Coalfields (Agioutantis and Karmis, 2013a), subsidence, horizontal displacement, and strain are calculated and shown in Figure 2. In this case, horizontal strain and directional strain along this transverse half profile are equal and are depicted as EX in Figure 2.

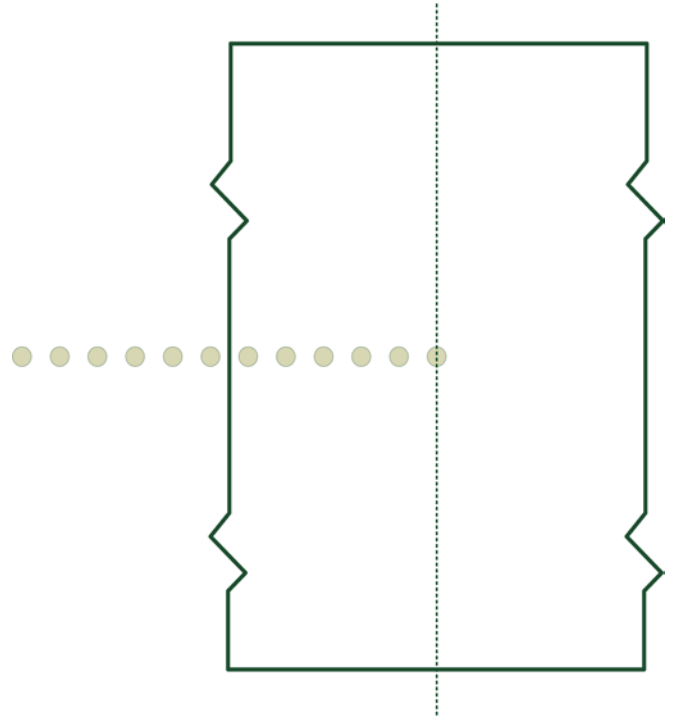


Figure 1. Plan view of example case study - rectangular extraction and transverse monitoring line.

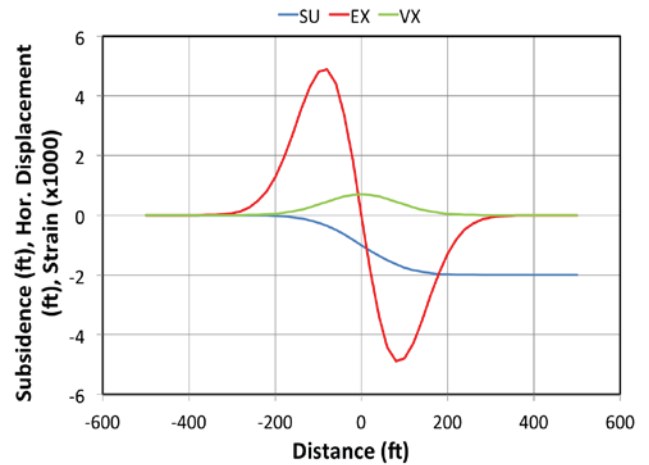


Figure 2. Calculated subsidence (SU), Horizontal Strain (EX), and Horizontal Displacement (VX) along the transverse monitoring line.

The axial strain is calculated using strain rotation for each monitoring point, as shown schematically in Figure 3. Axial strain values are calculated for each individual point. The directional vector is calculated by connecting each pair of consecutive points with a straight line. For the last point in the sequence, the directional vector calculated for the previous point may be applied.

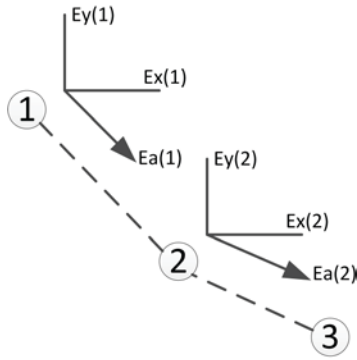


Figure 3. Axial strain along monitoring line points (points are shown in plan view).

GROUND STRAIN CALCULATIONS ON AN IRREGULAR SURFACE FOR POINTS ON A PROFILE

Ground deformation monitoring line points rarely lie on a horizontal surface. Thus, when the monitoring profile covers an irregular surface, the projection of the horizontal displacement (and the horizontal strain) on the ground surface varies and depends on the shape of the surface. Assuming that only the pre-mining elevations of the profile points are known, ground strain can be calculated at every point (P_i) of the profile as follows (Figure 4):

Temporary points A and B should be defined at mid-distance between points $P(i-1)$, $P(i)$ and $P(i)$ and $P(i+1)$ using pre-mining coordinate information ($x(i)$, $y(i)$, $z(i)$, etc.) for points $P(i-1)$, $P(i)$, and $P(i+1)$, as shown in Figure 4. The distance between A and B is calculated as $w = \|A - B\|$

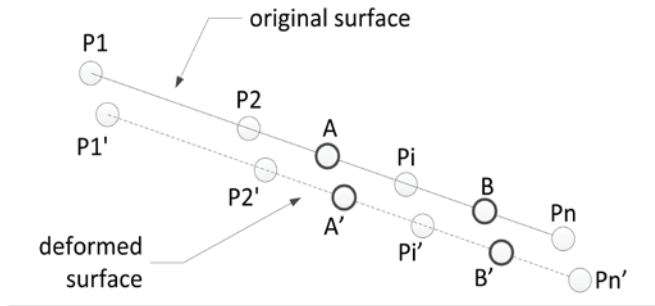


Figure 4. Surface monitoring points on a sloping terrain (points are shown in cross section view).

$$A = \left[\frac{x(i) + x(i-1)}{2}, \frac{y(i) + y(i-1)}{2}, \frac{z(i) + z(i-1)}{2} \right] \quad (5)$$

$$B = \left[\frac{x(i) + x(i+1)}{2}, \frac{y(i) + y(i+1)}{2}, \frac{z(i) + z(i+1)}{2} \right] \quad (6)$$

The procedure is repeated for points A' and B' at the deformed surface using the coordinates at the deformed state (i.e., $x(i) +$

$\delta x(i)$, $y(i) + \delta y(i)$, $z(i) + \delta z(i)$). The distance between A' and B' is then calculated as $w' = \|A' - B'\|$. Ground strain EG at point (i) is then calculated as follows:

$$EG(i) = \frac{w' - w}{w} = \frac{w'}{w} - 1 \quad (7)$$

Another approach commonly employed by surveyors monitoring surface points is to calculate strains for the midpoints of the linear segments between points. That means that although horizontal and vertical displacements are available for points $P(i-1)$, $P(i)$, $P(i+1)$, horizontal and ground strains are calculated for points A and B . With the approach highlighted above, strains are available for the actual monitoring points.

Figure 5 compares ground strain and horizontal strain calculated for the example shown in Figure 1, where the surface is sloping to the east by five degrees (or 5.6%). It is easily determined that the tensile strains increase on the uphill side while they tend to decrease when moving downhill. This is a well-known behavior and has already been documented by case studies. For example, Khair, Quinn, and Chaffins (1987) postulate that measured strains indicate that there is a significant increase in ground tensile strain when mining in a direction that approaches uphill topography. These amounts decrease when approaching the downhill slope side.

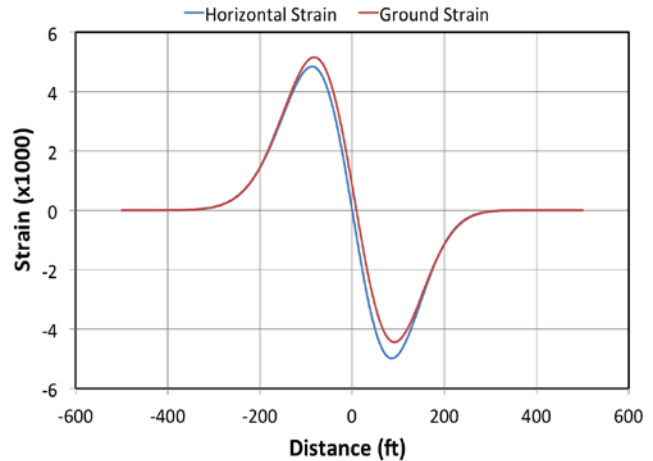


Figure 5. Comparison of ground and horizontal strain for a 5 degree sloping surface (surface point spacing = 20 ft).

Figure 6 compares the calculated ground strain for the same example where each line corresponds to different point spacing for the surface points, $P(i)$. It can be easily determined that for coarse point spacing, an error for the maximum calculated ground strain value up to 20 or 25% may be introduced.

In fact, the above adjustments can be applied to strain calculations of relatively uniform surfaces, which are deformed by underground mining operations. In the following example, the monitoring line over a longwall panel in southwest Pennsylvania was modeled, as shown in Figure 7. Calculations were performed using the Surface Deformation Prediction System software package

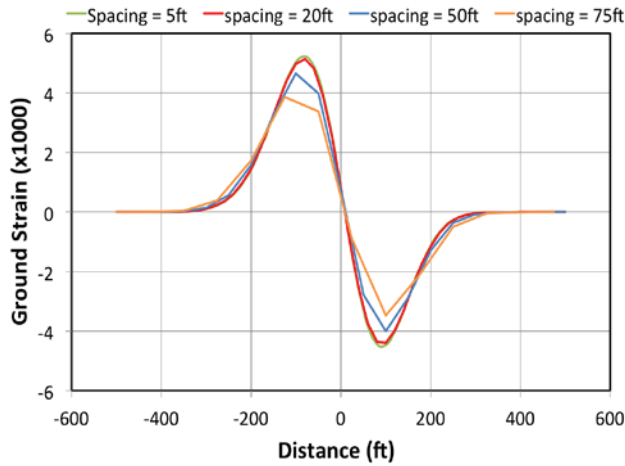


Figure 6. Comparison of ground strain curves on a 5 degree sloping surface for different surface point spacing values ranging from 5 to 75 ft.

(SDPS) (Agioutantis and Karmis, 2013a), assuming default parameters for the Appalachian coalfields and a hardrock amount of 27%. Figure 8 compares measured values, values for axial horizontal strain and values for ground strain, calculated, shown in equation 7. As can be easily observed, the axial strain calculation over predicts the measured strain profile.

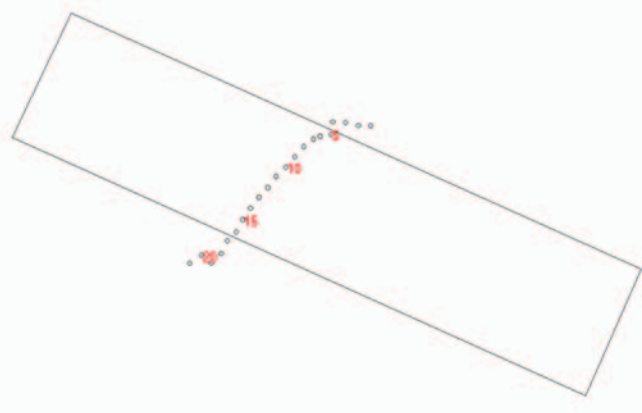


Figure 7. Layout of monitoring line over a longwall panel in Pennsylvania.

GROUND STRAIN CALCULATIONS WHEN SURFACE TOPOGRAPHY INFORMATION IS AVAILABLE

The surface topography over undermined areas is usually available as contour lines of different density. Using commercially available software, such as Carlson Software or Surfer, it is easy to convert these contour lines to grid files with appropriate density. For example, if topography is represented by 20-ft contour lines, the grid cell size of the surface grid should be on the order of 20 ft. Although gridding software may calculate grids of squares or rectangles, it is recommended to develop grids with square cells.

After developing the surface topography grid, ground deformations may be calculated for each grid point. Figure 9a shows a four square cell arrangement comprising nodes

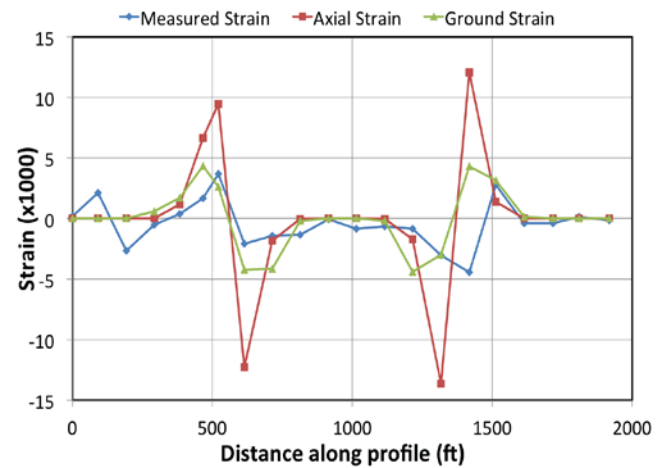


Figure 8. Comparison of measured, axial and ground strains along monitoring line profile.

A,B,C,D,E,F,G,H,X. In typical surface deformation calculations and when the surface points are within the influence area of the extracted panel, each of these nodes will undergo vertical and horizontal movements (Figure 9b). Using equation (4), the horizontal strain at each point may be easily calculated. However, ground strain cannot be calculated analytically, unless the deformed surface comprising these four square cells is defined. Alternatively, the maximum ground strain experienced at the center point (X) may be calculated using the following methodology.

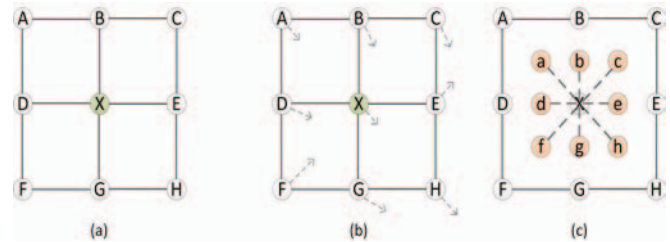


Figure 9. Calculating ground strain for point (X) using temporary points.

Temporary surface points a,b,c,d,e,f,g,h should be considered, which lie at the midpoint of the vertical, horizontal and diagonal connectors between the nodes around point X (Figure 9c). The ground strain due to compression or tension can be calculated along lines a-h, b-g, c-f, d-e using equations similar to (5) and (6) and selecting the maximum value as the maximum ground strain. Using the same procedure, ground strain at different directions may be calculated. Thus, the maximum ground strain can be calculated for point X based on the calculation of horizontal and vertical movements at this and neighboring points and information regarding the topography of the terrain.

INFLUENCE OF SLOPING TERRAIN ON GROUND STRAIN CALCULATIONS

Figure 10 shows a second example of a mined panel under sloping terrain. A perfect hill has been simulated over the right rib of the panel and about the middle lengthwise. The base elevation of the hill is 400 ft, and the apex is 600 ft above the mined seam.

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The surface over all of the remaining undermined area is set to an elevation of 400 ft. All other parameters are the same as in the previous example. Maximum horizontal strain and maximum ground strain were calculated over the entire area (Figures 11 and 12). Contour line magnitudes are lower around the hill area in the case of the ground strain plots (Figure 12).

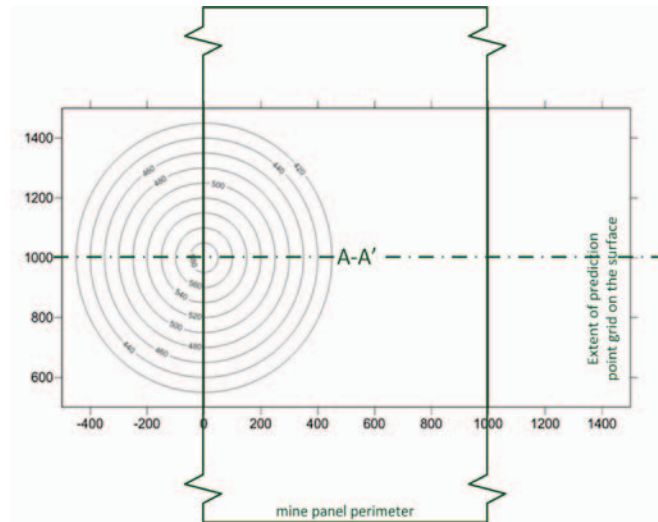


Figure 10. Synthetic mine plan and surface topography representing sloping terrain.

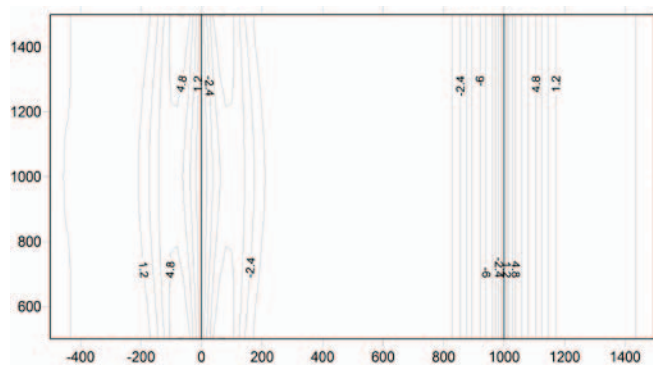


Figure 11. Maximum horizontal strain contours.

In addition, a cross-section comparing horizontal strain and ground strain along line A-A' is plotted in Figure 13. There is definitely a reduction in ground strain magnitudes over the hilly area. In particular, strains may even reverse (i.e., tensile to compressive to tensile and vice versa) under sudden changes of topography.

CONCLUSIONS

This paper discusses strain calculations for different topologies of surface points (flat surfaces, flat monitoring lines, monitoring lines on irregular terrain, sloping grids, etc.). A comparison between axial horizontal strain and axial ground strain for an actual case study shows that ground strain predictions will more accurately reflect the measured profile, while axial horizontal strain calculations can over predict the strain regime. The comparison between horizontal strain and ground strain shows that ground

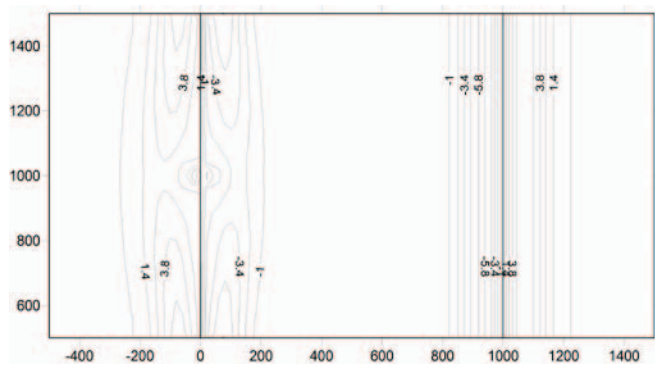


Figure 12. Maximum ground strain contours.

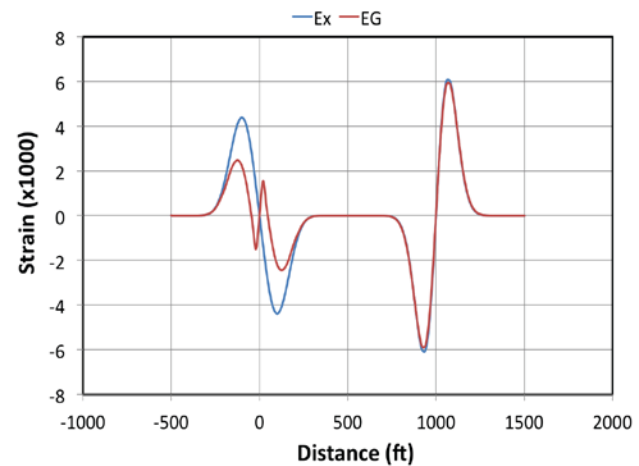


Figure 13. Comparison of horizontal strain (Ex) and ground strain (EG) along line A-A';.

strain is greatly affected by the topography of the terrain and that, in some cases, local reversals of strain conditions may occur.

All calculation tools and procedures have been implemented into the Influence function formulation program of the SDPS software package (Agioutantis and Karmis, 2013a). Work is underway to validate the procedure introduced for ground strain adjustment based on a grid of points representing the terrain around the monitoring line. Also, further research is needed to calculate ground strain under dynamic conditions (i.e., as a function of panel advance).

ACKNOWLEDGEMENTS

This study was sponsored by the Appalachian Research Initiative for Environmental Science (ARIES). ARIES is an industrial affiliates program at Virginia Tech, supported by members that include companies in the energy sector. The research under ARIES is conducted by independent researchers in accordance with the policies on scientific integrity of their institutions. The views, opinions and recommendations expressed herein are solely those of the authors and do not imply any endorsement by ARIES employees, other ARIES-affiliated researchers or industrial members. Information about ARIES can be found at <http://www.energy.vt.edu/ARIES>.

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Slope Stability at Open Cut Mining Due to Subsidence on Old Underground Mining Area

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ABSTRACT

An underground mine using room and pillar and retreat mining methods was developed during the 1950's. In 2009 an open cast mine was opened in the same reserve with the objective of mining five coal seams including the one previously mined. Recently, a series of slope failures were detected at the open cast mine. The surface mine map was overlain on the underground mine map, and it was observed that the slope failures were located above a retreat mined area where the overburden had caved and the ground subsided. The deformation and displacement attributable to subsidence caused a loss of strength in the overburden strata. The objective of this paper is to study the causes of slope sliding (or failure) due to the ground subsidence of rock layers above old-mined out areas. A map was created to overlay the two mines -- the abandoned underground mine and the active surface mining. Two- and three-dimension numerical modeling was conducted to simulate the slope behavior and to verify whether the mine subsidence was the cause of slope instability. This study indicated that the subsided rock layers were the cause of slope instability and sliding.

INTRODUCTION

An underground coal mine using room and pillar retreat mining was developed during the 1950's. Figure 1 shows the layout of the underground workings, showing the mining sections and mains. Some mining sections were completely mined out, which caused roof caving and subsidence within the overburden. At the time the mine was closed some sections were not completely retreat mined. The abandoned mine was flooded and the entries are now inundated. The average overburden of the coal seams is 50 meters. The mining sections average 65m wide and 1.5 to 2m high. Fender pillars were left between the caved area and the main entries to guarantee that the mains could be kept open for ventilation and access to other mining areas.

In 2009 an open-cast mine was started in the same reserves previously mined by underground methods. The objective is to mine five coal seams (Figure 2), including the previously mined seam. The previously mined coal seam is the M1/M2 coal seam. It is average 2.6m thick but only 1.5 to 2m had been mined underground. Because the coal is more stable and less sensitive to

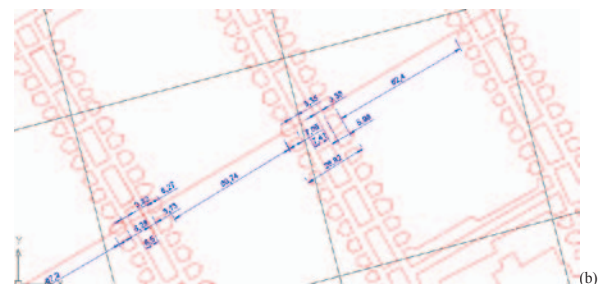
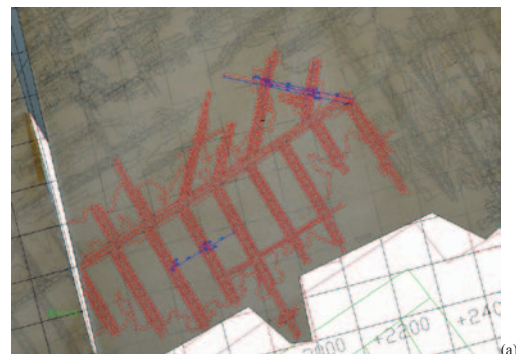


Figure 1. Layout of the old underground mining area; (a) and (b) showing the mining sections and mains and (c) shows the mining sequence.

moisture than siltstone in the roof, they usually leave top ahead of the coal to isolate the siltstone from moisture, and also because the excavation was done with hand tools. This limited the mining height 2m.

Prior to surface mining the water impounded within the abandoned mine workings was pumped out to avoid flooding of the active pits during surface mining. The surface mining company was concerned about pillar failure and roof collapses caused by dewatering the abandoned mine workings. It was expected that the impounded water imposed hydraulic or hydrostatic pressure on the pillar and roof. In addition the saturated rock layers may be weaker. During pumping, a camera was lowered into the mine

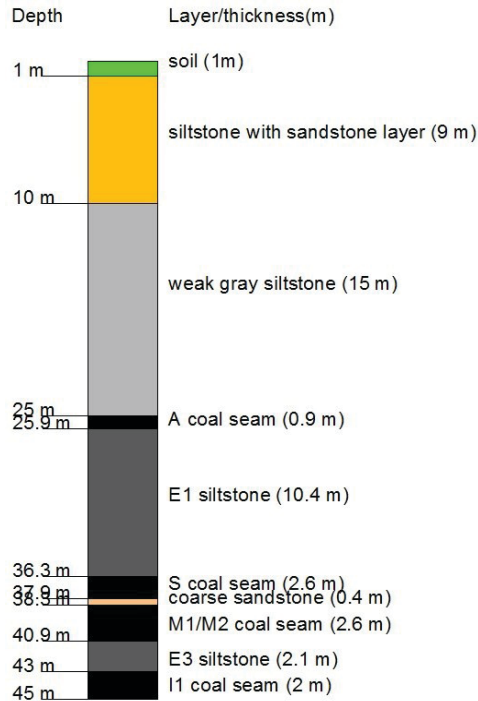


Figure 2 – Geology column for B3 Mine.

Figure 2. Geology column indicating the coal seam that was mined in the past.

entries via a borehole. It was observed that few roof collapses and no pillar failures occurred as a result of dewatering.

To keep abreast of roof collapse and subsidence problems, entries and surface were monitored during the pumping process. The collars of the dewatering wells and selected surface points were monitored for vertical displacement. The entries were also monitored by image recording to check if the roof and pillars were stable. No surface movement or underground failures were observed.

Since the pumping operation was deemed, no subsidence or roof or pillar collapse were detected. The focus was then moved to slope stability problems initiated by surface excavation of the overburden above the old underground workings. Where the underground mine sections were completely mined, the rock above the M1/M2 coal seam is caved and subsided due to the retreat mining. The deformation and displacement associated with the mine subsidence resulted in a loss of strength within the overburden strata.

The mine subsidence was observed on the slope face. Figures 3a and 3b illustrate mine subsidence on the highwall of the slope. It is clear that the strata immediately above the M1/M2 coal seam are caved. However, the layers above the immediate roof, the evidence of subsidence is not clear or notable. This is because the section width is narrow. Thus the amount of subsidence for the sections is

similarly small. As observed on figure 3a and 3b, the subsidence range 1.1 to 1.2m.

A series of slope sliding and failure was observed in the open-cast mine. The failure and slides range between local failure and slips to large scale slides (Figures 3a, 3b and 3c). The size of the sliding range from 2-3m to 10m, it means complete slope failure (Figure 3c).

SURVEY MAP

The surface mine map was overlain on the underground mine map (Figure 4). It was noted that the slope failure is located in an area where the underground sections had been retreat mined, resulting in overburden subsidence. The objective is to determine whether the slope failure is attributable to the loss of rock strength due to the mine subsidence.

SUBSIDENCE CALCULATION

Since the slope face survey shows that the caving of the sections occurred where retreat operations had been performed and that the roof failure occurred for single entries, the amount of subsidence is different depending the width of the panel or the area that was retreated. Therefore, the subsidence was simulated for different panel widths.

Subsidence in shallow coal seams had been studied by Peng et al., (1987). Mine subsidence was initially estimated using Dr. Peng's theory (1992 and 2008). The results showed very high subsidence (more than 1m), but the slope face survey showed that the subsidence was less. The equations proposed by Peng et al (1995).

Subsidence simulation was done with 2D numerical modeling using FLAC – Fast Lagrangian of Continua, Version 5, from Itasca Inc. Figure 5 shows the 2D numerical model. The numerical simulation followed the same geometry for the sections and the same geology sequence. The model sequence is:

1. Model building with boundary conditions, considering gravitational stress field and plain strain model
2. Excavation of the entries
3. Panel retreat considering different widths, and measuring the subsidence at the surface for panel widths, also considering full retreat for two panels.

The mechanical properties for each layer were estimated applying the Hoek-Brown failure criteria (Hoek & Brown, 1997), and the gravitational stress was principal. The horizontal/vertical stress ratio was 1. Table 1 shows the material properties for the rock layers. These same properties were used for slope simulation.

Figure 6 shows the subsidence profiles, and Table 2 shows the subsidence at the surface for each panel width. As expected the maximum subsidence was in the 64m wide panel. The subsidence for 30.5m was 6.5 centimeters, an amount that is sometimes imperceptible to the naked eye.

Table 1. Material properties for each rock layer.

Material	Density [kN/m ³]	Young's Modulus [kPa]	Poisson's Ratio	Failure Criteria	GSI	mi	Friction Angle [degree]	Cohesion [kPa]	Rock Mass Str [kPa]	s3max [kPa]	Intact Str [kPa]	mb	s	a
Soil	19	10000	0.4	Mohr-Coulomb			15	10						
Siltstone	26	500000	0.3	Hoek-Brown Generalizado	35	7	26.8	153.0	1232.9	1032.75	22800	0.197	0.0001	0.516
Coal	21	500000	0.3	Hoek-Brown Generalizado	40	7	31.0	200.0	1867.8	882.72	29020	0.259	0.0002	0.511
Weak Sandstone	26	500000	0.3	Hoek-Brown Generalizado	45	17	32.7	196.0	1175.0	1028.28	10000	0.828	0.0003	0.508
Corse Sandstone	26	500000	0.25	Hoek-Brown Generalizado	45	21	40.7	290.0	2979.9	1118.12	22800	1.023	0.0003	0.508
Sandstone Bas	26	500000	0.25	Hoek-Brown Generalizado	50	17	48.4	465.0	8210.5	1224.91	60000	1.090	0.0007	0.506

Table 2. Maximum subsidence for different panel widths.

Panel Width (m)	Subsidence (cm)
61.00	28.00
30.50	6.50
22.80	3.80
15.20	1.27
7.60	0.42

NUMERICAL SIMULATION OF THE SLOPE STABILITY

Slope stability is a very important issue for surface coal mining (Hoek and Bray, 1973; Wyllie and Mah, 2004). The issue is that the slope must be both safe and economically viable. The ideal result is to have a minimal slope safety factor (near to 1.2 or 1.3) because the slope is temporary with an average exposure time of approximately six months.

In this case study, the slope failure could be caused by the change of material strength due to caving and subsidence of the rock layers above the coal seam (M1/M2 coal seam). Once the rock layers caved, they change the material properties and loose strength because they turned broken or the movement force to appear some cracks. The broken or cracked rock layers reduce the strength parameters. To check this hypothesis, a 3D numerical simulation of the slope was built. Figure 7 shows the geometry of the slope and the 3D geometry model. The numerical model was developed using Plaxis 3D Tunnel Version 2.4.

The sequence simulation is as follows:

1. Build the 3D model, considering the same geology sequence, material properties and boundary conditions for the subsidence simulation.
2. Excavate the entries and mining section in retreat to simulate the subsidence.
3. Excavate the slope.

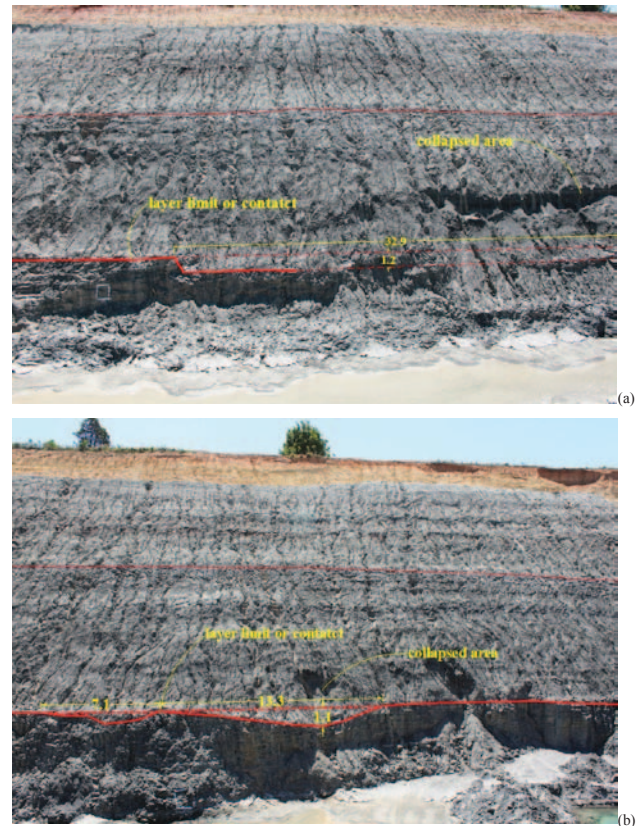


Figure 3. (A) and (b) indicate a subsidence survey on wall slope at surface mining area; (c) huge slope failure.

4. Build a slope model considering the same geometry without any underground excavation to calculate the safety factor for the slope.

The safety factor of the slope was calculated using 3D numerical modeling, considering the tools in the software that reduces the cohesion/friction angle by a ratio till the slope start to fail. The

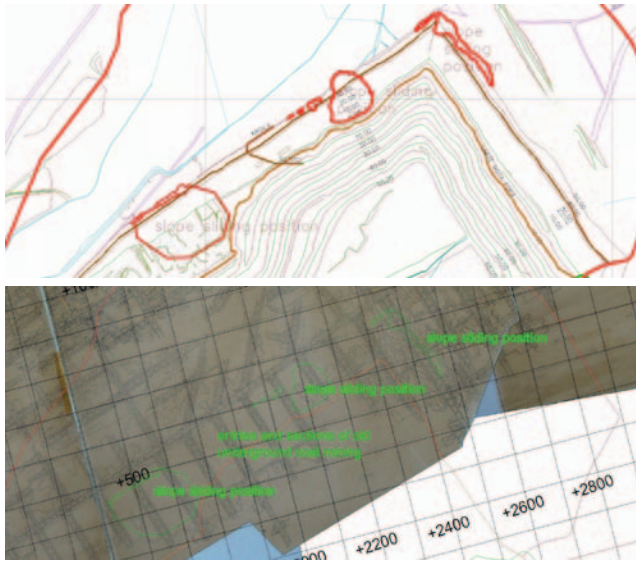


Figure 4. Slope sliding location and the overlay of the surface and underground mine maps.

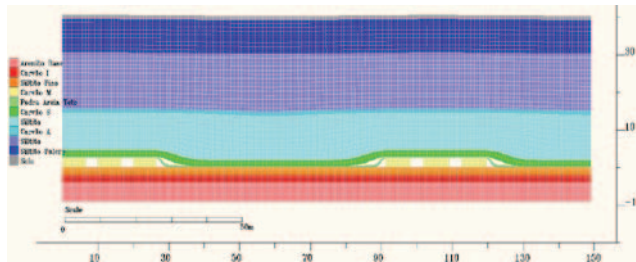


Figure 5. Numerical simulation of subsidence on old underground coal mining area.

initial parameters of cohesion and friction angle (Table 1) are reduced by some ratio until the slope begins to fail.

$C_n = C_0/p$, where C_n is the new cohesion, C_0 is the initial inputted cohesion, p is the reduction factor. The same approach is applied to friction angle.

The ratio will depend if the inputted parameters conduce the slope to failure or not. The strength reduction factor could be considered as the safety factor of the slope. For this case study, the safety factor was 1.83. The maximum displacement for the initial strength parameters is 44 millimeters.

In the model where the entries and retreated section was considered, the failure of the slope occurred exactly where the subsidence occurred. Figures 8 and 9 show the total strain and total displacement at the slope face above the retreated section of the old underground workings. The safety factor (or the reduction factor)

was less than one. Therefore the slope could fail at the area where there is subsidence effect on the quality of the material properties.

The displacements of the rock layers where the there was retreat mining reached 1.5 to 2m. Comparing the survey measurements (Figure 3) and the results of the model, it could be said that the results of the numerical model is quite similar to the survey mapping.

Therefore, when the rock layers received any deformation caused by the effects of the subsidence from underground mining, these effects could change the material behavior and could cause slope instability.

CONCLUSIONS

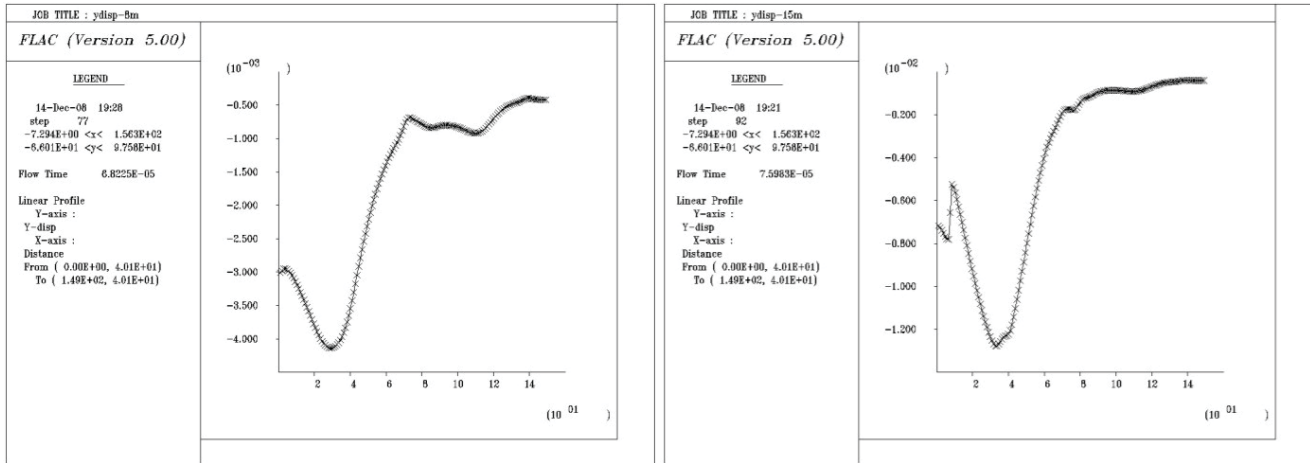
In conclusion, this study indicated that the subside rock layers can be the cause of slope instability and sliding. The geometry of the slope is considered safe if there is no influence from the effects of the subsidence.

The next steps for this research is improve the numerical model by calibration and also change the material properties of the rock layers and areas where there were subsidence effect, considering some material properties like gob material.

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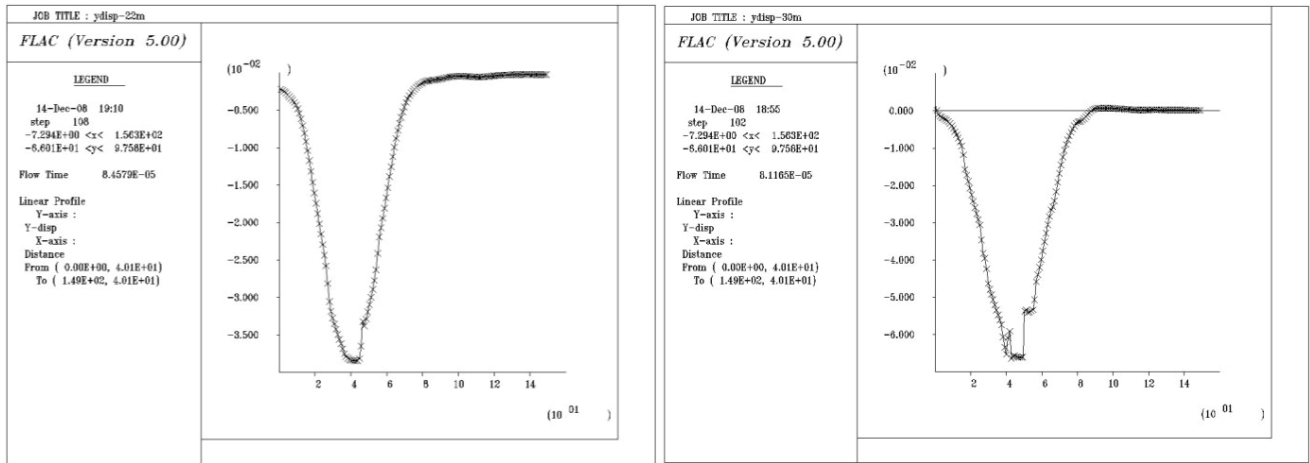
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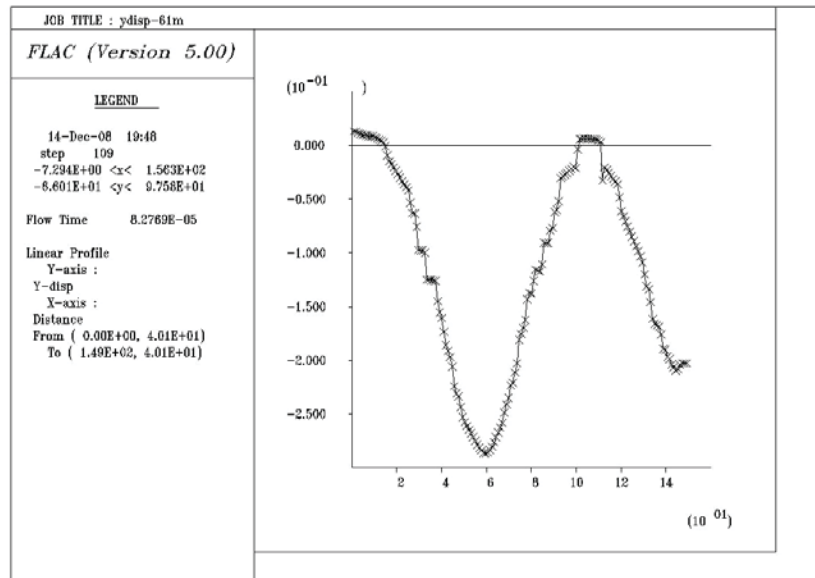
(a)

(b)



(c)

(d)



(e)

Figure 6. Subsidence profile for different panel widths, considering the widths (a) 7.6m; (b) 15.2m; (c) 22.8m; (d) 30.5m; and (e) 61.0m.

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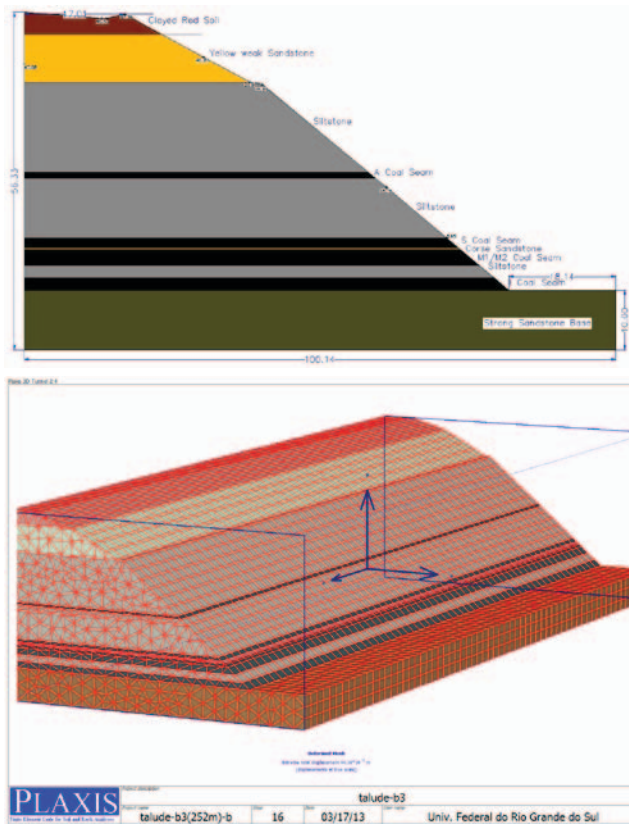


Figure 7. Geometry model for 3D slope simulation using Plaxis 3D Tunnel.

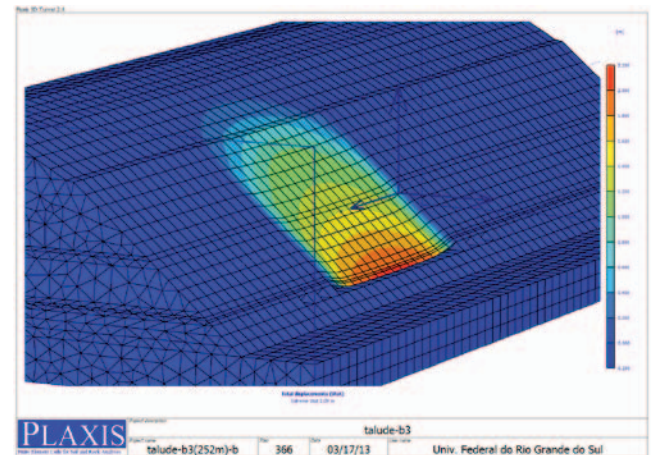


Figure 9. Total displacement of the slope face caused by the influence of the subsidence effects.

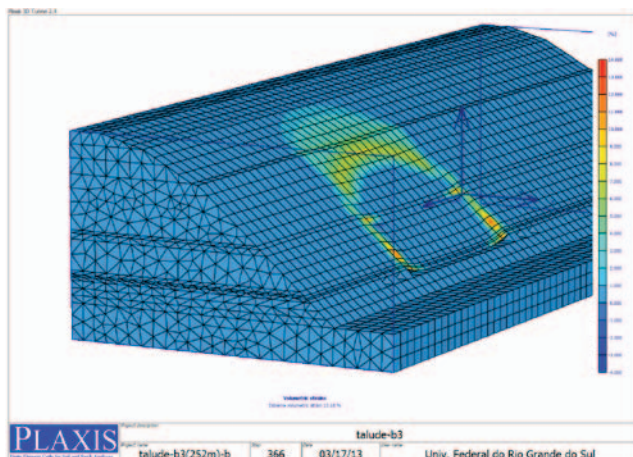


Figure 8. Total strain of the slope face caused by the influence of the subsidence effects.

Appropriate Mining System for Residual Coal Around End-slopes in Chinese Surface Coal Mines

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ABSTRACT

A great deal of coal will be left under the final end-walls in Chinese surface coal mines because of lower slope angle in shovel-truck mining systems and larger mining depths. In traditional surface mining systems, the coal under the end-walls will be buried by the inner dumping site, and these coal resources are generally abandoned. Considering these situations, the application of end-wall mining systems was considered in order to recover the residual coal around the end-wall. This mining system can improve economic benefits by exploiting haulage and ventilation roadways from end-walls and utilizing the existing transportation systems. In order to develop the appropriate pillar design methodology for the end-wall mining system, several empirical formulas and methods were discussed based on the formulas developed so far and the mechanics properties of coal and rocks in the Chinese coal mine. From the results of a series of theoretical and numerical analyses, it can be concluded that the coal pillar width calculated by the Mark-Bieniawski formula may serve as an important reference for pillar design in end-wall mining systems, and the end-wall mining system can increase the coal recovery from residual coal resources around highwall and reduce the effect of underground mining operation on slope stability.

INTRODUCTION

Surface mining is always playing a considerable role in the world's coal mining; it has many advantages such as large scale and production, low cost, simple mining technology and operating safety. The surface mining method has been adopted in major coal production countries such as the U.S., Australia, Indonesia, India, Russia and South Africa. According to statistical data, about 40% of the world's coal is produced from surface mines (Sinha and Singh, 2009). In the last decade, the surface mining technology in China also has been dramatically developed, and the number of surface mines with over 10Mt of production capacity is more than 20. Coal production from surface mines has reached 10.8% of the total amount of raw coal in 2011 as shown in Figure 1.

However, there are a great deal of differences for occurrence features of coal seams between China and other major coal production countries (Cai, et al., 2008). According to the data from the third Chinese national coal field prediction(1992-1997),

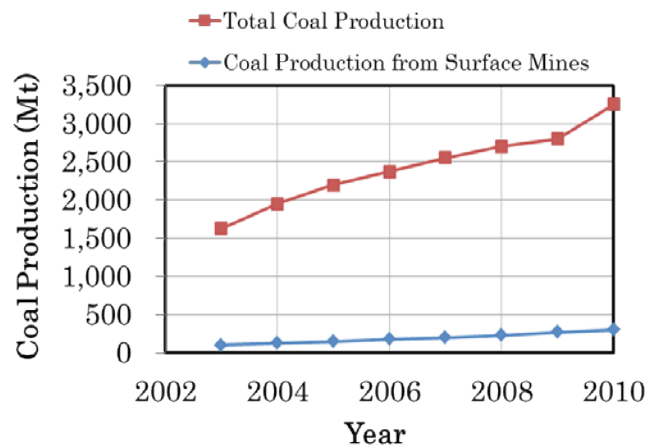


Figure 1. Coal production of surface coal mines in China.

the coal resources buried in depths less than 600m only cover about 20% of total reserves in China. As a result, the amounts of coal which can be extracted by surface mining in the country are very limited.

There are two main features in Chinese surface coal mines. One is the thick cover, and another is the application of the shovel-truck mining system. Due to higher stripping ratios under the thick cover, a large amount of coal is left under the end-walls in these mines. Meanwhile, in most of large-scale Chinese surface coal mines, coal seams are almost flat and the shovel-truck mining system is the most common selection.

For this system, as shown in Figure 2, and as the haulage roads of trucks are generally arranged in the end-walls, the slope angle of end-walls are designed in gentle. This is especially the case in some regions with poor geological conditions, as the angle of the final end-wall will be much lower. Hence, more coal is abandoned around these end-walls. In the current mining systems, the coal under the end-walls will be buried by the inner dumping site and these coal resources are normally abandoned, wasted in vain.



Figure 2. Haulage road in the end-walls.

To recover coal remnants around the end-walls, the application of the underground mining system is considered by excavating adits from the end-walls. In longwall underground mining systems, the full caving method is used to manage the coal roof and it will cause significant ground behavior and subsidence. Hence, strip mining is usually applied in some regions where only minor ground displacement is permissible (Guo, et al., 2003 and 2004).

In this mining system, the coal seam is divided into some regular strips; one strip is mined and the next strip is reserved. The reserved strips can support the load of the overburdened strata; as a result, the ground behavior due to seam extraction may be controlled. Thus, the remnant coal can be extracted without any obvious impacts on the surrounding area. As strip mining is an effective method to control the overlying strata and ground subsidence, it is usually used for extracting coal seams under buildings and railway lines. The application of the strip mining system for coal resources abandoned around the end-wall was considered in this research.

This paper describes the end-wall mining system by using the strip mining system and then discusses the applicability and the suitable design methodology for end-wall mining systems by means of theoretical and numerical analyses

OVERVIEW OF ATB COAL MINE

The ATB coal mine was chosen as the target mine in this research. The ATB open pit coal mine is located in northern part of China (as shown in Figure 3). There are three flat coal seam layers available for exploitation: the No. 4, the No. 9 and the No. 11. The averages of their seam thickness are 8m, 13m and 5m, respectively. Both the No. 9 and No. 11 coal seams contain high-sulfur coal, and the No. 4 contains low-sulfur. To meet the high demand of the low-sulfur coal for export as a cleaned coal, the No. 4 coal seam is used for coal blending.

The production of No. 4 coal is not enough to achieve the design yield of cleaned coal; on the other hand, plenty of No. 4 coal existing around the north and south end-slopes cannot be mined and is left as shown in Figure 4. Hence, it is one of the solutions to extract the end-slopes remnants by using the strip mining system with existing production.



Figure 3. Location of ATB coal mine, China.



Figure 4. Working pit and end-wall at AKB coal mine, China.

END-WALL MINING SYSTEM

To recover the residual coal of final end-walls, the interval haulage and ventilation adits are first driven from the end-wall. After the haulage and ventilation roadways are developed, exploitation roadways will be driven between the ventilation and haulage roadway, and are supported by flange beams and wire netting on the roof. After that, two working faces are laid out towards the left and right directions from the position of the ventilation roadway and a retreat mining manner is carried out (see Figure 5).

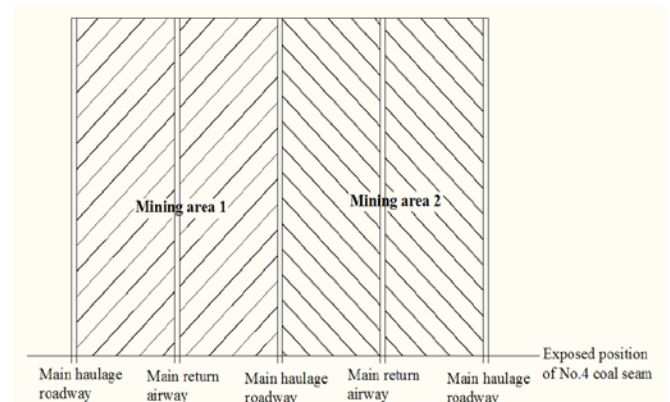


Figure 5. Mining area of strip mining system.

While mining, if the coal seam is very thick, it should be divided into top and bottom slices where the thickness of each slice is about 4m to 5m. In addition, each of the slices is subdivided into upper and lower slices. For each slice, the lower slice is extracted first by blasting and then the upper one is extracted by caving or blasting (see Figure 6).

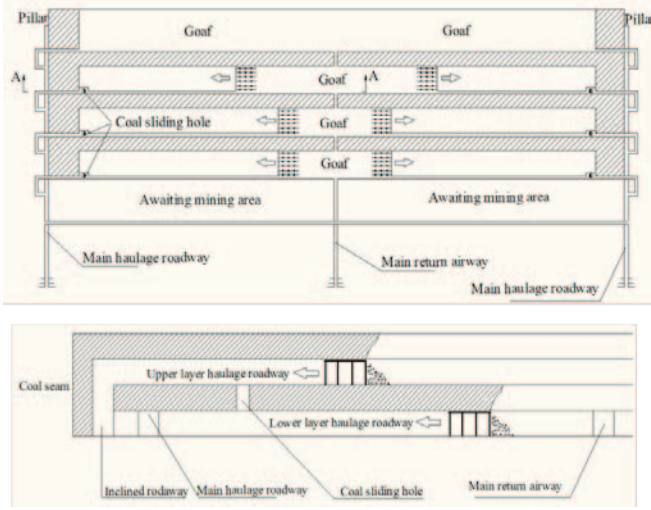


Figure 6. Strip mining system in end slopes.

The working face is supported by single hydraulic props and caps. After the lower slice has been blasted, the single hydraulic props are needed immediately to support the exposed roof. During this time, the cutting coal is transported by the scraper conveyor and, at the same time, as drilling and filling works are being carried out in the upper slice above the goaf, one row of single hydraulic props near the goaf is removed while the coal is blasted and loaded on the scraper conveyor. One row of temporary mine timbers near the coal face are then removed and the scraper conveyor is moved towards the coal face. This cycle is repeated.

COAL PILLAR DESIGN

Coal pillar design is one of the key subjects which has received attention from researchers the world over for several decades, particularly from the ground control point of view. Currently, there is no universally accepted formula or methodology to design coal pillars.

In general, there are three major steps in pillar design. The first step is the determination of the expected load on the pillar. The second step is the determination of pillar strength. And the last step is the determination of the safety factor.

Ground control issues are the most important factors for application of the end-wall mining system, which affects the whole surface mining operation. The whole final end-wall must be stable before it is buried by the inner dumping site. Meanwhile, the end-wall mining operation should not have an impact and/or damage on the haulage road benches, such as a failure or slide in order to maintain the transport system. As is well known, if the coal pillar width is designed to be too wide, even though the end-wall can be maintained in stable condition, coal is left around the end-wall, reducing coal recovery. A. On the other hand, if the coal pillar

width is not designed reasonably, serious end-wall failure may occur and disturb the regular operation of the whole surface mine.

Coal pillar strength

There are many empirical formulas for evaluation of coal pillar strengths developed from various coal fields around the world. A general agreement among researchers is that coal pillar strength increases with the increase of coal pillar width-to-height ratio, and it can be expressed by the two general types of expressions linear and exponential.

(1) Linear type

Obert-Duvall/Wang (Obert and Duvall, 1967)

$$\sigma_s = \sigma_i (0.778 + 0.222 \frac{W_p}{H_p}) \quad (1)$$

Bieniawski (Bieniawski and Van, 1975)

$$\sigma_s = \sigma_i (0.64 + 0.36 \frac{W_p}{H_p}) \quad (2)$$

Mark-Bieniawski (Mark, et al., 1988)

$$\sigma_s = \sigma_i (0.64 + 0.54 \frac{W_p}{H_p}) \quad (3)$$

(2) Exponential type

Holland (Holland and Gaddy, 1964)

$$\sigma_s = \sigma_i \sqrt{\frac{W_p}{H_p}} \quad (4)$$

Salamon-Munro (Salamon and Munro, 1967)

$$\sigma_s = 7.176 \cdot \frac{W_p^{0.46}}{H_p^{0.66}} \quad (5)$$

where σ_s is coal pillar strength, MPa, σ_i is in-situ critical cubical coal strength, MPa, W_p is the coal pillar width, m, H_p is the coal pillar height, m.

Coal pillar stress

The mining width is not as large in the end-wall mining system. Only the immediate roof rock may be caving while the main roof may not be. Most of the coal pillars will have regular dimensions in the whole mining area. Based on these results, it is assumed that all the coal pillars will bear the load of the upper rock layers in the

tributary area theory, and two halves of the rock mass above the adjacent mining openings are acting on the coal pillar as shown in Figure 5. The average load on the coal pillar is:

$$\sigma_L = \frac{(W_p + \frac{W_m}{2} + \frac{W_m}{2})\gamma H}{W_p} = \frac{(W_p + W_m)\gamma H}{W_p} \quad (6)$$

where σ_L is the average load on the coal pillar, MPa, W_m is the mining opening width (m); W_p is the coal pillar width, m, γ is the average bulk density of the cover rock, MN/m³, H is the thickness of the cover rock, m.

Pillar width

It is assumed that the safety factor is f , then the coal pillar strength and load on coal pillar will have following relations:

$$\sigma_L = f \cdot \sigma_L \quad (7)$$

Based on the previous discussion results, the coal pillar width calculated by all the empirical formulas can be concluded as follows:

Obert-Duvall/Wang

$$W_p = \frac{(f\gamma H - 0.778\sigma_t)H_p}{0.444\sigma_t} + \sqrt{\frac{(0.778\sigma_t - f\gamma H)^2 H_p^2 + 0.888\sigma_t H_p W_m f \lambda H}{0.444\sigma_t}} \quad (8)$$

Bieniawski

$$W_p = \frac{(f\gamma H - 0.64\sigma_t)H_p}{0.72\sigma_t} + \sqrt{\frac{(0.64\sigma_t - f\gamma H)^2 H_p^2 + 1.44\sigma_t H_p W_m f \lambda H}{0.72\sigma_t}} \quad (9)$$

Mark-Bieniawski

$$W_p = \frac{(f\gamma H - 0.64\sigma_t)H_p}{1.08\sigma_t} + \sqrt{\frac{(0.64\sigma_t - f\gamma H)^2 H_p^2 + 2.16\sigma_t H_p W_m f \lambda H}{1.08\sigma_t}} \quad (10)$$

Holland

$$\sigma_t W_p^{1.5} - \sqrt{H_p} \gamma H f (W_p + W_m) = 0 \quad (11)$$

Salamon-Munro

$$7.176 W_p^{1.46} / H_p^{0.66} - \gamma H f (W_p + W_m) = 0 \quad (12)$$

As a result, the results of coal pillars in the studied coal mine are shown in Table 1.

Table 1. Coal pillar width calculated by the empirical formula.

Empirical Formulas	Panel Width (m)	Pillar Width (m)
Obert-Dwvall/wang	40	22
Bieniawski		19
Mark-Bieniawski		17
Holland		19
Holland-Gaddy		12
Salamon-Munro		28

Mining Width

The mining width is related to the mining depth. Practice shows that lumpy displacements will not occur on the surface if the mined width meets the following condition (Hu, 1995):

$$\frac{H}{10} \leq B_m \leq \frac{H}{3} \quad (13)$$

where B_m is mining width, m; H is the mining depth, m.

Because No.4 coal seam is about 120m from the ground in the studied mine, and according to equation (13), the mining width of coal seam is: $2.0 \text{ m} \leq B_m \leq 40 \text{ m}$.

NUMERICAL ANALYSIS

In order to further discuss the stabilities of coal pillars and end-wall slopes under the mentioned pillar design methodology in the end-wall mining system, numerical simulations were carried out with the two-dimensional finite element method (FEM) software Phase2. The analysis model used in this research is shown in Figure 7. This model represents the geological conditions of end-wall slope in ATB open pit coal mine. This model contains thirteen different layers. The average height of bench is 15m, and the slope angles of loess and rock bench are 55 degree and 70 degree, respectively. There are four haul benches and their width is 35-40m. The overall end-wall slope height, width and angle are 198m, 312m and 32 degree, respectively.

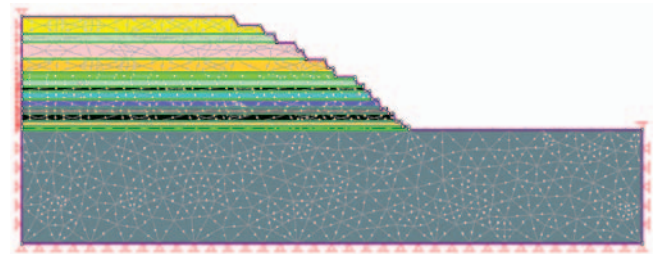


Figure 7. Finite element model.

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In this model, the width and height of mining panel in the No.4 coal seam are 40m and 8m, respectively. The different widths of pillars between panels were set according to the results of theoretical analysis as shown in Table 1. The width of boundary pillars from the end-slope to the adjacent mining panel was variable. The effects of these pillar widths on the stabilities of the end-wall and pillar were discussed.

The analytical initial conditions were supposed as follows:

- a) Two-dimensional elasto-plastic analysis and plane strain;
- b) Mesh type is uniform and element type is six nodes triangles;
- c) Failure criterion; the Mohr-Coulomb criterion;

Table 2 shows the mechanical properties of the rocks used in this analysis.

Table 2. Mechanical properties of rocks.

Lithology	Depth	Thickness	Unit weight (kN/m ³)	Young's Modulus (MPa)	Poisson's ratio	Cohesion (MPa)	Friction angle (deg)	Tensile strength (MPa)
Loess	30	30	19.6	15	0.42	0.085	12	0.0125
Weathered sandstone	44	14	23.0	2,000	0.36	2.5	38	0.75
Sandstone	74	30	23.8	4,200	0.32	3.0	39	0.9
Mudstone	98	24	24.9	2,800	0.34	2.0	38	0.6
Siltstone	110	12	23.2	4,600	0.32	3.5	36	1.0
Sandstone	122	12	23.8	4,000	0.30	4.0	40	1.2
No.4 coal seam	130	8	14.4	1,000	0.38	1.62	36	0.7
Shale	145	15	24.5	2,400	0.33	3.0	42	0.9
Siltstone	160	15	26.0	4,800	0.32	5.0	38	1.2
Shale	170	10	25.8	3,500	0.35	5.0	38	1.0
No.9 coal seam	183	13	13.3	1,200	0.36	1.62	36	0.7
Sandstone	195	12	23.8	2,900	0.28	5.0	41	1.2
No.11 coal seam	198	3	14.0	1,300	0.35	1.62	36	0.7

According to the code for design of open pit mine of coal industry in China (China planning press, 2005), the slope safety factor is required to be more than 1.1 for non-working slopes when its service life is less than 10 years. The end-wall slope safety factor in the studied coal mine was about 1.32 before end-wall mining, as shown in Figure 8.

From this figure, it is found the whole end-slope is very stable unless the loess layer is laid on the upper part. In other words, the loess bench is the weakest layer which determines the safety factor of the end-wall slope. On the other hand, the lower layers are mainly made up of sandstone, which has the relative high strength; as a result, it seems to be a good condition for the application of the end-wall mining.

Figures 9 (a) through (f) show the distribution of maximum principal stresses and failure zones around the mining panels/pillars in different pillar widths. It was found that the yield zones around the mining panel/pillars decreases with an increase in the pillar width. Moreover, when the pillar width is larger than 17m, the pillar width can meet the requirements of pillar width and its stability can be maintained. The value of 17m is the same as that calculated by Mark-Bieniawski formula. Therefore, the Mark-Bieniawski formula may serve as an important reference for the pillar design in end-wall mining systems.

Figures 10 (a) through (c) show the distribution of maximum principal stresses and failure zones around the end-slope and mining panels in different widths of boundary pillars when the pillar width was set as 17m. When the boundary pillar is 10m, the mining operation has an obvious impact on the stability of end-slope and haulage road. As a result, the shallow subsidence and large slide may occur due to the mining operation. On the other

hand, the stability of the end-slope increases with the increase in the width of the boundary pillar. When the boundary pillar width is larger than 30m, no failure occurs in the end-wall slope and the mining operation has little impact on the haulage road. Moreover, from Figure 11, the safety factor around the end-slope after an end-wall mining operation is about 1.43 and the value can also be meet the safety requirement.

These results represent the possibility of application of the end-wall mining system to this mine in case that the coal pillars are designed appropriately.

CONCLUSIONS

Using the results of a series of theoretical and numerical analyses, the following conclusions can be made:

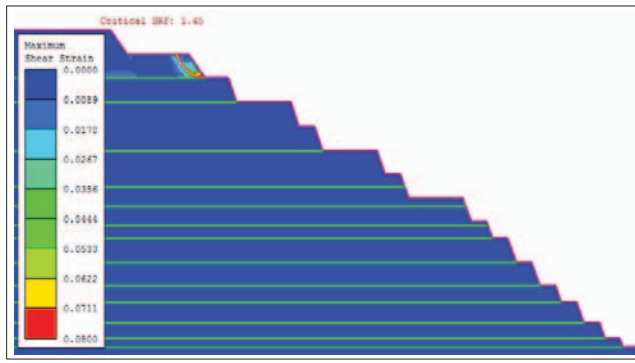


Figure 8. Safety factor and distribution of the maximum shear strain in the end slope before end-slope mining.

(1) The end-wall mining technology may be the useful method for the extraction of residual coal around the end-wall in ATB open pit coal mine without instability of the end-wall and disturbance of transportation in open pit mining operation in case that the coal pillars are designed appropriately. Moreover, since the coal around end-walls is exposed and there is no extra stripping engineering quantity, then the economic benefit of the open pit mine may be improved.

(2) The coal pillar width will be larger with the increase of mining openings' width and height. However, the coal recovery ratio can be improved. To improve the recovery ratio, we should increase the mining width and height as much as possible when conditions permit. Using analytical calculations, the optimal mining width should be 40m and the pillar width is 17m in the studied coal mine.

(3) According to the results of numerical simulations, when the boundary pillar width is 30m, no failure occurs in the end-wall slope. At same time, it is found that the safety factor of the end-wall slope after end-wall mining system is about 1.43, which also can meet the safety requirements.

ACKNOWLEDGEMENT

The authors would like to express their gratitude to the manager and engineers of the ATB coal mine in China. The authors also gratefully acknowledge Professor Qingxiang Cai and Prof. Tao Shang for valuable suggestions and important feedback.

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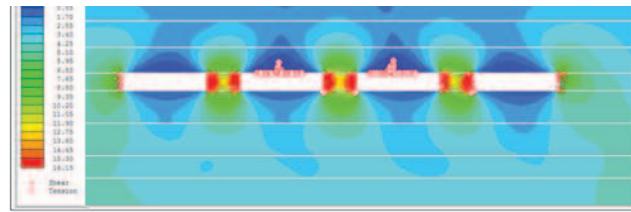
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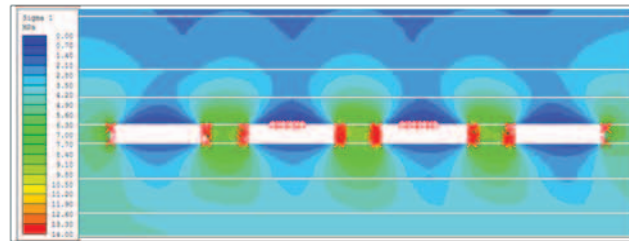
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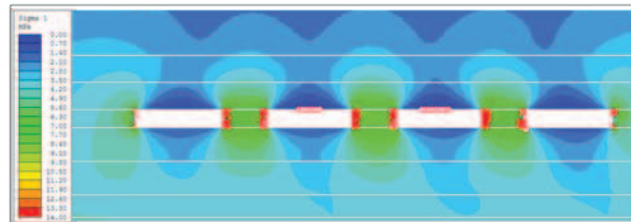
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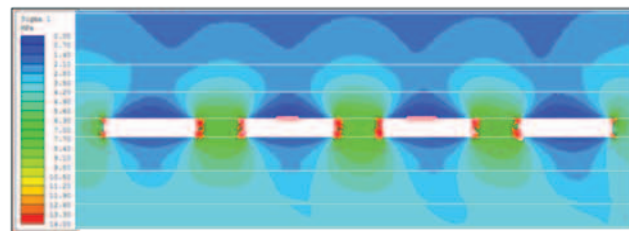
(a) Pillar width=12m



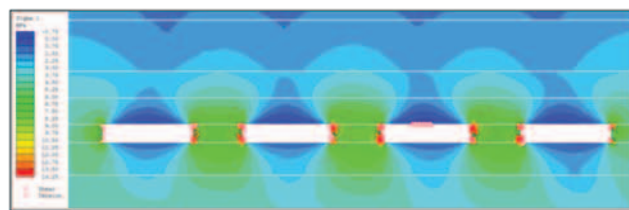
(b) Pillar width=17m



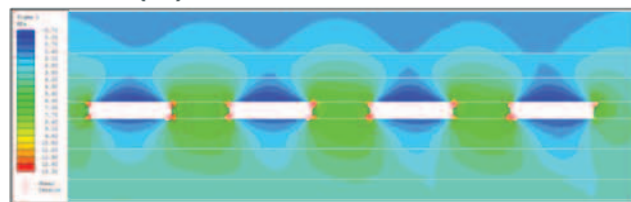
(c) Pillar width=18m



(d) Pillar width=19m



(e) Pillar width=22m



(f) Pillar width=28m

Figure 9. Distribution of the maximum principal stress and yield element around the mining panel in different pillar widths (the width of working face is 40m).

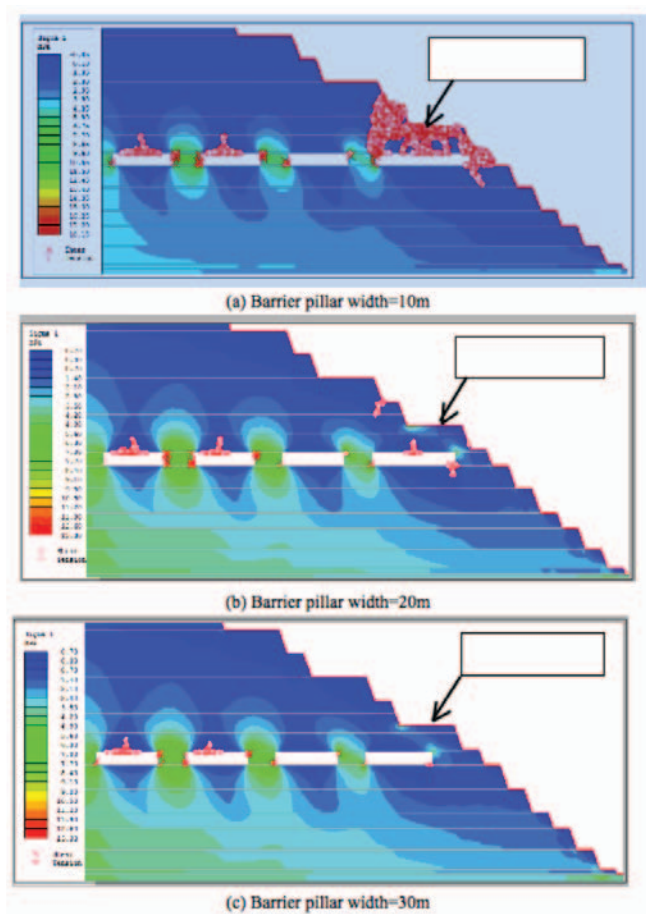


Figure 10. Distribution of the maximum principal stress and yield element in the end-wall in different barrier pillar widths (working face width: 40m, the pillar width: 17m).

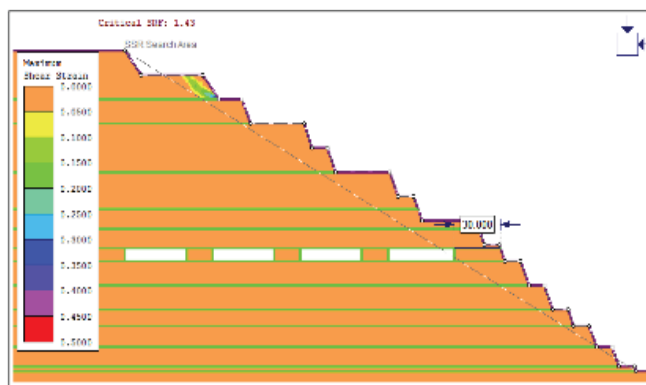


Figure 11. Strength factor in the end-wall after end-wall mining. (the width of working face: 40m, the pillar width: 17m, the boundary pillar width: 30m).

Investigation of the Influence of Mining with Caving on Deformation of Roadway Located in Close Proximity of Longwall Panel

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ABSTRACT

In Polish coal mining due to specificity of seams lying the multiple extraction is used. In this kind of mining, it is essential to take into account all geological and mining conditions which have an influence on stability and functionality of workings during their using.

In view of above, for one of Polish coalmines, by means of Phase² program, the numerical calculation was made for assessment of the influence of advancing excavation front on convergence of roadway located 30 meters above longwall was analysed. In order to verify developed prognosis, in analyzed roadway the underground investigations in scope of convergence measurement and changes of load exerted on working support were carried. As a result of these investigations it was found that real convergence on certain part of roadway was significantly larger than determined in prognosis. Additional calculations and analysis showed that the main cause of differences was underestimation of influence of dynamic load, which was an effect of numerous low energetic rockmass tremors. These tremors, related to excavated longwall, were located in close proximity of roadway in its part where the biggest convergence occurred. In current practice, in this type of convergence prognosis, the maximal value of dynamic load exerted on roadway resulted from single rockmass tremor is taken into consideration. Quantitative consideration of all tremors including their location is impossible at the stage of convergence prognosis because of lack of this type of data.

This paper presents results of roadway convergence prognosis compared to results of underground investigation. Conclusions formulated on that base allow in future similar designing works for obtaining more accurate results.

INTRODUCTION

The multiple seam extraction in the Polish hard coal mining industry results in most newly driven mine workings being located in zones of influence of old mine workings in neighbouring seams. The old workings undoubtedly have an effect on the value of rock mass pressure, which can lead to increased deformations of headings, and in the worst situations, to a total loss of functionality. Practical experience indicates that the largest rock

mass displacements and support deformations most frequently take place at those headings that experience influence from extraction activities performed nearby. Intensive deformations are observed in workings located below and above mined seams (Prusek, 2008).

Recently, such a situation has taken place at JAS-MOS colliery, which is a part of the JSW Group and the biggest coking coal producer in Poland. In that colliery, incline Z-1 has been affected by the approaching longwall No 2 face in the lower seam. The longwall was located in seam 510/2 about 30 m below a heading (Figure 1). Incline Z-1 is an essential element of the underground infrastructure of the mine because it is an important transportation and return airway.

Due to the stability concern of the incline Z-1, the colliery turned to the Central Mining Institute (Katowice, Poland) and requested to perform a computer simulation (numerical modelling) to evaluate the influence of active longwall face No 2 in seam 510/2 on the deformation/convergence at incline Z-1 in seam 510/1.

Based on the mining and geological conditions in seams 510/1 and 510/2, the vertical and horizontal convergence of incline Z-1 in seam 510/1 as a function of distance to the face of longwall No 2 were calculated with the Phase² program (Finite Element Method). To verify results of the modelling calculations, measurement stations at incline Z-1 that monitored actual convergence values in the working were prepared.

The results of the numerical calculations and of the underground measurements presented in this paper should broaden the knowledge on predicting deformations of mine workings exposed to the front abutment zone of lower seam longwall working.

Geological and Mining Conditions

Incline Z-1 was situated in seam 510/1 which is located at a depth of approximately 680 m. The working was placed between gob of previously mined coal areas, one at a distance of 60 m north and the other 140 m south. In the area of incline Z-1, seam 510/1 is composed of one or two coal strata, has an overall thickness ranging from 0.8 to 3.2 m, and is inclined to the north and northwest at an angle varying from 6° to 26°.

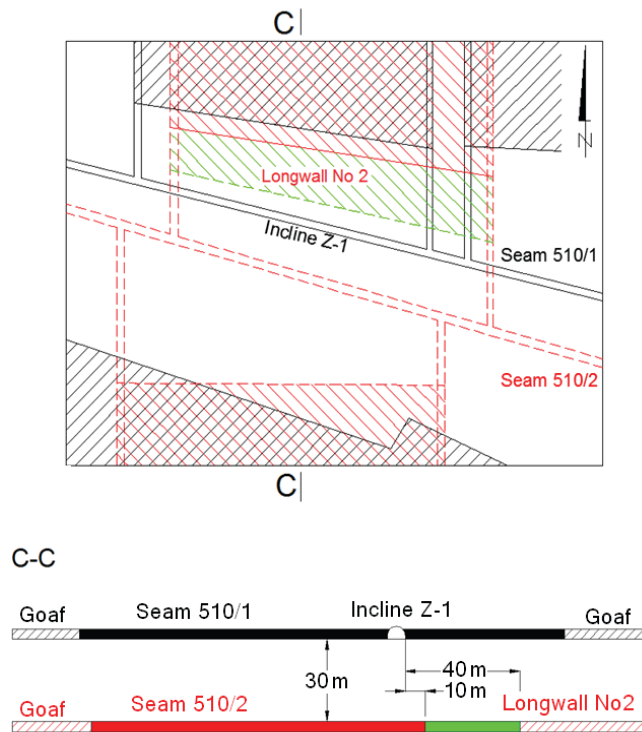


Figure 1. Schema showing location of incline Z-1 in seam 510/1 and active longwall No 2 in seam 510/2.

The immediate roof of seam 510/1 consists of the following strata: shale with coal intercalations (4.5 m), sandstone (2.0 m), sandy shale (2.9 m), and sandstone (48.7 m). The immediate floor is made up of clay shale stratum (1.7 m) and, farther below, sandstone stratum (6.6 m). A more complete illustration of its geological setting/profile is presented in Figure 2.

Seam 510/2, where the extraction with caving was active, is located below seam 510/1 at a distance of about 30 m. The longwall, with a height of up to 2.6 m, mined seam 510/2 at its full thickness. Average daily advance of the longwall face, when it neared its panel's end, amounted to about 2.5 m. Longwall No 2 was about 250 m wide and 900 m long. The roof of seam 510/2 (Figure 2) consists of a 20-m-thick sandstone, while its floor is composed of sandy shale (0.4 m) and sandstone (2.6 m). Because of rockburst hazard, which occurred in seam 510/1 and seam 510/2, the colliery predicted possible rock mass tremors with a maximum energy of 10^5 J. Incline Z-1 was supported by yielding steel arches. The distance between arches was 1.0 m. Steel arches were made from steel V-shaped cross sectional profiles with an elementary mass of 29 kg/m. The roof strata in incline Z-1 was reinforced with 6 meters long flexible bolts installed between each arch. Figure 3 presents the bolting pattern.

PREDICTION OF INCLINE Z-1 CONVERGENCE

Numerical calculations of incline Z-1 convergence were carried out using the program Phase² based on the Finite Element Method, assuming that the rock mass is an elastic-plastic and isotropic medium. The boundary state conditions were calculated according

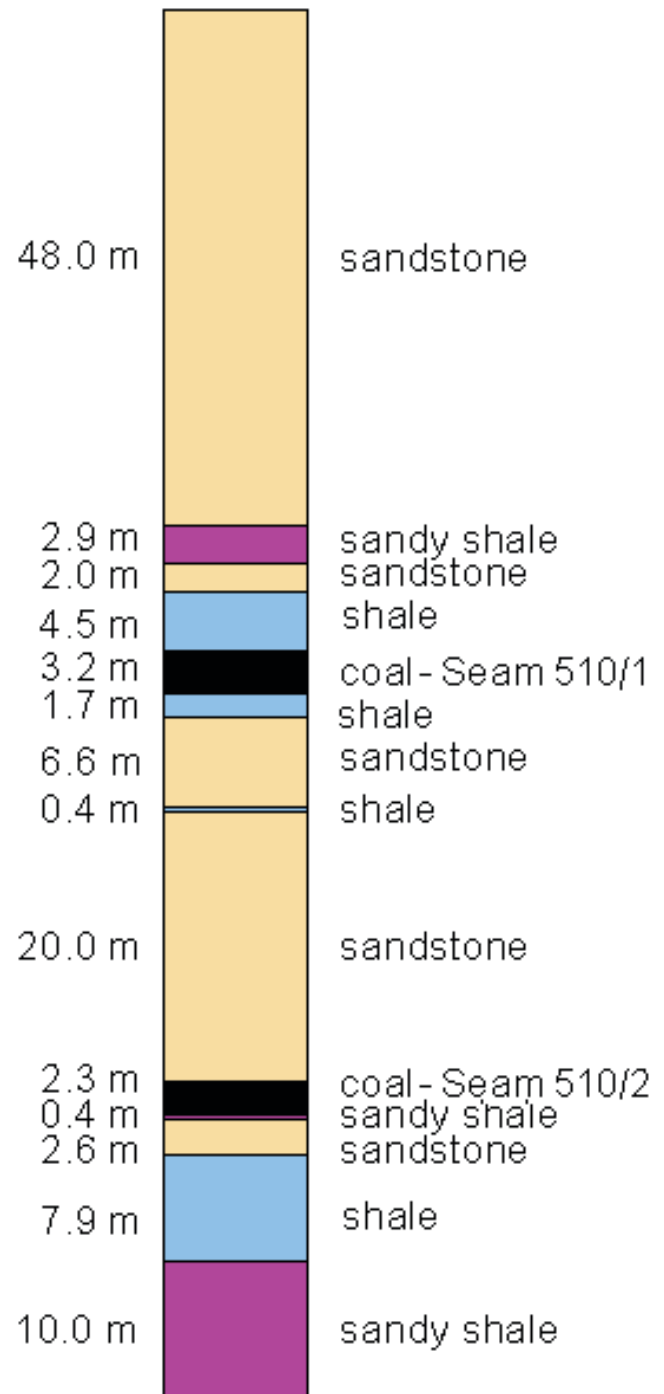


Figure 2. Rock strata around seam 510/1 and seam 510/2.

to the Hoek-Brown criterion, for which the fractured rockmass is defined (Hoek, Carranza-Torres, Corkum, 2002; Hoek, 2007):

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a$$

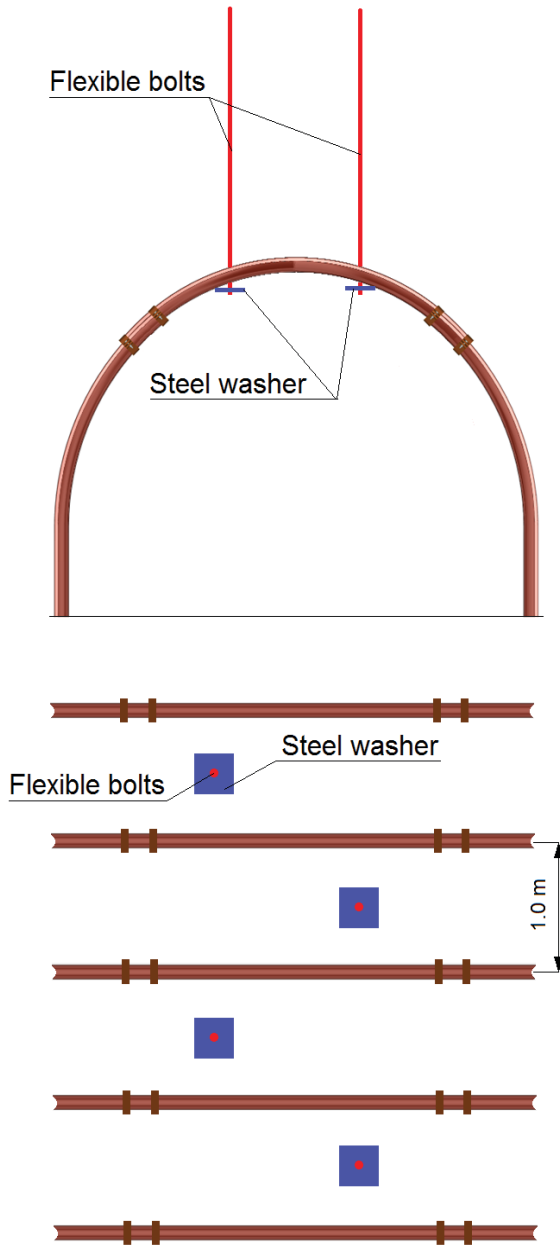


Figure 3. Diagram of support for incline Z-1.

Where

σ'_1 and σ'_3 = effective maximum and minimum stress at failure (MPa)

m_b = value of the Hoek-Brown parameter for the rockmass if

s and a = parameters, determined on the basis of rock mass properties

σ_{ci} = uniaxial compressive strength of rock sample (MPa)

A two-dimensional numerical rock mass model with dimensions 120 m x 120 m was prepared to calculate the convergence values of incline Z-1. In the model, types of rock strata and their

arrangement were assumed according to the geological profile presented in Figure 2. Calculations were planned to simulate the longwall's approaching face to the incline's side wall, beginning with a distance of 40 m and ending at a distance of 10 m. Figure 4 presents an example of such a model.

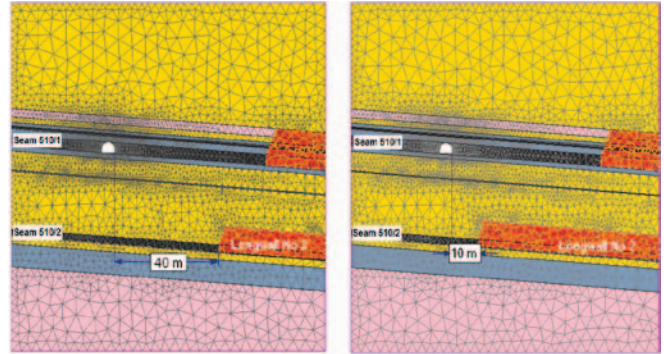


Figure 4. Models of rock mass around incline Z-1 and longwall No 2 at 40 m and at 10 m.

The basic values of rock layer parameters adopted for calculations are presented in Table 1; among them are those describing the Hoek-Brown criterion (Hoek, Carranza-Torres, Corkum, 2002; Hoek, 2007).

Moreover, it has been adopted in the model that the initial stresses correspond with the primary state of stresses, which results from the depth of workings position ($H=700$ m) and average weight by volume of overburden rocks ($\gamma=0.025$ MN/m³). The model also considers elements of workings' support such as LP frames and string bolts, according to the assumptions described in previous publications (Walentek et al., 2009; Walentek and Masny, 2010). A value for tremor energy (10^5 J) was also taken into account in calculations by applying it to model upper boundary.

The results of calculations of rock mass displacements and convergences that occurred at incline Z-1 at varying distances from longwall No 2 performed with the assumptions mentioned above are listed in Table 2.

An analysis of the numerical calculation results, as listed in Table 2, showed that approaching longwall No 2 in the underlying seam had moderate influence on convergence values at incline Z-1. The calculations have indicated a maximum value of vertical convergence of 227 mm, which makes up about 6.5% of the working height, and a maximum horizontal convergence of 398 mm, which translates to about 8.0% loss in the working width. The convergence values proved to be acceptable from the point of view of the working's functionality. As a result, longwall No 2 was permitted to approach incline Z-1 up to a 10-m distance.

Underground Measurements in Incline Z-1

In order to verify the results of the numerical calculations, underground measurements of deformations and load exerted on support at incline Z-1 were taken during the longwall face approach. The deformations were evaluated by measuring vertical convergence (V) and horizontal convergence (H), according to the methodology developed at the Central Mining Institute (Prusek, 2008). Measurements were performed at two measurement stations

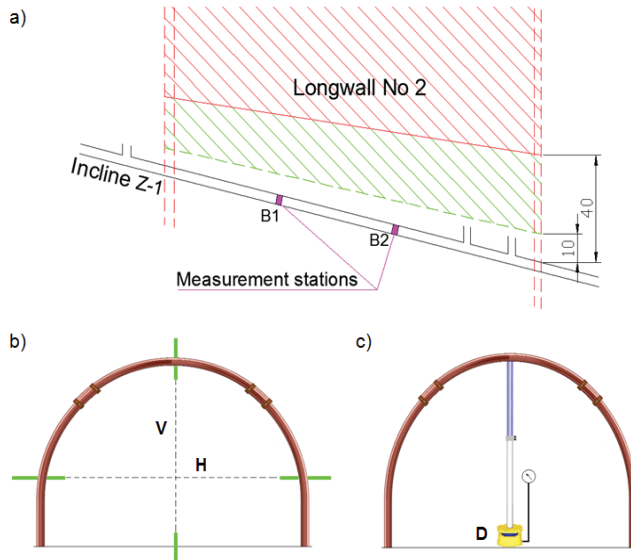
Table 1. Basic parameters of rock layers adopted for numerical calculations.

Types of rocks	Young's modulus E (MPa)	Poisson's ratio ν	Compression strength R_c (MPa)	Parameter m_b	Parameter s
coal	1,200	0.30	7.3	0.610	0.0008
shale	2,800	0.23	20.0	1.080	0.0016
sandy shale	6,621	0.23	40.2	1.294	0.0035
sandstone	12,891	0.21	59.0	2.775	0.0105
goaf	500	0.40	5.0	0.275	0.0002

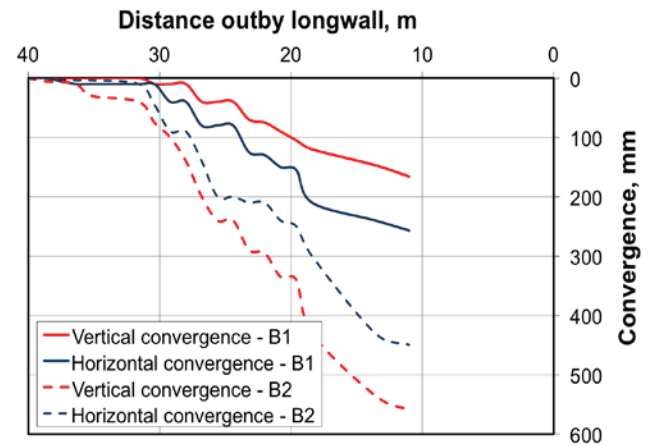
Table 2. Results of numerical calculations of convergences at Incline Z-1 depending on distance to longwall No 2.

Distance of longwall front, (m)	Roof subsidence, (mm)	Floor heave, (mm)	Vertical convergence, (mm)	Horizontal convergence, (mm)
40	106	68	174	359
30	114	73	187	376
20	126	79	205	385
10	139	88	227	398

(B1 and B2) located at incline Z-1 (Figure 5a). They included daily recordings of changes in the distance between benchmarks installed at the working's roof and floor as well as at its side walls (Figure 5b). In addition, a hydraulic dynamometer (D) was installed under a friction prop erected in the middle of a roof arch to determine changes in the load exerted on the working's support (Figure 5c). The first measurements were performed when longwall No 2 was located at a distance of 40 m from incline Z-1, and the last ones were taken at a distance of 10 m. The results of convergence and load measurements for incline Z-1 are presented in Figures 6 and 7.


Figure 5. Schema of measurement stations in incline Z-1.

Analyzing vertical and horizontal convergence measurement results obtained at stations B1 and B2 (Figure 6) shows considerable differences between the maximum values. The maximum value of vertical convergence at B1 amounted to 166


Figure 6. Vertical and horizontal convergence measurement results for incline Z-1 (stations B1 and B2).

mm, whereas at B2 it was much as 560 mm. Similarly, horizontal convergence measurements showed changes of 257 mm at B1 and 450 mm at B2.

Interesting results were also obtained from measurements of support load changes.

The maximum load value noted at measurement station B1 amounted to 187 kN, whereas it was 170 kN at B2. The values are similar but the times at which they occurred are not. The maximum value noted at B2 occurred earlier than the maximum value at B1 because the vertical convergence occurred earlier at B2 than at B1. Also, load values began to consistently fall and at the same time the vertical convergence increases. Normally, increases in vertical convergence should not be accompanied by lasting load decrease. This resulted from placing the hydraulic dynamometers under friction props, and the load lines are most likely the support characteristics of the friction props.

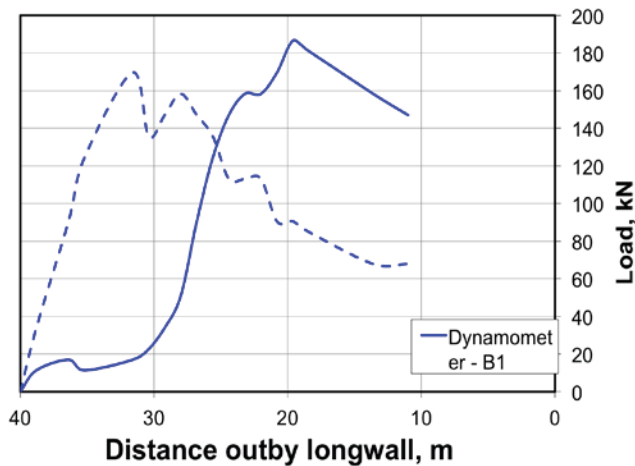


Figure 7. Results of measurements of load exerted on support for incline Z-1 (stations B1 and B2).

COMPARISON OF NUMERICAL CALCULATION RESULTS AND UNDERGROUND MEASUREMENTS

After comparing the convergence values at incline Z-1 obtained from numerical calculations with ones obtained from in situ measurements, similarities were seen only in the results from measurement station B1. For measurement station B2, the numerical prediction was considerably underestimated.

While analyzing geological and mining conditions, it was concluded that the more intensive convergence of incline Z-1 in measurement station B2 could have been caused by an increased seismic activity of rockmass that took place when longwall No 2 neared the end of the panel. The seismic services in the colliery recorded about 400 tremors with energy magnitudes ranging from 10^2 J to 10^5 J. Figure 8 shows some of the tremors that occurred near incline Z-1.

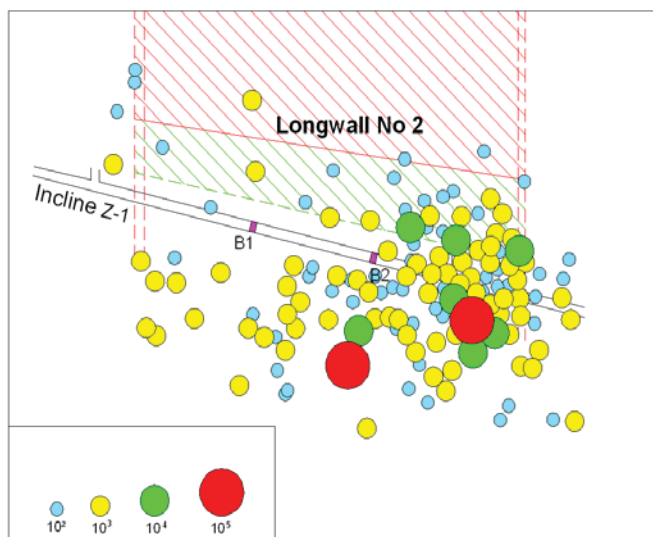


Figure 8. Location of rock mass tremors that occurred near incline Z-1.

According to Figure 8, the highest intensity of rock mass seismicity took place near measurement station B2 where convergence values were considerably larger. Unfortunately, only the single maximum value of rock mass tremor energy has been assumed for the numerical calculations. Taking into account all tremors at the stage of convergence prediction is unfeasible due to lack of appropriate data and the difficulty in obtaining that data. Thus, as follows from observations, a large frequency of low-energy tremors concentrated in a small area may also have a negative effect on near located workings and make it unstable.

SUMMARY

This paper presents the calculated and measured convergence values in incline Z-1, which was influenced by a longwall in an underlying seam; this allows for formulation of the following conclusions:

- Modeling with the Phase² program is suited for cases when the convergence value depends mainly on influence of abutment pressure of advancing longwall.
- It is not possible to use the Phase² program to model when a large number of tremors in the heading area influence convergence.
- Further work is necessary to consider numerical modeling for a large number of rock mass tremors that affect the underground mining openings.

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Initial Results From Implementing a Laminated Overburden Model Into ARMPS

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ABSTRACT

In a recent research effort, a laminated overburden loading model (as implemented in LaModel3.0) was compared with the results of ARMPS 2010, and the laminated overburden model was found to more accurately classify a ARMPS database of 52 deep cover case histories by a factor of 60% versus 54%. However, the LaModel analysis of each case study required several days to complete, as compared to a requirement of only several minutes for each ARMPS analysis. In the research presented in this paper, a computer code (ARMPS-LAM) has been developed to effectively implement the laminated overburden model into the ARMPS program. This program takes the basic ARMPS geometric input for defining the mining plan and loading condition and then automatically develops, runs, and analyzes a full LaModel analysis of the mining geometry to output the stability factor on the Active Mining Zone (AMZ), all without further user input. The present laboratory version of ARMPS-LAM can be run in batch mode; therefore, the ARMPS 2010 database of 645 case histories can be analyzed very quickly. In an initial logistic regression analysis of this database, the ARMPS-LAM program was seen to be slightly more accurate than ARMPS 2010 by a factor of 71% versus 63%.

INTRODUCTION

ARMPS

The original version of ARMPS was developed by NIOSH in the mid-1990s (Mark and Chase, 1997) to aid in the design of room and pillar retreat mining operations in underground coal mines. ARMPS was created with a very user-friendly interface, and it was simple and fast to use. Over the years, it has been widely used in the United States, and undoubtedly, it has greatly improved the safety of retreat room and pillar mining operations. In the ARMPS program, the stability factor of the Active Mining Zone (AMZ) is determined by comparing the strength of the pillars (determined using the Mark-Bieniawski pillar strength formula) in the AMZ against the development loads (determined using the tributary-area method) and abutment loads (determined using the abutment angle concept) applied to those pillars. The value of the calculated pillar stability factor is then compared against a large database of successful and unsuccessful case histories to determine the chance of success of the chosen design. This large database of case

histories is the real strength of the program and provides very solid empirical support for the application of the ARMPS method.

In the wake of the widespread pillar failure at the Crandall Canyon mine, research efforts with ARMPS focused on improving the loading half of the stability factor determination. In particular, a “pressure arch” algorithm was added to the tributary-area loading calculation in the newest version of the ARMPS program (ARMPS 2010) in an effort to more accurately model deep cover mines (Mark, 2010). The pressure arch loading model takes the tributary area loading on the AMZ and reduces it by the “pressure arch factor” (F_{pa}). The reduced AMZ loads are then transferred to the barrier pillars. If the barrier pillars are too small to carry all of the applied loads, then the pressure arch loads are transferred back to the AMZ. Initially, three different pressure arch loading functions were analyzed. Ultimately, the optimum pressure arch equation was determined to be the logarithmic function shown below (Mark, 2010), which only applies when the depth of cover is greater than the panel width plus 80 ft.

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

where:

F_{pa} = the pressure arch factor

H = overburden depth

P_w = panel width.

The manner in which the pressure arch factor was determined guarantees that the new pressure arch loading model provides the optimum separation between the successful and unsuccessful cases in the database. Further, the pressure arch loading model eliminates the previous depth effects in ARMPS 2002 and allows for the use of a constant value for the recommended stability factor (SF) regardless of depth. Based on the latest statistical analysis with an database of 645 case histories, the 2010 design criteria require a minimum ARMPS SF of 1.50 for depths less than 650 ft, and a minimum ARMPS SF of 1.50 and a minimum barrier pillar SF of

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1.50 for depths over 650 ft. With these design criteria, ARMPS 2010 has a 82% accuracy in predicting the unsuccessful case histories, a 61% accuracy in predicting the successful cases, and an overall classification accuracy of 65%(Mark, 2010).The new ARMPS 2010 works very well as a guideline for designing room-and-pillar mines, but there is still plenty of room for improving the overall prediction accuracy of the program.

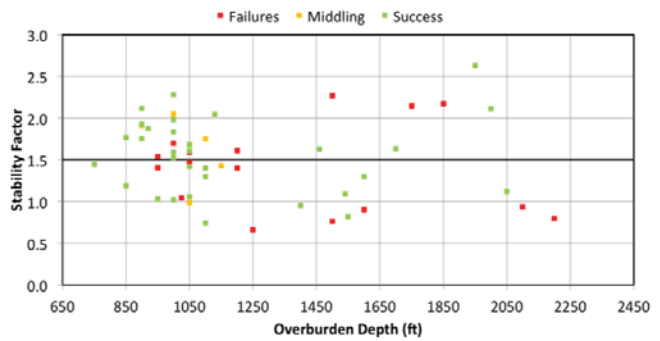
LaModel

In contrast to the empirical ARMPS program, the LaModel program is a numerical program based on simplified overburden and seam mechanics. It simulates the overburden as a stack of homogeneous, frictionless plates (Heasley, 1998), and within the seam horizon, the elements are simulated with various stress-strain behaviors (elastic, elastic-plastic, strain-softening, and strain-hardening) intended to model the various behaviors of the in-seam coal and gob. When the program is executed, it takes the user input behavior of the overburden and balances it with the in-seam material behaviors and solves for the resulting displacements, stresses, pillar safety factors, etc. The LaModel program requires quite a bit more time, effort, and complexity to run than the ARMPS program, but with its ability to fairly accurately model complex mining geometries, variable topography, multiple-seams, and many other situations and/or detail beyond the capability of ARMPS, it has found quite extensive use throughout the coal mining industry.

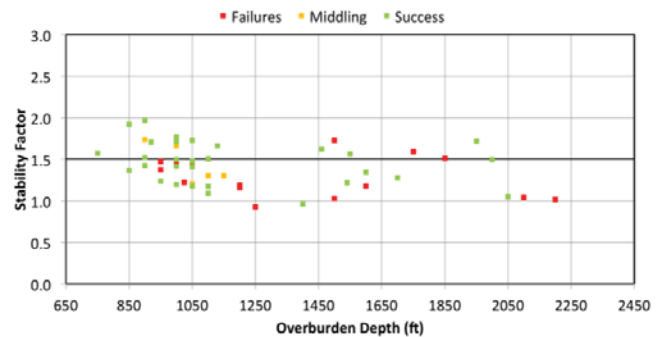
One of the difficulties of using LaModel (or any numerical model) is determining the appropriate values for the input parameters. Given the complexity of the geologic environment, laboratory determined parameter values are typically not accurate for simulating the *in situ* geomechanical behavior of the mine structure. Rather, the best way to produce an accurate numerical model is often to calibrate the critical input parameters against measurements and/or observations in the field. In the last few years, a formal procedure for calibrating the critical input parameters for LaModel versus field observations has been developed and implemented into the program's version 3.0 pre-processor (Heasley et al., 2010; Heasley, 2008). Essentially, the lamination stiffness in LaModel is calibrated against the measured/observed/empirical extent of the abutment stress, the gob stiffness is calibrated against the expected gob/abutment loading, and the *in situ* coal strength is input using the NIOSH default of 900 psi or is back-calculated (Heasley, 2008).

In a recent analysis of 52 deep-cover case histories, where the results from calibrated LaModel3.0 runs were compared with the results of ARMPS 2010, it was found that the LaModel program more accurately classified the case histories by an overall factor of 60% versus 54%. Further, the LaModel program more accurately classified the critical design failures by a factor of 76% versus 52% (Tulu et al., 2010).

The results of the database analysis are presented in Figure 1 (a and b). For the ARMPS 2010 analysis shown in Figure 1a, 52% of the failed case histories and 55% of the successful case histories were correctly classified for an overall classification accuracy of 54%. For LaModel3.0, as shown in Figure 1b, 76% of the failed case histories and 48% of the successful case histories were correctly classified for an overall classification accuracy of 60% (Tulu et al., 2010).



a. Stability factor analysis with ARMPS 2010



b. Stability factor analysis with LaModel 3.0

Figure 1. Comparison between ARMPS 2010 and LaModel3.0 (after Tulu et al, 2010).

In this analysis of the relatively small deep-cover database, the laminated overburden model showed significant promise for improving the accuracy of the ARMPS calculation; however, in this research, the LaModel analysis of each case study required several days to complete, as compared to a requirement of only a few minutes for each ARMPS analysis. Using LaModel3.0, the model input parameters and mine grid needed to be entered by hand, and the output needed to be specifically analyzed to determine the stability factor within the active mining zone. The user expertise and attention for LaModel required considerable more time and effort than performing a simple ARMPS analysis. However, if only a simple ARMPS type mining plan is to be analyzed, and the user is satisfied with a 900 psi Mark-Bieniawski coal pillar strength and a 21° abutment angle type of gob loading (as used in ARMPS and as used in the calibrated LaModel), then it is certainly possible to automate all of the parameter input, grid generation, and output analysis associated with the LaModel program. This automation would greatly increase the speed of using a laminated overburden model in an ARMPS-type room-and-pillar analysis and potentially increase the overall accuracy of the pillar design.

ARMPS-LAM

In the research presented in this paper, a computer code (ARMPS-LAM) has been developed to effectively implement the laminated overburden model into the ARMPS program. This ARMPS-LAM program takes the basic ARMPS geometric input for defining the mining plan and loading condition and then

automatically develops and analyzes a complete LaModel file of the mining geometry to calculate the stability factor on the AMZ (and barrier pillars), all without further user input. For the user, only the traditional ARMPS input is required, and similar to ARMPS, the output is the stability factor of the AMZ and barrier pillars; however, it is now calculated using the laminated overburden model. Based on the deep-cover analysis described above, it was anticipated that this new ARMPS-LAM program would have the potential to significantly improve the accuracy of the mine design classification, and therefore, greatly improve the safety of the underground coal mines.

To completely automate the model building and analysis process, numerous complex algorithms and mathematical formulas needed to be developed and implemented. Fortunately, many of these algorithms and formulas associated with material properties and the stress solution had already been developed and implemented in previous research. However, to completely automate the mine plan gridding, quite a number of new algorithms needed to be developed. The major steps in the automated development and analysis of the LaModel file in ARMPS-LAM are listed below and further detailed in the following paragraphs:

Pre-Process:

- **Data Import:** Read the input file containing the ARMPS 2010 data (primarily geometry).
- **Property Calibration:** Determine the appropriate properties for the overburden, coal, and gob materials based on the formal LaModel calibration procedure.
- **Boundary Pillar Sizing:** Based on the input mine geometry and calibrated overburden properties, determine the radius of displacement and stress influence and, thereby define the required size for the boundary pillars around the edge of the model.
- **Mine Model Sizing:** Based on the input mine geometry and the required boundary pillar sizes, determine the overall required size for the model grid.
- **Mine Plan Generation:** Based on the input mine geometry and the required boundary pillar sizes, generate the exact boundary lines for the pillars and gobs, and output as an AutoCAD file.
- **Element Sizing:** Based on the overall model dimensions and the dimensions of the entries and pillars, determine an optimum element size.
- **Grid Generation:** Insert the coal and gob materials into the mine grid based on the pillar and gob locations.
- **Yield Zone Application:** Apply the yield zones to the pillars based on the Mark-Bieniawski stress gradient.
- **Input File Generation:** Generate and output the LaModel input file based on the final material properties and mine grid (this input file is a fully compatible LaModel3.0 input file).

Process:

- **Run the Model:** Solve the model using LaModel and generate the output file (this output file is a fully compatible LaModel3.0 output file).

Post-Process:

- **Calculate Stability Factors:** From the LaModel output file, determine stability factors of the AMZ and barrier pillars.

EXAMPLE CASE HISTORY

In order to illustrate the automated model building and analysis process of ARMPS-LAM, one example case history from the ARMPS database was selected and is presented in detail here. This case history, located in Kentucky, has a cover depth (H) of 860 ft, an entry height (h) of 7.7 ft, a crosscut angle of 60°, an entry width of 20 ft, a crosscut spacing of 60 ft, uses the default abutment angle of 21°, and has an active and one side gob (loading condition of three). There are five entries with the same center-to-center spacing of 55 ft. The front active gob extends in by the pillar line a distance of 1,000 ft, and the first side gob has a width of 340 ft with a 75 ft wide barrier pillar separating the two retreat sections that has slab cut of 50 ft.

Pre-process – Data Import

The proceeding ARMPS parameters are stored in a one-line input file, which is read into ARMPS-LAM in the data import step.

Pre-process – Property Calibration

Next, the program uses the formal LaModel calibration process (Heasley et al., 2010) to automatically calibrate the overburden, coal, and gob material properties. For this sample case history, a lamination thickness of 307 ft, a coal strength of 900 psi (as implemented in the Mark-Bieniawski formula), and a final gob modulus of 1.66×10^6 psi (as implemented in the exponentially hardening gob) were developed.

Pre-process – Boundary Pillar Sizing

To determine the appropriate width of boundary pillars to use around the LaModel grid, the radius of stress/displacement influence is determined. Given a lamination thickness of 307 feet and a depth of 860 ft, the radius of influence is determined as 2,087 ft (Heasley, 1998).

$$R_p = \sqrt{\frac{2tz \ln(1-p)}{\sqrt{3(1-v^2)}}} = \sqrt{\frac{2 * 307 \text{ ft} * 860 \text{ ft} * \ln(1-0.999999)}{\sqrt{3 * (1-0.25^2)}}} = 2,087 \text{ ft} \quad (2)$$

where:

R_p = Radius of influence, ft

P = Percentage of displacement/stress influence that is included (99.9999%)

t = Lamination thickness, 307 ft

v = Poisson's Ratio of laminations, 0.25

z = Vertical distance between points, 860 ft

Pre-process – Mine Model Sizing

The overall size of the LaModel mine grid is determined by the input mine geometry and the calculated boundary pillar width. The total mine grid width (TWM) in ARMPS-LAM is then the sum of the width of the first side edge boundary pillar (WEB(3)), the width of the first side gob (WSG(3)), the width of the first side bleeder pillar (WSBP(3)), the width of the first side barrier pillar (WBAR(3)), the width of the extraction panel (WEP), the width of the second side barrier pillar (WBAR(4)), the width of the second side bleeder pillar (WSBP(4)), the width of the second side gob (WSG(4)), and the width of the second edge boundary pillar (WEB(4)). See Equation 3 below. (For these width variables, as used in ARMPS, the first side is designated by the index of 3 and the second side is designated by the index of 4.) In the example case history, there were no bleeder pillars left in the first side gob, and there was no second side gob, and therefore no second side barrier pillar or second side bleeder pillars. As a result, their widths were assigned as zero. The equation for the total width of the mine grid model (TWM) is given below and calculates a total model width of 4,864 ft.

$$\begin{aligned} \text{TWM} &= \text{WEB}(3) + \text{WSG}(3) + \text{WSBP}(3) + \text{WBAR}(3) + \text{WEP} \\ &\quad + \text{WBAR}(4) + \text{WSBP}(4) + \text{WSG}(4) + \text{WEB}(4) \\ &= 2,087 \text{ ft} + 340 \text{ ft} + 0 + 75 \text{ ft} + 275 \text{ ft} + 0 + 0 + 0 + 2,087 \text{ ft} \\ &= 4,864 \text{ ft} \end{aligned} \quad (3)$$

The total mine grid length (TLM) is the sum of length of top edge boundary pillar (LTEB), the length of central active gob (LCA), the breadth of the AMZ, and the length of bottom edge boundary pillars (LBP). The equation for the total length of the mine grid is given below and calculates a distance of 5,332 ft. Figure 2 shows a scaled schematic of the model case history with the distance parameters labeled.

$$\begin{aligned} \text{TLM} &= \text{LTEB} + \text{LCA} + \text{AMZ} + \text{LBP} \\ &= 2,087 \text{ ft} + 1,010 \text{ ft} + 148 + 2,087 \text{ ft} \\ &= 5,332 \text{ ft} \end{aligned} \quad (4)$$

Pre-process – Mine Plan Generation

The total mine grid sizes determined in the previous step along with the input mine geometry and boundary pillars are then used to generate precise polygon outlines for the mine pillars and gob. This graphical pillar geometry is also saved as an AutoCAD DXF file. A sample of the case history mine geometry at the retreat line generated by ARMPS-LAM is shown in Figure 3a, where the coal pillars are shown with black polygons and the gob boundary is shown with a magenta polygon.

Pre-process – Element Sizing

Next, a fairly complex algorithm examines the critical dimensions in the model (including the entry width, the production pillar dimensions, the barrier pillar widths, the remnant pillar widths, the gob dimensions, and the overall mine grid dimensions) and determines an optimum element size that will best fit all the

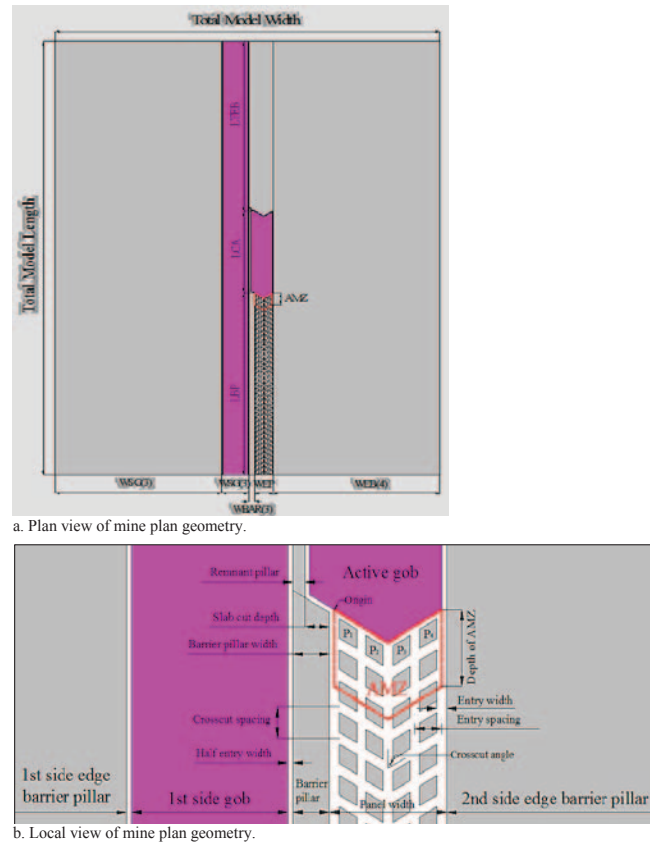


Figure 2. Mine plan geometry generated by ARMPS-LAM.

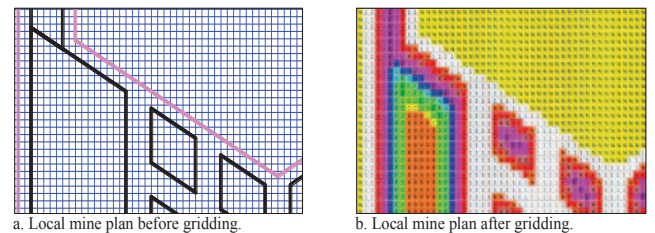


Figure 3. Plan view of mine plan gridding.

required dimensions. In this example case history, the algorithm determined that an element size of 5.0 ft was appropriate. With this element size, the final mine grid has 965 elements in the X direction and 1,066 elements in the Y direction.

Pre-process – Yield Zone Application

With the grid geometry and element size known, as shown in Figure 3a, the next step is to apply the materials to the mine grid. Using the pillar (or gob) polygons and associated grid element squares, an integrated scan line filling algorithm developed by Foley (1996) and a polygon clipping method developed by Vatti (1992) are used to apply the appropriate material code to the grid elements. Essentially, each grid element will be checked to determine how much of its area is within a pillar or gob polygon. If the enclosed element area is more than 50%, the element will be assigned the same material as the polygon. This is the same

procedure used in the LaModel automatic grid generation function written for AutoCAD.

After all of the base coal and gob materials and remaining openings are applied to the mine grid, an appropriate set of material properties for a yield zone is applied to the coal pillars. This yield zone provides a stress gradient on the pillar consistent with the Mark-Bieniawski pillar strength formula. Initially, when the coal materials were generated, the material properties for each yield material were designed to coincide with a specific distance/location within a coal pillar. Now, once the location of the coal elements are known, these yield materials are applied to the coal elements based on the location of the given element within the coal pillar. The results of the material and yield zone application are shown in Figure 3b.

Pre-process – Input File Generation

Finally, with all of the pre-processing steps completed and the final material properties and mine grid developed, ARMPS-LAM outputs the LaModel input file of the example case history. This input file is a fully compatible LaModel3.0 input file and could be further modified or enhanced using the LaModel pre-processor program, LamPre.

Process – Run the Model

LaModel is then run using the input file, and the classic LaModel output file (.out file) is produced. This output file could be analyzed using the LaModel post-processor program, LamPlt, as shown in Figure 4, which presents the colored square plot of the pillar stress safety factor on the pillar line of the example case history.

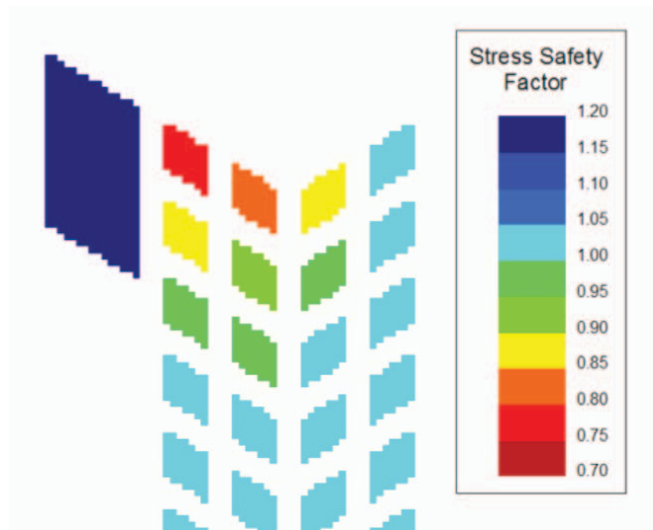


Figure 4. Colored square plot of stress safety factor.

Post Process – Calculate Stability Factors

With the LaModel output file, ARMPS-LAM then calculates and reports the stability factor of the AMZ and the barrier pillars (comparable to an ARMPS analysis). To determine the stability factor of the AMZ, an area-weighted average of the stability factor

of the pillars and partial pillars within the AMZ is used. For the example case history, the first two complete rows of pillars and about 40% of the third row of pillars are included in the AMZ. Table 1 details how the pillar safety factor for each pillar is weighted by the given pillar's percentage of area within the AMZ, and then the weighted stability factor from each pillar is summed to get the average stability factor for the AMZ. For this case history, the final AMZ SF is determined to be 0.93.

For this example case study, the complete ARMPS-LAM analysis took about five minutes of computer time, with most of that being taken to solve the LaModel grid. To develop the model, run the analysis, and calculate the AMZ stability factor would have taken several hours by hand. Thus, the automation functions of ARMPS-LAM can save the user considerable time and effort. From the user perspective, only the traditional ARMPS input is required, and the output is comparable to the traditional ARMPS stability factor of the AMZ.

ARMPS 2010 DATABASE

The ARMPS 2010 database is supplied in the installation directory of ARMPS 2010. This 2010 database includes 645 case histories covering all of the United States coal fields. The 645 cases histories in the ARMPS 2010 database were extracted from the original 692 case histories by excluding the "borderline" and "floor heave" cases. Figure 5 shows the entire ARMPS 2010 database with the ARMPS 2010 SF plotted against the depth, with the suggested design criterion of 1.50 marked (Mark, 2010).

Within the ARMPS 2010 database, the mining height ranges from 3 ft to 18 ft with an average of 6 ft; the depth of cover ranges from 145 ft to 2,200 ft with an average of 892 ft; the entry width ranges from 9 ft to 37 ft with an average of 20 ft; the crosscut spacing ranges from 30 ft to 150 ft with an average of 78 ft; the center-to-center entry spacing ranges from 25 ft to 140 ft with an average of 70 ft; the crosscut angle ranges from 60° to 90° with an average of 85°; and the number of entries ranges from 2 to 10 with an average of 7. Also, all of the case histories use the same abutment angle of 21°. Table 2 presents the ranges of the input parameters of the ARMPS 2010 database.

STATISTICAL ANALYSIS

Logistic Regression

The present version of ARMPS-LAM discussed in this paper is a research-oriented design that is capable of batch solving numerous case histories. When ARMPS-LAM was initially coded, it was validated against some of the ARMPS case histories by manually performing the analysis and then comparing the results to the automated ARMPS-LAM results, as demonstrated with the example case history above. To further evaluate the overall accuracy of the ARMPS-LAM program and compare it to the accuracy of the previous ARMPS 2010 program, a statistical analysis of the ARMPS-LAM results for the complete ARMPS 2010 database was performed. To be consistent with the previous accuracy evaluation of ARMPS 2010 (Mark, 2010; Mark, Chase, and Pappas, 2007), a similar technique of logistic regression is used to evaluate the ARMPS-LAM results.

Table 1. AMZ stability factor determination.

Pillar	Row	Col.	Stress Safety Factor	Area Percentage within AMZ	Weighted SF
1	1	1	0.79	10.38%	0.08
2		2	0.83	10.38%	0.09
3		3	0.89	10.38%	0.09
4		4	1.01	10.38%	0.10
5	2	1	0.90	10.38%	0.09
6		2	0.92	10.38%	0.10
7		3	0.99	10.38%	0.10
8		4	1.02	10.38%	0.11
9	3	1	0.98	4.24%	0.04
10		2	0.98	4.24%	0.04
11		3	1.01	4.24%	0.04
12		4	1.02	4.24%	0.04
Sum				100.00%	0.93

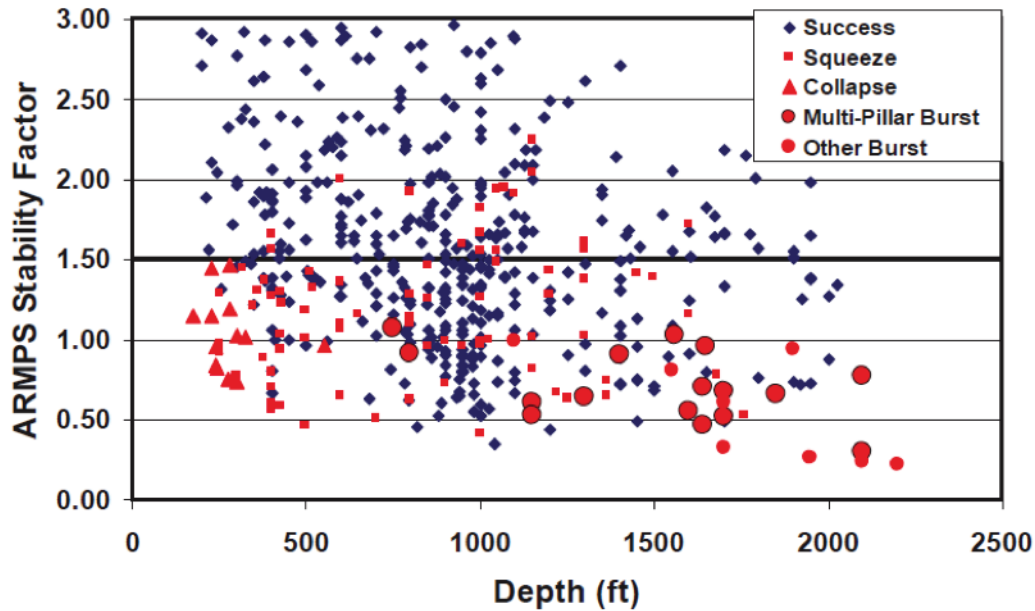


Figure 5. Recommended ARMPS SF (after Mark, 2010).

Table 2. Range of the input parameters of the ARMPS 2010 database.

Item	Min	Max	Average	Standard Deviation
Mining Height (ft)	3	18	6	2
Depth of Cover (ft)	145	2,200	892	436
Entry Width (ft)	9	37	20	1
Crosscut Spacing (ft)	30	150	78	18
Crosscut Angle (°)	60	90	85	11
Number of Entries	2	10	7	2

Logistic regression is the most commonly used multivariate statistical technique when the outcome, or dependent, variable is binary (in this case “success” or “failure”). In order to obtain the coefficients of the independent variable, the outcome has to be in “binary” form (i.e., “1” or “0”); therefore, the panel success was chosen to be equal to “1,” while the panel failure was chosen to be “0.” The goal of logistic regression analysis is to develop a model that can best fit the “outcome” using a combination of the independent variables. In the regression analysis, different mathematical models need to be compared to determine the best fit model. To compare the goodness-of-fit of different logistic regression models, the Receiver Operating Characteristic (ROC) curve is employed. Specifically, the area under the ROC curve (AUC) is proportional to the ability of a given model to achieve a correct discrimination between the two outcomes (success or failure) within the database. The AUC of the ROC curve ranges from 0.5 (no discrimination or same as chance) to 1 (perfect discrimination) and provides a goodness of fit parameter that is somewhat analogous to the R-squared parameter in linear regression (Hosmer and Lemeshow, 2000).

In the initial part of the logistic regression analysis, numerous independent variables and various ways of representing those variables were examined in order to determine the variables that most significantly affect the outcome (success or failure of the section). Ultimately, the five most significant variables, in order of significance, were found to be:

1. Stability factor of the AMZ (AMZ SF)
2. Mining height (h)
3. Stability factor of the barrier pillar (BP SF)
4. Depth of cover (H)
5. Panel width (P_w)

Table 3 presents the logistic regression results with the above variables. In the table, the coefficients represent the multiplicative factor of the variable; the standard error is an estimate of the data standard deviation; the z-value (z) is calculated by dividing the coefficient by the standard error, which provides an assessment of the portion of the standard error in the answer caused by each variable; and the P-value denotes the significance of the individual variable, where a variable is more significant as its P-value decreases. It was very assuring to see that the AMZ SF was the most significant variable, followed two spots down by the BP SF. However, the fact that the seam thickness (h) showed as being second most significant leads to a number of questions that will probably be addressed in future research.

Table 3. Logistic regression results for the model containing five variables.

Variable	Coefficient	Standard Error	z	P
AMZ SF	1.679	0.2905	5.781	7.42E-09
h	-0.2453	0.05689	-4.312	1.62E-05
BPSF	0.7733	0.2436	3.174	1.50E-03
H	1.406	0.3262	4.309	1.64E-05
Pw	0.0005436	0.0008502	0.6390	0.5226
Intercept	-1.185	0.7886	-1.503	0.1328

In the database, there were many ARMPS case histories (loading condition 1 and 2) where there was no barrier pillar,

but rather a solid block of coal. In this situation, it was not clear how to accurately determine/input a “barrier pillar” stability factor. Obviously this solid block of coal would have a very high stability factor, but the exact value was unknown. To handle this conundrum, it was decided to input the barrier pillar safety factor in a “binary” form around the safety factor of 1.5, where the BP SF is equal to 0, if the BP SF is smaller than 1.5 (unstable), and the BP SF is equals to 1 if the BP SF is larger than 1.5 (stable). The BP SF was input in this binary manner for the results shown in Table 3. Similarly, there appeared to be a binary effect in the database with the depth, so the depth of cover (H) was also input in binary form around 650 ft, with H equal to 0 if the depth was less than 650 ft, and H equals to 1 if the depth was greater than 650 ft. The other three variables—AMZ SF, h, and P_w—were input consistent with their real values.

Based on the relative significance of the above variables, six different logistic regression models with an increasing number of independent variables were developed and analyzed in order to determine the practical accuracy of the different models. Model No.1 only has the most significant independent variable, AMZ SF, and Model No. 2 has both the AMZ SF and the seam thickness (h). Model Nos.3 and 4 each have three variables; however, Model No.3 uses the depth (H) as the third variable and Model No. 4 uses the BP SF as the third variable. Model No. 5 has the four most significant variables, while Model No.6 contains all five of the significant variables. Table 4 contains the equations with the regression coefficients for each of the trial models.

Database Classification Comparison

According to the logistic regression performed by Mark (2010), ARMPS 2010 with only the one variable of stability factor(SF) achieved an AUC value of 0.7569, which implies good discrimination between the two outcomes (success or failure) within the database (see Table 5). In this initial logistic regression analysis of ARMPS-LAM, the single variable model with the AMZ SF only obtained an AUC value of 0.7026. However, Model No. 2, which contained both ARMPS SF and seam thickness (h) has an AUC value of 0.7958, a significant improvement and slightly better than ARMPS 2010. The next models, Nos. 3 and 4, only show a slight increase in the AUC with the additional variable (see Table 5). However, Model No. 5, with all four of the most significant variables, shows essentially the best AUC of 0.8206, since Model No. 6, with the addition of the panel width (P_w) does not show any significant improvement.

For ARMPS 2010, the design criteria were ultimately determined to be an ARMPS SF of 1.5 where 82% of the failed

Table 4. Logistic regression models with different variables.

Model No.	Model Equation
1	$g(x) = 1.331*AMZ_SF - 0.6285$
2	$g(x) = 1.172*AMZ_SF - 0.3300*h + 1.876$
3	$g(x) = 1.823*AMZ_SF - 0.2268*h + 1.410*H - 0.7674$
4	$g(x) = 1.060*AMZ_SF - 0.3424*h + 0.7625*BP_SF + 1.588$
5	$g(x) = 1.730*AMZ_SF - 0.2367*h + 0.7736*BP_SF + 1.442*H - 1.123$
6	$g(x) = 1.679*AMZ_SF - 0.2453*h + 0.7733*BP_SF + 1.406*H + 0.0005436*P_w - 1.185$

Table 5. Logistic regression results of different models.

Model	Variables of equation	Area under ROC curve (AUC)
ARMPS 2010	AMZ SF	0.7569
1	AMZ SF	0.7026
2	AMZ SF, h	0.7958
3	AMZ SF, h, H	0.8079
4	AMZ SF, h, BP SF	0.8061
5	AMZ SF, h, BP SF, H	0.8206
6	AMZ SF, h, BP SF, H, P_w	0.8207

case histories and 59% of the successful case histories were correctly classified, for an overall classification accuracy of 63% (see Table 6). The 1.5 value of stability factor for ARMPS 2010 was not the optimum overall classification point, but rather, the recommended stability factor was raised to more accurately classify the critical unsuccessful case histories. For the regression models of ARMPS-LAM, the same criteria of 82% accuracy in classifying the failure cases was applied, and the results are shown in Table 6. For Model No. 1 using only the ARMPS-LAM SF, the overall accuracy is only 55% (compared to 63% for ARMPS 2010). However, for Model Nos. 2 and 4, the overall accuracy is 66%, and for Model Nos. 5 and 6, the overall accuracy is 71%, significantly better than ARMPS 2010. Because Model No. 5 has essentially the same classification accuracy as Model No. 6 but with fewer variables, it is the preferred option.

Although, this is only a preliminary analysis of the ARMPS-LAM results, and a number of potential improved ARMPS-LAM algorithms will be tried in the near future, these initial results are very encouraging. In a direct comparison with ARMPS 2010, which correctly classified 82% of the failed case histories and 59% of the successful case histories, for an overall classification accuracy of 63%, the ARMPS-LAM analysis was able to accurately classify 82% of the failed case histories and 69% of the successful case histories for an overall classification accuracy of 71%. In the future, this classification accuracy may yet increase with more research efforts.

SUMMARY AND FUTURE WORK

In the research presented in this paper, a computer code (ARMPS-LAM) has been developed to effectively implement the laminated overburden model into the ARMPS program. This program takes the basic ARMPS geometric input for defining the mining plan and loading condition and then automatically develops, runs, and analyzes a full LaModel analysis of the mining geometry to output the stability factor on the AMZ, all without further user input. In particular, the program automatically calibrates the material properties, sizes the boundary pillars, sizes the mine model, generates the pillar and gob polygons, sizes the elements, fills the mine grid with material codes, creates a LaModel input file, runs the LaModel file, and determines the stability factor of the AMZ and barrier pillars. What would take a user several hours to accomplish manually, the ARMPS-LAM program accomplishes in about 5 minutes. From the user perspective, only

Table 6. Different models with same classification accuracy for failures.

Model	Successes, % of success/% of total cases		Failures, % of failures/% of total cases		Total, % of total cases	
	Correct classification	Incorrect classification	Correct classification	Incorrect classification	Correct classification	Incorrect classification
ARMPS 2010	59%/48%	41%/33%	82%/16%	18%/4%	63%	37%
1	49%/39%	51%/41%	82%/16%	18%/4%	55%	45%
2	62%/50%	38%/31%	82%/16%	18%/4%	66%	34%
3	59%/48%	41%/33%	82%/16%	18%/4%	64%	36%
4	63%/50%	38%/30%	82%/16%	18%/4%	66%	34%
5	68%/55%	32%/26%	82%/16%	18%/4%	71%	29%
6	69%/56%	31%/25%	82%/16%	18%/4%	71%	29%

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the traditional ARMPS input is required, and the output is the traditional stability factor of the AMZ.

The present laboratory version of ARMPS-LAM can be run in batch mode; therefore, the large ARMPS database of 645 case histories can be analyzed very quickly. In an initial logistic regression analysis of this database, the ARMPS-LAM program was seen to be slightly more accurate than ARMPS 2010 by a factor of 71% versus 63%.

Beyond just improving the accuracy of the ARMPS loading algorithm by implementation of a laminated overburden, the combination of the ARMPS-LAM implementation and the capability of batch analysis of the ARMPS database has the potential to quickly evaluate the utility of any number of new ground control approaches. For instance, the utility of a new abutment loading algorithm proposed by Tulu (2012), or the utility of using strain-softening coal properties (Karabin and Evanto, 1999) can both be quickly evaluated with the new program. Further, the ARMPS-LAM program outputs an AutoCAD drawing of the mine plan, as well as the input and output files from the LaModel analysis. This feature potentially allows the ARMPS-LAM program to be used as a preprocessor for further refinement of the mine plan or model input parameters beyond the initial automatic inputs, and the output file can be further analyzed beyond just the pillar stability factors. With all of this potential, it is anticipated in the future that the new ARMPS-LAM program will follow in the path of ARMPS and LaModel and further improve the safety of retreat room and pillar mining operations in the United States.

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Stability Analysis of Shell Structure Around the Longwall Mining Face Area in Steeply Dipping Seam

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ABSTRACT

Taking the longwall mining face area in a steeply dipping seam as the research object, the stability of an asymmetrical rock mass structure is analyzed using shell theory and masonry beam theory. The results show that an asymmetrical shell structure is formed at the top strata of the mining stope and its magnitude is mainly determined by the mechanical characteristic of the overburden strata of working face, mining space size and so on. Localized instability of the shell structure is the main factor that leads to the overall structural instability in the stope. The instability modes includes: the shell roof instability dominated by tensile failure, the shell shoulder instability initialized by compression-shear failure, and shell bases composite instability occurs in tensile and compression-shear modes. The shell structure stability dedicates rational strata control measures to choose, and the caving height of overburden strata is the key for shell structure instability.

INTRODUCTION

The steeply dipping seam is defined by the coal seam angle between 35° ~ 55° and has complex geological conditions. The reaction of the overburden structure and support-surrounding rock system in the steeply dipping seam for mining is very different from that in flat or moderately pitched coal seams. Numerous practices, experimental research and theoretical analysis indicate that (Wu, 2006; Wu, Yun, Zhang, 2000; Shi, 1999; Wu, Xie, Ren, et al., 2010; Huang, 2002; Zhou, Wu, Wu, et al., 1996; Huang, Wu, Li, 2009; Wu, 2004; Wu, Xie, Yang, et al., 2007; Wu, Xie, Yang, 2008) and inclined masonry structure is formed at the lower roof and asymmetrical shell structure is formed at the top strata of the mining area, which are the special structures for mining steeply dipping seam. Their stability dedicates rational strata control measures to choose. Studies on the stability of shell structure have great significance to the stability of incline masonry structure and support system in the longwall working face of steeply dipping seams.

MECHANICAL MODEL OF SHELL STRUCTURE AROUND THE MINING FACE AREA IN A STEEPLY DIPPING SEAM

In order to find out the shell structure's characteristic of stability to instability, the mechanical model of the spatial structure is

established, as show in Figure 1a. The spatial shell structure can be divided into section on the strike and incline direction, as show in Figure 1b and Figure 1c, the shell structure is made up of a conical shell and an ellipsoidal shell, which is named a composite shell structure. The conical shell is located at the shell shoulder area, and the ellipsoidal shell is located at the shell roof area.

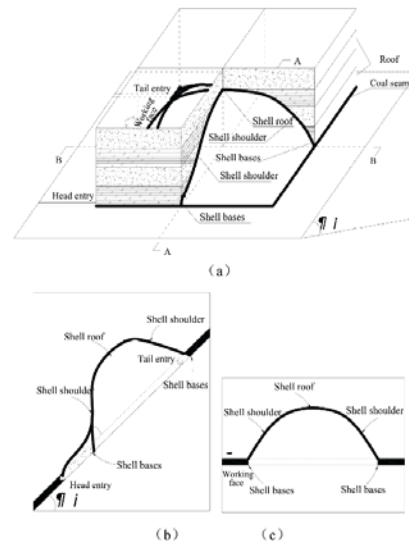


Figure 1. Shell structure around the mining face area: (a) three-dimensional model, (b) A-A section plane, (c) B-B section plane.

STABILITY ANALYSIS OF THE SHELL STRUCTURE

The shell structure can be regarded as a thin and uniform thickness shell structure which is loaded by the overlaying strata (Xue, 1988; Liu, 1987), and which is also a no moment shell, as shown in Figure 2. We have developed a hypothesis to the thin membrane shell as follows:

- (1) The ratio of shell thickness and middle surface curvature radius is much less than 1;
- (2) Deflection, against the shell thickness, is small;

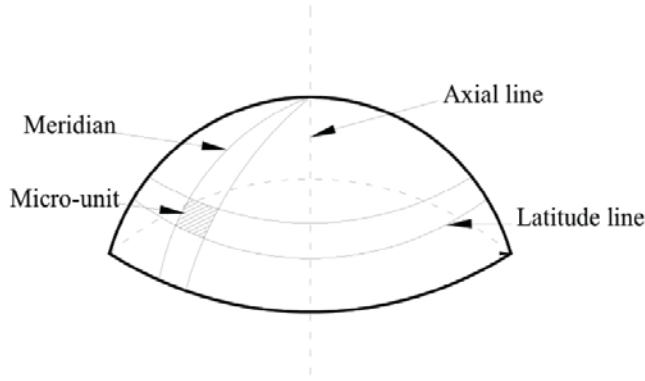


Figure 2. Shell structural model.

(3) After the shell bears bending, the orthogonal flat section, which through the shell and surface, still keeps flat and orthogonally after middle surface bending.

Take the micro-unit ABCD of shell structure in the steeply dipping seam mining area, and membrane force include circumference force N_θ and meridian direction force N_ϕ , the external force can be expressed by the component p_y and p_z , r_1 and r_2 is the main curvature radius of micro-unit, as show in Figure 3.

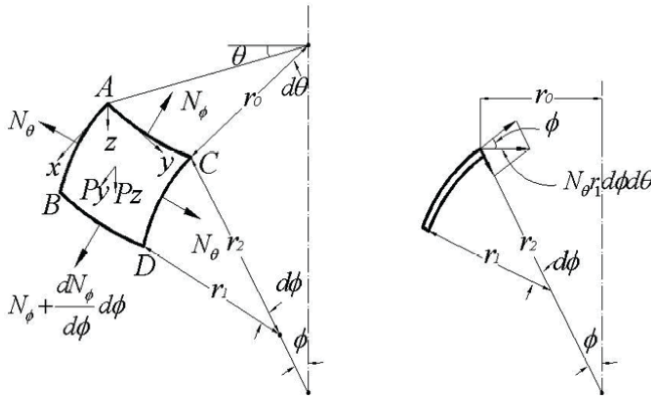


Figure 3. Shell structure micro model and its mechanics analysis.

Based on the micro-unit analysis, the equation: $N_\phi r_0 + N_\theta r_1 \sin \phi + p_z r_0 r_1 = 0$ can transform to one of the basic loading equations:

$$\frac{N_\phi}{r_1} + \frac{N_\theta}{r_2} = -p_z \quad (1)$$

On the meridian direction, y direction, the force balance can be

expressed as : $\frac{d}{d\phi}(N_\phi r_0) d\phi d\theta - N_\theta r_1 d\theta d\phi \cos \phi + p_y r_1 d\phi r_0 d\theta = 0$,

and divided by $d\theta d\phi$, the equation on the y direction force can transform to:

$$\frac{d}{d\phi}(N_\phi r_0) - N_\theta r_1 \cos \phi = -p_y r_1 r_0 \quad (2)$$

The basic stress equations of the shell structure are equation 1 and equation 2.

According to the feature of shell structure when mining the steeply dipping seam, the composite shell structure is made up of truncated conical shell and ellipsoidal shell, as show in Figure 4. Due to the periodic loading of the overlaying strata, one or more truncated conical shells are formed in the mining area, in other words, stack truncated conical shells, stress analysis of which can be regard as a stack of more truncated conical shells.

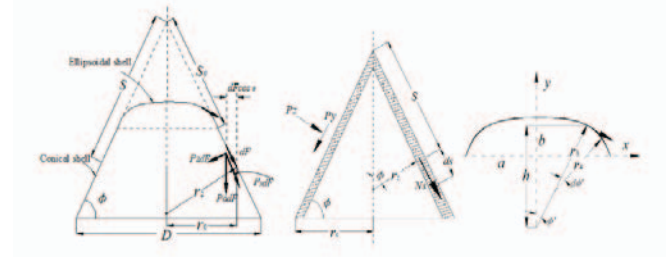


Figure 4. Mechanical analysis of rock structure: (a) composite shell, (b) conical shell, (c) ellipsoidal shell.

As show in Figure 5, in the truncated conical shell, ϕ (strata caving angle) is a constant, and using coordinate S instead of ϕ , S means the distance from the apex to the point on the generatrix. And we mainly think about the stress induced by gravity (P_G), the component of which as show in Figure 5b, the stress formula of truncated conical shell is as follow:

$$\sigma_{\theta(\text{conical})} = -p_G S \frac{\cos^2 \phi}{t \sin \phi} \quad (3)$$

$$\sigma_{s(\text{conical})} = -\frac{p_G}{2St} (S^2 - S_0^2) \cot \phi \quad (4)$$

Similarly, the stress formula of ellipsoidal shell is as follow:

$$\sigma_{\phi(\text{ellipsoid})} = \sigma_{\theta(\text{ellipsoid})} = \frac{p'_G \cos \phi' a^2}{2bt'} \quad (5)$$

INSTABILITY MODE OF THE SHELL STRUCTURE

Numerous practices, experimental research and theoretic analysis indicate that the key reason of the overlying strata instability in the steeply dipping mining area is local failure of rock mass, which is a typical localization phenomena, the instability pattern include shear failure between rocks and crushing failure at the contact point. Therefore, the local failure of shell structure (shell base, shell shoulder, shell roof) is the main factor of the

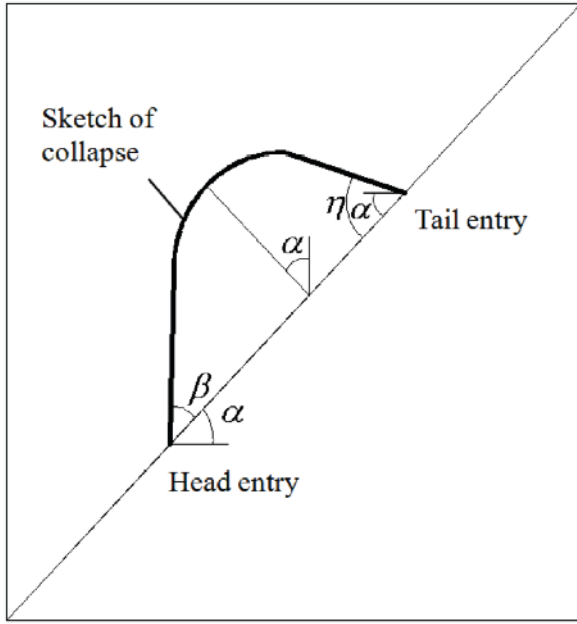


Figure 5. Dip angle relations of shell structure on the inclined section.

intact structure's collapse, so studying the stability of the shell base, shell shoulder, shell roof is the key point to reveal disaster-causing mechanism of overburden. Generally, the surrounding rock failure condition is as follow:

$$\sigma_{\max} > [\sigma] \quad (6)$$

Where: $\sigma_{\max}$ is the maximum stress of rock, $[\sigma]$ is the strength of rock in the corresponding stress state.

The instability mode of shell structure is as follows:

(1) Instability Mode of Tension Failure on the Shell Roof

The shell roof strata are subjected to squeezing force which from shell membrane stress $|\sigma_{\text{ellipsoid}}|$. If the strata strength is large and the strata don't fail under the squeezing action, then the maximum tension stress in the strata is:

$$\sigma_{T(\text{rock})} = \frac{2C \cos \varphi}{1 - \sin \varphi} + |\sigma_{\phi(\text{ellipsoid})}| \frac{1 + \sin \varphi}{1 - \sin \varphi} = \frac{2C \cos \varphi}{1 - \sin \varphi} + \left| \frac{\gamma' \cos \phi' a^2}{2bt'} \right| \frac{1 + \sin \varphi}{1 - \sin \varphi}$$

Namely, when the approximate maximum normal stress which

exerted on the hanging rock strata $\sigma_{\max(\text{rock})} = \frac{qL^2}{2h^2} > |\sigma_{T(\text{rock})}|$, then the

shell roof encounters tension failure; When the shell roof on the horizontal condition (strike section of working face), the tension failure instability condition is:

$$\sigma_{\max(\text{rock})} = \frac{qL^2}{2h^2} > \sigma_{T(\text{rock})} = \frac{2C \cos \varphi}{1 - \sin \varphi} + \frac{\gamma a^2}{2b} \left(\frac{1 + \sin \varphi}{1 - \sin \varphi} \right) \quad (7)$$

When the shell roof on the inclined state (inclination section of working face), as show in Figure5, the shell roof is subjected to uniformly distributed load, then the tension instability condition on the shell body is:

$$\sigma_{\max(\text{rock})} = \frac{q \cos \alpha L^2}{2h^2} > \sigma_{T(\text{rock})} = \frac{2C \cos \varphi}{1 - \sin \varphi} + \frac{\gamma a^2 \cos \alpha}{2b} \left(\frac{1 + \sin \varphi}{1 - \sin \varphi} \right) \quad (8)$$

Where: C is rock cohesion, φ is friction angle, q is loading per unit length (Qian, 1983; Qian and Shi, 2000), L is the hanging strata length, h is rock strata thickness, t' is total overlying rock strata thickness, γ is average unit weight of the rock strata above the shell, α is seam dipping angle, a and b are the length of long and short axis of ellipsoid shell respectively.

The instability analysis of the shell roof shows that the tension failure mainly occurs at the shell roof area, which is on the top of the mining area and belongs to top instability.

(2) Instability Mode of Compression-Shear Failure on the Shell Shoulder

This situation mainly occurs at the shell shoulder zone where is consisted of many rock layers and overlying strata, its instability criterion is:

On strike section of working face, instability condition due to the shell shoulder subjected to squeezing action:

$$|\sigma_{S(\text{conical})}| = \left| -\frac{\gamma}{2St} (S^2 - S_0^2) \cot \phi \right| > \sigma_{P(\text{rock})} \quad (9)$$

On inclined section of working face, instability condition due to shell shoulder, where at the top of inclined panel, subjected to squeezing action:

$$|\sigma_{S(\text{conical})}| = \left| -\frac{\gamma}{2S} (S^2 - S_0^2) \cot(\eta - \alpha) \right| > \sigma_{P(\text{rock})} \quad (10)$$

On the same inclined section, instability condition due to shell shoulder, where at the bottom of inclined panel, subjected to squeezing action:

$$|\sigma_{S(\text{conical})}| = \left| -\frac{\gamma}{2S} (S^2 - S_0^2) \cot(\alpha + \beta) \right| > \sigma_{P(\text{rock})} \quad (11)$$

Where: t is total rock strata thickness above the shell, γ is average unit weight of the overlying rock strata, α is seam dipping angle, η is caving angle at the top of inclined panel, β is caving angle at the bottom of inclined panel, S_0 is head length between the head

and intersection of composite shell and ellipsoidal shell, S is length between head and one point of shoulder.

The results indicate that the rock mass located at the shell shoulder is less likely instability, especially the shoulder at the bottom of inclined panel, but on the top of inclined panel, the instability of shell shoulder is more possible occur than the bottom because of small angle and large $|\sigma_{S(\text{conical})}|$, and also where the collapsing instability of rock mass is induced, intense pressure and impact would destroyed the local supports, and leaded to “support-rock” system dynamic disaster.

(3) Composite Instability Model of Compression-Shear and Tension Failure at the Shell Base

On strike section of working face, shell bases are located at the rear of the gob and front of the working face. For the shell base in front of the working face, there are two instability criterions: the first one is shell base rock squeezing instability, which presents in the form of roof sag and coal rib spalling. The instability condition for this case is:

$$|\sigma_{S(\text{conical})}| = \left| -\frac{\gamma}{2St}(S^2 - S_0^2)\cot\phi \right| > \sigma_{P(\text{rock})} \quad (12)$$

The second is shear failure of the hanging rock over the shell base suffering the load from the overlying strata which presents in the form of roof cutting, etc. The instability condition for this case is:

$$\tau_{\max(\text{rock})} = |\sigma_{s(\text{conical})}| > \tau_{s(\text{rock})} \quad (13)$$

On the inclined section of working face, Figure 5, the instability condition due to the squeezing failure at the top shell base is:

$$|\sigma_{S(\text{conical})}| = \left| -\frac{\gamma}{2S}(S^2 - S_0^2)\cot(\eta - \alpha) \right| > \sigma_{P(\text{rock})} \quad (14)$$

On the same inclined section, the instability condition at the bottom shell base is:

$$|\sigma_{S(\text{conical})}| = \left| -\frac{\gamma}{2S}(S^2 - S_0^2)\cot(\alpha + \beta) \right| > \sigma_{P(\text{rock})} \quad (15)$$

Different shell base instability models occur in different areas, periodic shell base instability occurs in coal wall ahead the working face. The shell base at the bottom of inclined section is less likely instability; because the shell base is locate at the filled rock and coal wall area, which is also affected by the caving and slipping strata. The shell base at the top of inclined section, which is located at the area of tail entry, is easiest failure because there is a large movement space under the shell base due to caving strata slipping down.

CAVING HEIGHT OF OVERBURDEN

Studies on the caving height of overburden not only have great significance to the stability analysis of the shell structure, but also attaining the location of key strata, meanwhile, the caving height is also an important criterion parameter of instability of the shell structure.

Shell shoulder and shell base occurs periodic instability along with the working face ahead, but not to the shell roof which is pressed by the horizontal force. Therefore, the caving height (height of shell roof) is calculated by the tension failure criterion of the shell roof:

When the shell roof on the horizontal condition (strike section of working face), the critical condition of tension failure is:

$$\frac{qL^2}{2h^2} = \frac{2C \cos \varphi}{1 - \sin \varphi} + \frac{\gamma a^2}{2b} \frac{1 + \sin \varphi}{1 - \sin \varphi} \quad (16)$$

The relation of a , b , L is:

$$a = b \cot \phi + L/2 \quad (17)$$

b can be approximate to thickness (h) of downward stratum which next to the top shell roof, Figure 4c, uniform load q is:

$$q = \frac{E_{t1}h_1^3(\gamma_1h_1 + \gamma_2h_2 + \dots + \gamma_nh_n)}{E_{t1}h_1^3 + E_{t2}h_2^3 + \dots + \gamma_nh_n} \quad (18)$$

Where h_i is thickness of upward stratum, γ_i is density, E_n is Elastic modulus ($i=1, 2, \dots, n$), and:

$$L = \sqrt{\frac{2bq(1 - \sin \varphi)}{4bC \cos \varphi + \gamma a^2(1 - \sin \varphi)}} \cdot h \quad (19)$$

Substituting formula 17 and 18 into formula 16, the length of hanging rock strata L will be obtained.

Base on the physical and numerical simulation results, the caving angle on the strike section is commonly during $45^\circ \sim 65^\circ$, the maximum hanging length L can be obtained by back calculate from the immediate roof one by one, if $L > D$ (advancing distance), Figure 4a, the stratum is stable, if $L < D$, the caving height on strike section can be obtained and the caving height on inclined section can be calculated by using the caving angle and coal bed angle.

CONCLUSIONS

(1) Asymmetrical shell structure, which is formed in the overburden, is a kind of special structure in mining steeply dipping seam. The magnitude of the “Asymmetrical shell structure” depends on the overlaying strata characteristic and the mining spatial.

(2) The key reason of the overlying strata instability in steeply dipping mining area is local failure of rock mass, generally,

the rock instability mode of the key part of stress arch-shell in steeply dipping seam includes compression failure instability, tension failure instability, shear failure instability and composite failure instability.

(3) The height of shell and length of hanging rock strata can be determined by the following equation:

$$\begin{cases} L = \sqrt{\frac{2bq(1-\sin\varphi)}{4bC\cos\varphi + \gamma a^2(1-\sin\varphi)}} \bullet h \\ q = \frac{E_{t1}h_1^3(\gamma_1h_1 + \gamma_2h_2 + \dots + \gamma_nh_n)}{E_{t1}h_1^3 + E_{t2}h_2^3 + \dots + E_{tn}h_n^3} \\ a = b\cot\phi + L/2 \end{cases}$$

ACKNOWLEDGMENTS

The author would like to gratefully acknowledge the support for this project from the National Natural Science Foundation of PRC (51074120, 51204132) and Scientific Research Program Funded by Shaanxi Provincial Education Department (11JK0785).

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Deformation Mechanism and Floor Heave Control Technology in Deep-Buried Full Seam Chamber

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ABSTRACT

With the intensification and expansion of deep coal mining, the surrounding rock stress environment becomes further deteriorated because of the complicated geological environment and powerful mining effect, so the major deformation of roadway appeared frequently. The proportions of floor heave are increasing and , the most serious problem is the floor heave of preparatory workings. The coal seam of Binchang coal area in Shaanxi province is at a depth of 621.6 ~ 817.0 m, the average thickness of the coal seam is 25 m, and the overburden and coal measure strata are the main regional aquifers and are highly permeable. The properties of water-weakening are obvious in the floor strata. The main preparatory workings or chambers were driven in the coal seam. The severe floor heave of the water chamber appeared during early mining and the amount was 180 ~ 910mm, and seriously affected the normal installation and use of the water chamber. Theoretical study and numerical simulation as well as the in-situ measurements were carried out to investigate the deformation and failure characteristic of the surrounding rock with non-support in the floor and the main reasons which caused the floor heave were determined. A new combined support technology was proposed which consists of overcutting, grouting, backfilling and inverted-arch and bolt-grouting. The field monitoring showed that the cumulative convergence of the water chamber was 20mm, the maximum convergence speed is 1.2 ~ 2mm/d, the accumulative convergence of two ribs was 36mm and the maximum convergence speed is 2mm/d. The combined support technology will enhance the support intensity between the roof and floor, the two sides of the entry and improve the loading capacity of the floor effectively and the floor heave can be controlled effectively.

INTRODUCTION

As the depth of coal mining increases, the surrounding rock stress environment becomes further deteriorated because of the powerful mining effect and the complicated geological environment, such as higher stress, weaker strata and more geologic structure. As a result the major deformations of roadway appeared frequently (Kang, Wang and Lin, 2010. The proportions of floor heave are increasing with the increase of mining depth (Xid and Chang, 2010 and Jiang, Zhao, and Liu, 2004), especially, the most serious problem being the floor heave of preparatory

workings (Bai, et al., 2011). More strict requirements are needed in the chamber supports because of the complex structures the high stress state and the longer length of service needed. Floor heave is non-linear instability produced under the combined effects of the original rock stress the mining pressure and the stress state of the roadway's sides, and its mechanism is closely related with the geological conditions, floor rock properties and stress state of roadways (An, Zhang, and Ni, 2008 and Zhang, et al., 2011). Some results (Wang, Feng, and Tao, 2005, Hou, et al., 1995, and H, et al., 2009) have shown that there are obvious structure effects between the roof, floor and the bottom angle of roadway as a result of the mining pressure. The floor is influenced easily with any changing of the mechanical properties and the stress state, so the support consisted of reinforcing the sides and the bottom angle and the roof for an effective method to control floor heave of the roadway openings. The deformation mechanism under the higher stress and powerful mining pressure and the main influencing factors were analyzed based on the severe deformation behavior of the water sump chamber in Hujiahe colliery during the early days of excavation. A new complementary supporting approach which consists of overcutting-grouting-backfilling and inverted-arch and bolt-grouting was put forward in view of the comprehensive idea.

STRATA STRUCTURE CHARACTERISTICS OF THE MINING AREA

Binchang mining area is one of the largest national coal production areas and is located in the middle of Huanglong Jurassic Period coal field in Shaanxi province. The primary mineable coal bed (4 coal seam) is located in the Jurassic Period Yan'an group characterized by small anticlines and synclines because of past geologic activity and is at a depth of 621.6 ~ 817.0 m. The thickness of the coal seam is 13.0 ~ 26.3m and the average is 25 m, and the overburden and coal measure strata are the main regional aquifers which is highly permeable. The immediate roof of NO.4 coal seam is sandy mudstone with a slight interlayer of sandstone and the main roof usually consists of medium sandstone and strong coarse sandstone. There are mainly some unconsolidated organic mudstones which weaken rapidly in the floor mass in the presence of water.

DEFORMATION CHARACTERISTICS OF THE FULL SEAM CHAMBER

Mechanical property of surrounding rock

Numerical simulation code FLAC2D is utilized to analyze the stress distribution and the deformation and damage of the entries. The numerical model of the water sump chamber is set up based on the geologic conditions and the Mohr-Coulomb strength criterion of materials and brick elements and cable units are selected for the simulation of the rock mass and bolt or anchor member. The vertical displacement was fixed at the bottom of model and the side horizontal displacement was fixed and the overburden equivalent load was applied on the upper of the model. All the mechanical parameters of strata are showed in Table 1.

The pressure distribution of the surrounding rock was changed after mining. The roof and floor is in the distressed condition and extend further as the calculation advances further, the maximum range of fracture zone in floor is 4 ~ 5m which is larger than the ones obtained from the roof. It is the main factor that influences the type and the size of support. The region of stress concentration occurs on the ribs of the roadway and the maximum abutment is double the stress of intact rock as showing in Figure 1.

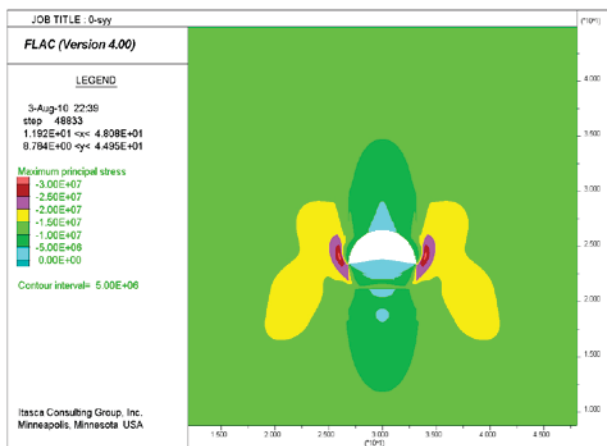


Figure 1. The vertical stress vector of surrounding rock with non-support.

According to the calculation results, the larger displacement of the rock mass with non-support is located at the floor and roof. The maximum displacement is 633mm at the middle of the floor and the maximum sagging is 106mm at the middle of the roof, the maximum displacement of the ribs of the roadway is 69mm. There is an obvious floor heave as shown in Figure 2.

Field observation of deformation characteristics

The water sump chamber of Hujiahe colliery was drifted in the coal seam with the thickness of 2.4 ~ 3.1m along the floor and the cross-section is a semi-circle arch with a width of 5.5m. The primary support is anchor net fixed to the roof arch with seven bolts. The row & line spacing is 0.7m & 0.8m, the length of the

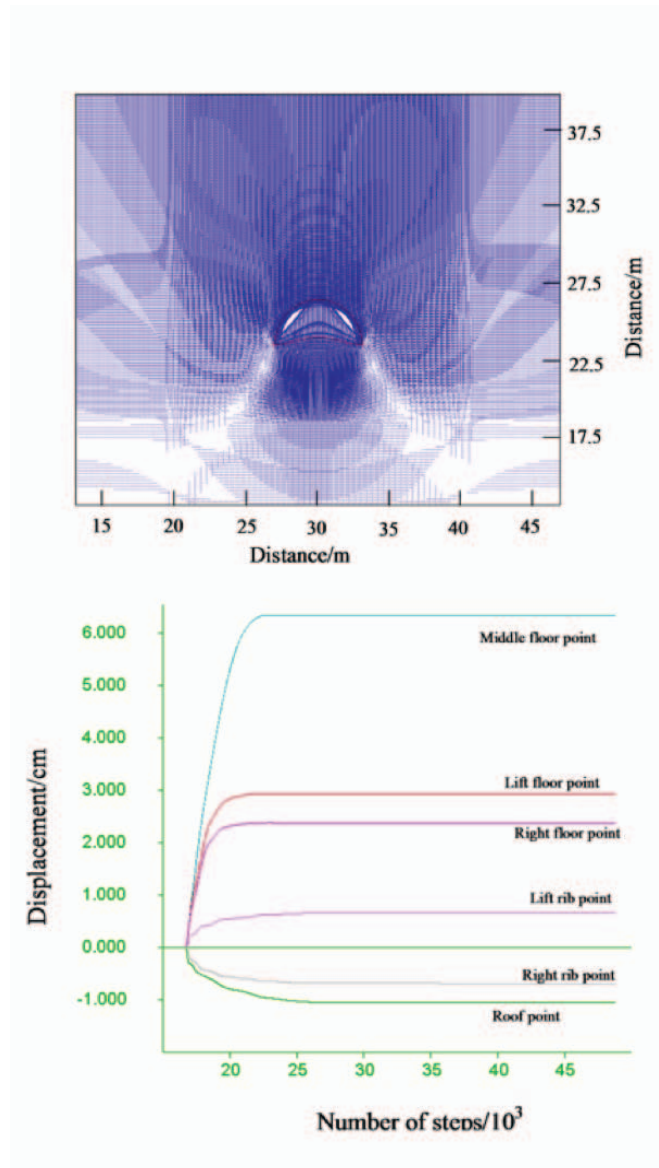


Figure 2. Vertical displacement vector of the roadway with non-support.

bolts is 2.0m and the pulling resistance is at least 50 kN. The primary support section is shown in Figure 3. The measurements of deformation were carried out on the 50-metre length of heading in May, 2010. The partial roof subsidence was felt during the early excavation process and floor heave occurred approximately 15 ~ 20m behind the working face. And the maximum convergence between roof and floor of the roadway was 600 ~ 910mm (Table 2) which seriously affected the following concrete spraying and normal use.

Deformation mechanism of the full seam chamber

The main reasons or influencing factors of the severe deformation are listed in the following based on the results of research:

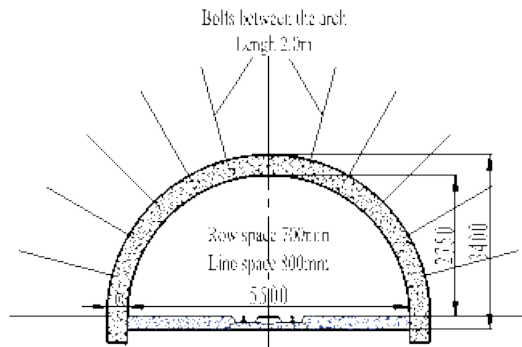


Figure 3. The primary support section.

- (1) Water- weakening characteristic of the floor rock mass. There are some unconsolidated organic mudstones that will weaken rapidly when coming in contact with underground water. Then expansion pressure was produced while fissure water of roof and surrounding coal seam permeated along the secondary interface without floor support during the excavation process. It is in fact that this is the main reason for the floor heave.
- (2) The higher stress is also an important condition of the deformation. The water sump chamber located at the depth of approximately 710m was mined in the coal seam. The shear failure occurred towards the free space of roadway caused by higher stress and the additional pressure concentration caused by mining effects. And the tensile failure of the floor appeared under the under the action of extrusion on both ribs transferring from the roof stress, resulting as floor heave.
- (3) The insufficient primary support. The primary support is put forward using the engineering analog method. Combining the roof arch bolting and the partial anchor cable isn't able to efficiently become self-bearing and self-supporting. The stability of the roof and the sides was easily destroyed by the floor without supporting. The roof subsidence and the floor heave damage were induced by the additional roof pressure.
- (4) Obvious Secondary mining effect. The pressure distribution of the surrounding rock was affected frequently because of the location and the geological conditions. There is an obvious non-linear increasing of surface displacement due to the weak coal seam in the sides during the excavation of the preparatory workings such as the main and vice-vertical shaft, shaft station and main roadway.

FLOOR HEAVE CONTROL TECHNOLOGIES AND CONSTRUCTION METHOD

Each entry consists of ribs, roof and floor, and they will interact with each other mutually. The single support elements combine into a united support which is the key to achieve the integration of support and homogenization of load Jiang, et al., 2009 and Wu, et al., 2011. The plastic flow towards to the place of the floor was induced easily under the additional mining stress around the roadway and due to the general weaker floor mass. So the effective measures should be carried out based on the before-mentioned analysis, especially to strengthen the shear capacity to control the floor heave. A new multiple supporting approaches consist of overcutting-grouting-backfilling and inverted-arch reinforced beam and spouting is put forward in view of the integration idea, as

shown in Figure 4. This technique is shown under the consideration of the achievements and the principle of support research.

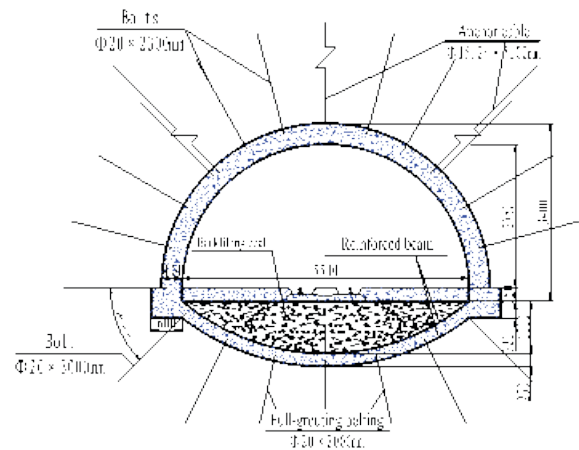


Figure 4. Inverted-arch support structure.

Its main construction method list as follows:

- (1) Based on the original design, the bolting parameters with a length of 2.3m and row & line space of 0.7m & 0.7m were amended and the anchor cable with a length of 7.1m and row space of 2.8m were applied and the pulling resistance is at least 100kN. The decrease of the roof separation is caused by the bolts which fastened the immediate roof; the self-supporting capacities of deep roof are mobilized by one or two pairs of anchor cable which were fixed alternately.
- (2) Firstly, the coal seam of the floor was excavated to a depth of an additional 1.25m and the cross-section was formed into an inverted-arch. Then the full-grouting bolting with the length of 2.0m and row space of 1.0m and were installed, the reinforced beam was lay along tangential direction to close the exposed floor. The thickness of the beam is 0.25m and the line interval is 2.0m. Finally, coal gangue or coal was filled on the floor after waterproofing was completed.
- (3) The bolts with higher pre-stress were fixed at the bottom angle of the sides along the direction of 45 degree and the length is 3.0m and the line interval is 2.0m.
- (4) The spraying of the entire section was carried out to reinforce the surrounding rock and the thickness of concrete is 0.4m and the foundation thickness of semi-circle arch is 0.25m. The united support structure was forming to control the crack seepage.

The fundamental advantage of the schemes presented above is a considerable improvement of roof strength. This improves the capacities of its self-supporting by applying several roof bolts or long anchor cable and fixing bolts in the sides for reinforcement of surrounding rock mass. The application of active support in roof and sides also reduces the stress concentration in floor. With the intensity of sides strengthened, the effective width of floor is reduced and the ranges constrained by side for the floor heave is shortened.

The next advantage is reinforcement of the floor using the inverted-arch and reinforced beam and grouting. The combing

support is significant to limit expansion pressure and control the sustained large deformation of floor. With the longer rigid bolt, the pressure from side decreased and the shear resistance at corner is improved.

RESULTS OF MEASUREMENTS

The support effect and convergences of surrounding rock were estimated based on the coupling support. The largest displacement of the roof is 56.3mm and was reduced by 44% ~ 50% and only 111.9mm in the floor which was reduced by 80% ~ 85% in comparison with unsupported state. Obviously, the new coupling support is an effective method to control the large deformation of full-seam roadway in deep mining, as shown in Figure 5.

Measurements of convergence were carried out and the results indicate that the surface displacement decreased significantly and the maximum value of convergence between roof and floor of the roadway achieved 20 mm and the highest velocity was 1.2 ~ 2mm/d, the two sidewall accumulative convergence value of roadway achieved 36 mm in maximum and the highest velocity was 2mm/d in inner chamber. The maximum value of convergence between roof and floor of the roadway achieved 19mm and the highest velocity was 1.0 ~ 1.2mm/d in the outer chamber. There were no obvious displacements or floor heave after using the new support approach in September, 2011.

CONCLUSIONS

Based on the simulated results and the underground measurements with the coupling support, the following conclusions may be achieved:

- (1) The higher stress and the weaker floor as well as the mining effects have great influence on the surrounding rock stability which is the main reasons of the large deformations. The loading capacity of the floor can be exploited to achieve an optimal support effect.
- (2) The inner bearing structure was formed in the deep floor and allowed for better protecting of the floor against continuous damage and also improves the pressure state and shear resistance at corner. The combined support guarantees the unit of stability for a long period of time.
- (3) Taking into consideration advantages, the combined system of inverted-arch and reinforced beam and grouting plays a more essential role in the deep-buried roadway with the higher stress and weaker floor characteristic.

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Establishment and Application of Deformation Monitoring System of Roadway Based on the Three-Dimensional Model Experiment

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ABSTRACT

Roadway deformation and stress migration occur with uncertainty and randomness because of geology, tunneling and mining influence. In order to study the surrounding rock deformation and failure mechanisms in three-dimensional stress state, an air return roadway of Yuanyang Lake mine of Shenhua Ningxia Coal Industry Group Co., Ltd. was chosen as a research object. The three-dimensional model was a designed and completed experiment with a dynamic-static coupling load. The model was loaded by hydraulic cylinder, simulating the in-situ rock stress field and the induced stress field of surrounding rock. The deformation monitoring system of the roadway model was constituted by optical total stations, borehole observation instruments, acoustic emission monitoring, and stress sensors. A wealth of data was obtained through an omni observation of surface displacement, characteristics of stress distribution, and deformation and failure of surrounding model material. By acoustic-optic-electric methods, a real-time testing on physical, mechanical, and damage parameters was realized through the experimental process. The results showed that the data of deformation and failure obtained from the deformation monitoring system of the roadway were authentic, which has great significance for warning and forecasting safety hazards of surrounding rock in coal mines.

INTRODUCTION

The study of the simulation experiment refers to the production of a model similar to a prototype in the laboratory. The authors used the test instrumentation to observe the mechanical parameters and its distribution pattern in the model. Using the results of the study on the model, inferring mechanical phenomena that can occur in the prototype, and taking into consideration the rock pressure distribution law, we can work to resolve practical problems in mining engineering. A similar simulation experiment is a laboratory research method, based on the similarity theory and dimensional analysis, which has become one of the more effective research methods to solve the problems of ground control in mining engineering. Previous studies that mainly deal with plane strain and plane stress models fail to resolve the boundary effect of the model, loading and testing approaches, and some other related problems. There is significant difference between the experimental results and engineering practice. It is relatively ideal for the research object

under the three-dimensional stress state to adopt three-dimensional loadable simulation experiment, but there are some difficulties in model building. The enlargement of experimental devices and the selection of the related loading system are the bases for realizing the simulation. The measurement of physical quantities is the key for achieving the experimental objective. Therefore, it is necessary to establish a roadway deformation monitoring system in three-dimensional simulation experiment. It can lay the groundwork for analyzing the deformation and failure mechanisms of surrounding rock and provide a scientific basis for determining and optimizing the reasonable support parameters of the roadway. In this paper, an air return roadway of Qingshuiying coalmine of Shenhua Ningxia Coal Industry Group Co., Ltd. was chosen as the research object, where we established a deformation monitoring system of the roadway and completed a static-dynamical three-dimensional loadable simulation experiment.

ESTABLISHMENT OF DEFORMATION MONITORING SYSTEM OF ROADWAY

The deformation monitoring system of the roadway consists of the PENTAX R-322NX optical total station, SWAES digital intelligent acoustic emission monitoring, YS (B) borehole observation instrument, and the 108 road pressure data acquisition system, as shown in Figure 1.

It is possible to get the surface displacement data of the rock surrounding the roadway through the PENTAX R-322NX optical total station, and the visualization program will transform the observation data. The drawing software draws the roadway strata movement process directly. It can successfully solve the difficult problem of the deformation observation in the three-dimensional model experiment.

SWAES acoustic emission monitor

The phenomenon of material gathering energy to produce transient elastic wave is called acoustic emission (AE). AE is a common physical phenomenon; most material has acoustic emission when it deforms and fails. Presently, AE technology can monitor the sound source widely: plastic deformation of material, formation and expansion of cracks, steel fiber fracture, matrix cracking, interface separation of rock base composite,



Figure 1. Deformation monitoring system of roadway.

friction, fracture and instability of hard rock. In fact, the band of a detected acoustic emission signal is very wide, from dozens of Hz to 2 MHz, and the amplitude is lower, so we must use sensitive electronic instrument to test. The elastic wave from the sound source propagates in the material and is received by a sensor that is placed on the surface of object. The sensor will transform the received waveform into an electrical signal, then amplify it through a signal amplifier treatment, and the background noise and invalid signal components are removed by the filter. The effective signal goes through amplification again, is then processed by the computer, and then formed the signal parameters of AE. With this monitoring system, we use an AE monitor to observe and forecast the destruction and instability of surrounding rock.

YS (B) borehole observation instrument

The YS (B) borehole observation instrument is used for observing rock looseness and fissures in any direction, identifying lithology and probing the situation of the drilling, detecting roadway roof abscission layers and fissures, managing drilling pictures, and video recording and logging. The YS (B) drilling observation instrument uses a high-resolution probe and color display device that can distinguish between a 1 mm crack and different lithology. Connecting with the computer directly, it can easily display the image in real time. The accompanying software can manage pictures and video data files, add text and comments to borehole data, play drilling images and audio video, and has drilling retrieval functions. Using the YS (B) borehole observation

instrument, we can monitor the deformation and failure of the surrounding rock.

108 road pressure data acquisition system

The distribution regularity and the characteristics from ground stress, or excavation disturbance stress, play a decisive role during the excavation, supporting design, dynamic damage, and instability control in underground engineering. Obviously, surrounding rock deformations in the process of an underground excavation is caused by the evolution of the stress field. In order to obtain the characteristics and rules of stress distribution in different embedded depths (horizons), we can use the CL-YB-114 pressure sensor and the 108 road pressure data acquisition system to test the surrounding rock stress.

THE ROADWAY DEFORMATION MONITORING OF THE THREE-DIMENSIONAL MODEL EXPERIMENT

The Engineering General Situation

The roof and floor rock of this air return roadway are soft and broken, and the degree of consolidation is poor. Its main roof is kern stone and poor aquifer, rock fissure-developed, and the ability of resisting deformation is weak. The shape of the roadway is a straight wall semicircle arch. The support way is bolt-mesh support and bolt-shotcrete support. Different degrees of local instability and failure appear in the construction; it is mainly roof falling in small areas. The maximum height is 3.5m; the length of roof falls is between 4.5–7.2m. The excavation depth of the roof falling area is 250–350m; its overburden pressure is 8MPa. The average dip angle of the roadway is 26.5°.

Establishment of the Three-Dimensional Model

(1) Experimental parameters

- 1) The section size of roadway: wide 4.8m, height 4.2m
- 2) Model size: 2.50m×2.25m×1.95m
- 3) Similar conditions of what?

In order to achieve the similar process of surrounding rock deformation and failure in the roadway, the excavation between the model and the prototype must be determined to be reasonably similar conditions. Through the feasibility analysis, the main conditions of similarity are as follows:

Geometric similarity constant: $C_l = l_p / l_m = 10$; Displacement similarity constant: $C_s = C_l = 10$;

Density similarity constant: $C_y = y_p / y_m = 1.6$; Force/load similarity constant: $C_\sigma = C_l C_y = 16$;

Time similarity constant: $C_t \sqrt{C_l} = \sqrt{10}$

p is prototype, m is model.

(2) Similar material

To ensure the process and results of similar simulation experiments are more consistent with the actual engineering, we considered the role of the far field stress factors, the structure of the surrounding rock (mixed gneiss), and the strength characteristics in the process of the model pavement. We used the experimental apparatus of the side guard plate and the high toughness of the concrete block to solve the lateral stress. Concrete block has high toughness. To a certain extent, it represents a certain toughness of gneiss, with a mixed sedimentary environment and features.

From the field sampling and rock mechanics performance test, we obtained the main parameters of the similar simulation experiment. Based on the similar relationship of various parameters between the model and the prototype, we determined the reasonable ratio of similar materials, sand as aggregate, gypsum and calcium carbonate as binder. According to the ratio of similar materials, we mixed the materials and stirred well and then added the right amount of water and stirred well again to pave the model. It strengthened, and we used mica as the interface layered material. The ratio of similar materials is shown in Table 1.

Table 1. The table of ratio of similar materials in the three-dimensional model.

No.	rock name	rock stratum thickness(m)	ratio	model dimension(cm)			quality of similar materials (kg/cm)			
				length	width	height	sand	gypsum	calcium carbonate	coal
1	siltstone	6.68	537	250	195	66.8	26	1.56	3.64	
2	medium-grained sandstones	1.3	546			13	26	2.08	3.12	
3	mudstone	3.11	646			31.1	26.7	1.78	2.68	
4	coal	7.88				78.8	7.8	0.5	1.56	5.85
5	siltstone	4.92	537			49.2	26	1.56	3.64	

(3) Model pavement

This experiment used a three-dimensional simulation experiment device. The design scheme is shown in Figure 2. The layout of the monitoring system is shown Figure 3.

Experimental Process

After paving the model and letting it dry naturally, we carried on with the experiment. We used hydraulic loading to simulate the buried depth of the roadway and excavation disturbance. To facilitate the analysis and to study the surrounding rock deformation characteristics in the process of the excavation disturbance, we loaded and unloaded 42 times in this experiment.

In order to simulate the in-situ stress field of the roadway, at the start of the experiment, lateral and vertical pressure of 8 MPa was applied. The first stage: the whole section excavation fast without support. We used excavation equipment at the beginning of 750 mm without support.

The second stage: step-by-step excavation without support. We excavated the roadway step-by-step during the next 850 mm without support. During the excavation, 300 mm for the first

time, part of block crumbs fell off from the upper of model. After the second excavation of 300 mm, due to the fact that the model surface was not subject to constraints, more block crumbs fell off, and the rock mass of the upper model heaved significantly, as shown in Figure 4. Fracture had a tendency to continue to extend. The third excavation of 100 mm, the floor heave of roadway was obvious, as shown in Figure 5. In the fourth excavation, 150 mm into the excavation process, we encountered fault: the surrounding rock was broken; the load was bigger also; the roof of roadway failed seriously, accompanied by slack off, as shown in Figure 6. At that moment, the roadway needed support.

The third stage: step by step excavation with support. We excavated the roadway step-by-step during the next 350 mm, but we had to support the roadway, using a U (arch) steel strip for temporary support, as shown in Figure 7. The fifth excavation, 350 mm, with the oil cylinder to fully unload, the model was in a state of static load. We then added the lateral pressure and the roof pressure to 16 MPa step-by-step, and large pieces of rock mass caving appeared in the loading process. When unloading finally completed, the unloading process was also accompanied by large pieces of rock mass caving, as shown in Figure 8.

Analysis of Characteristics of Roadway Deformation

In this three-dimensional model experiment, we used the roadway deformation monitoring system to monitor the deformation of the roadway. Due to the limited length of this article, we only analyzed the acoustic emission characteristics of surrounding rock in special circumstances.

(1) The lateral pressure and the top pressure was all 12 MPa.

The relationship between AE events and arrival time, as well as energy rate and arrival time in the model rock mass when the lateral pressure and top pressure was 12 MPa, is shown in figure 9a and (b). The roadway excavated to 1500 mm immediately into the fault zone. Before it had been repeatedly loaded and unloaded, the initial crack within the rock mass had already completed the process of closing and opening. The collected signals by the acoustic emission monitoring system were new fracture changes within the modeled rock mass. It can be seen from the Figure 9a and Figure 9b, during this period, the number of AE events was high, but they were mainly small events. Amplitude was about 45 dB, and less than 47 dB events accounted for 70% of the total. When loading to 122.76 s, the maximum amplitude reached 57.8

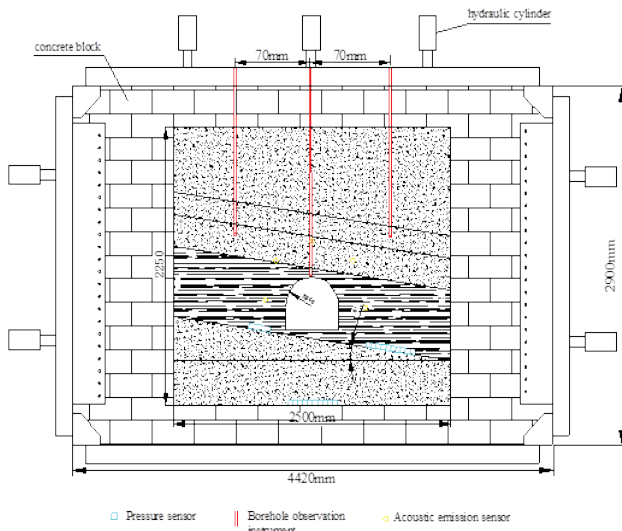


Figure 2. Design scheme.

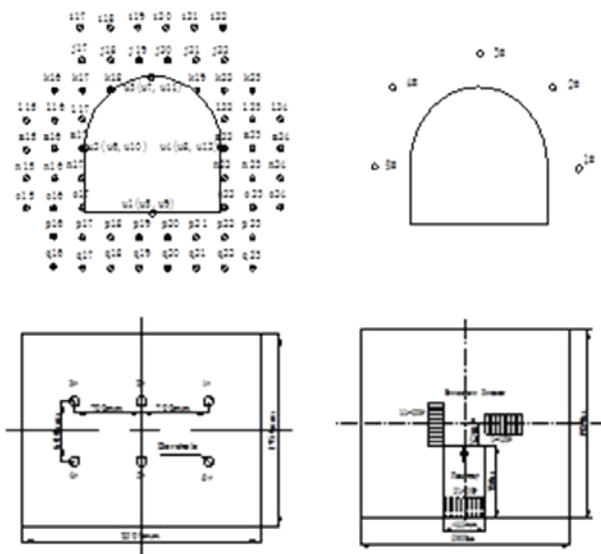


Figure 3. The layout of the monitoring system.

dB. The largest ringing number was 672 times per second; the maximum energy rate was $87mv \times \mu s/s$, and the maximum duration was $157895.4 \mu s$. Thus, it can be seen that it was a main small range fracture of a high number, small energy, and a long duration in the damage of the model material in the roadway excavation before encountering the fault zone. Figure 9c shows the curves of the relationship between total events and arrival time. Also, the total energy and arrival time shows that their slopes had no mutations and that the number of AE events increased continuously and steadily in this loading, and the range of fissure and fracture of the model rock mass expanded steadily.

(2) After unloading several hours, the lateral pressure was 2 MPa, and the top pressure was 0 MPa.

After completing excavation and unloading several hours, the lateral pressure was small (2 MPa), and the top pressure was 0

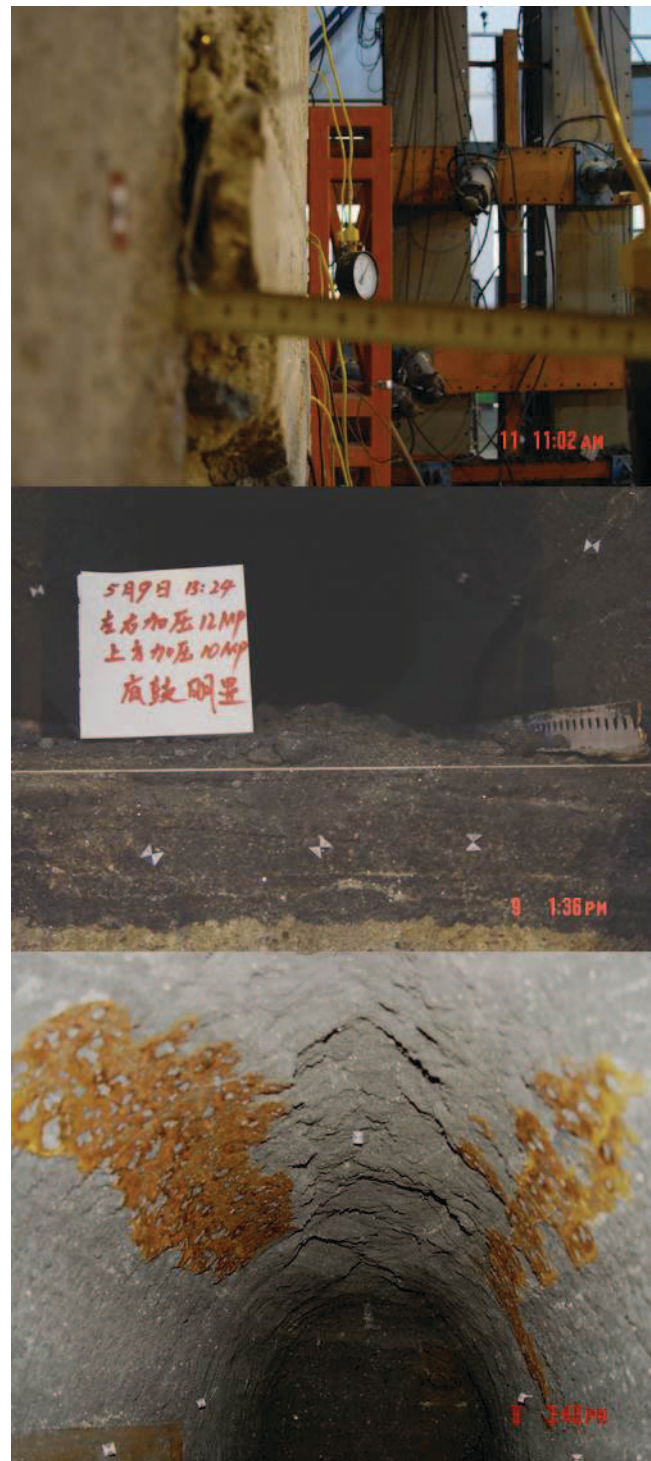


Figure 4. The rock mass of the upper model heaved.

MPa. In Figure 10, the first half of the total events curve was close to level. The slope suddenly increased after the 2569 s. It showed that AE events were scarce at the beginning of the loading of the first half, about within 2500 s. After loading to 2569 s, AE events increased rapidly. After unloading in a period of time, rock mass would be in a relatively stable period. In this period, the extension of fissure and failure within the rock mass was less. However, if an external disturbance (such as engineering blasting, earthquake, etc.)



Figure 5. Floor heave of roadway.



Figure 6. The roof of roadway damaged seriously.

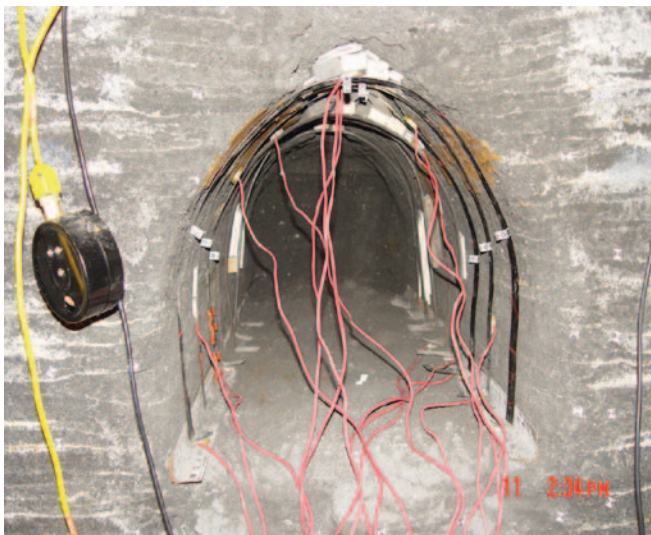


Figure 7. The support of roadway.

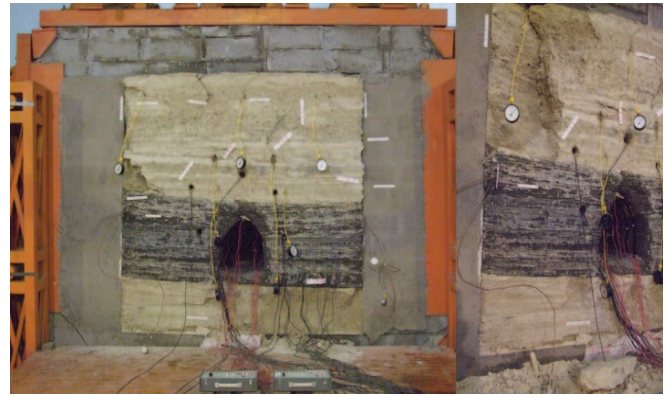


Figure 8. Large pieces of rock mass caving.

were introduced, it could lead to a wide range of surrounding rock failure, triggering disasters, such as rock instability, rockburst, and so on. At the same time, it also inspired us to choose a rational support time. We could take the time to carry out primary support on the roadway in the relatively stable period after completion of the excavation, thereby reducing the probability of accidents.

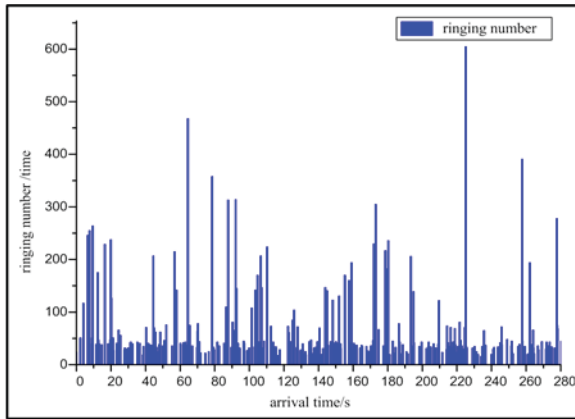
(3) The lateral pressure was 10MPa and the top pressure was 16MPa.

The monitoring was conducted before the “wenchuan earthquake.” In Figure 11a, the number of AE events was relatively well-distributed in the first 1700 s; the duration has been in the oscillation. The number of AE events increased after 1700 s, and the duration lasted longer. At 14:28, the Wenchuan earthquake broke out, and the distance monitoring was 1321s, duration was about 300 s, and the monitoring time was 1621s. After the earthquake, the duration of the AE events increased significantly. It showed that a large number of cracks within the model rock mass expanded continuously under the load of earthquake. The range of its internal failure of rock mass were also expanding. Figure 11b is the curve of AE events per minute and duration. During the 20 minutes before the earthquake, AE activities oscillated continuously within a certain range. After a period of silence after the earthquake (about 350 s), large scale and long duration events began to appear. This showed that, when the AE signal undergoes oscillatory and quietness, a large number of long time and persistent failure may appear in the rock mass.

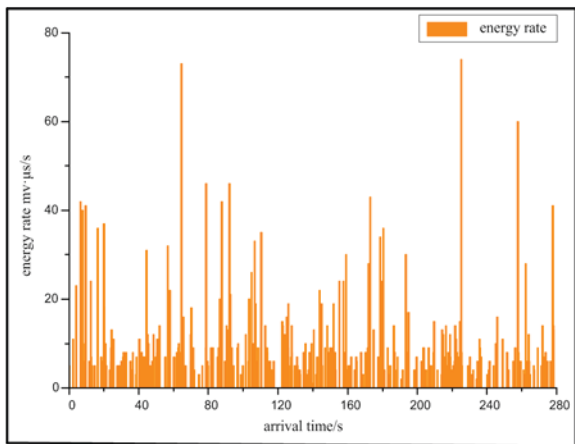
CONCLUSION

(1) The establishment of the deformation monitoring system of a roadway based on the three-dimensional model experiment builds a scientific platform for prediction technology of the deformation and failure of complex rock masses and provides the reference for improving the design and testing of the model experiment.

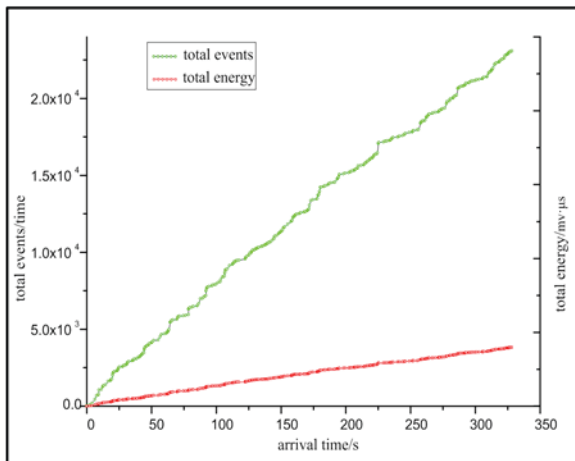
(2) It can be seen from the AE data in this three-dimensional model experiment, when rock masses undergo a period of “oscillation and silence,” it indicates that it will appear a large number of times and show persistent failure. It has great significance to provide warnings and forecasts for safety hazards of the surrounding rock in coal mines.



(a) The relationship between ringing number and arrival time



(b) The relationship between duty and arrival time.



(c) The relationship between total events/ total energy and arrival time

Figure 9. Character of acoustic emission.

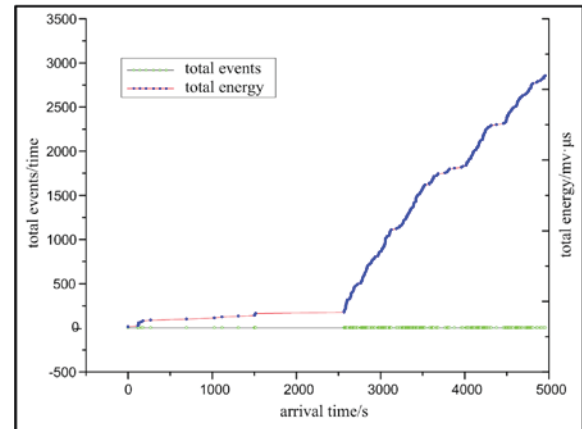


Figure 10. The relationship between total events/ total energy and arrival time.

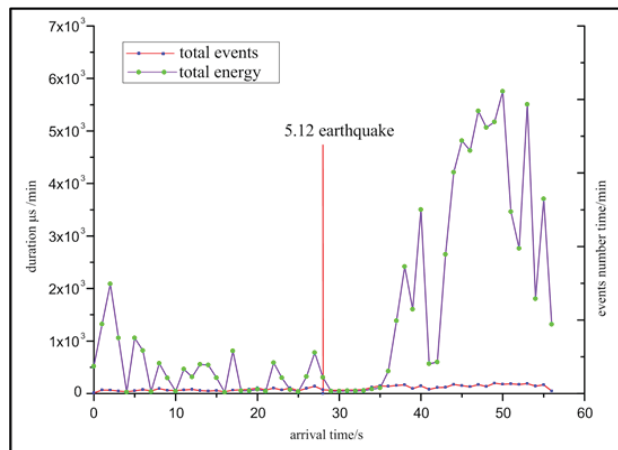
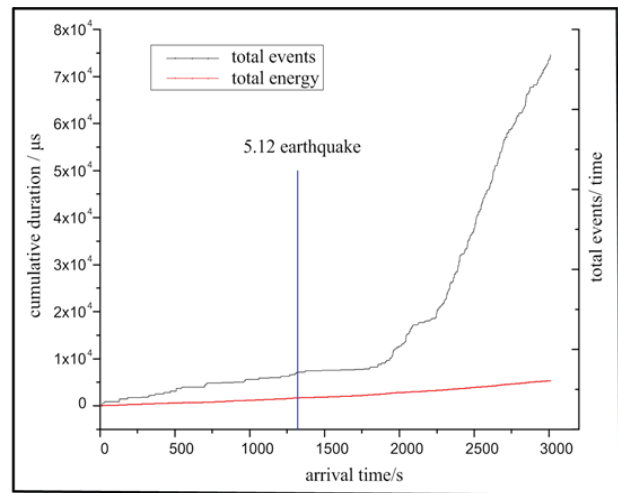


Figure 11. Character of acoustic emission.

ACKNOWLEDGMENTS

The author would like to gratefully acknowledge the support for this project from the National

Natural Science Foundation of PRC (51074120, 51204132) and Scientific Research Program Funded by Shaanxi Provincial Education Department (11JK0785).

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Numerical Modeling of Stress Distribution of Gateroads Subjected to Dynamic Pressure of Longwall Mining in the Immediate Overlying Seam, a Case Study

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ABSTRACT

The entries that are subjected to dynamic pressure are much more difficult to maintain due to repeated disturbance. A case study of an entry stress distribution was performed using the FLAC^{3D} program. By analyzing the distribution of stress field around the entries, this paper provides guidelines for entry support design in the lower (#16) seam subjected to the repeated disturbance from longwall mining in the immediate overlying (#17) seam, 72ft (22 m) above. The results showed that there was stress superposition inside the barrier pillar in the lower seam, 15ft (5m) to 45ft (15m) deep, during development. The influence zone of the front abutment pressure in the upper (#16) seam was about 100ft (30m) outby the faceline. The stress-relieved zone was from 0 to 100ft (30m) inby the face. The area further inby the face (i.e., 230 ft (70 m) inby the face) was within the gob compaction zone. Along the face advancing direction, the affected zones of the entries in the lower (#17) seam that were subjected to the dynamic pressure from longwall mining of the immediate overlying seam could be divided into four stages: pillar development stress stage, front abutment pressure stage, unstable gob stage, and stable gob stage. Those entries within an area from 100ft (30m) outby to 100ft (30m) to 230ft (70m) inby the faceline should be reinforced with special supports. The numerical modelling matched the monitoring results well.

Keywords: numerical modeling; immediate overlying seam; dynamic pressure; stress distribution

INTRODUCTION

China is the world's largest coal producer and consumer. In order to satisfy the national demand, all coal mines must be highly productive. One of the requirements for a productive mine is rapid development of gateroads. However, most coal reserves in China have multiple-seams, and gateroad development may be badly delayed due to seam interaction. Consequently, gateroad development and support must consider the effects of under- or over-mining so that gateroad supports are properly designed. Otherwise, gateroads become unstable and require remedial measures that result in very slow gateroad development.

The gateroad developed under longwall mining in the immediate overlying seam is called a dynamic pressure gateroad. As the longwall becomes more productive, gateroad development very often cannot keep up with longwall mining. *In fact, there are many cases where the next panel was not ready when the current panel was completed, and impromptu measures had to be adopted. With a quoted revenue loss of \$500 to \$2,000 per minute of longwall idle time, inadequate panel development simply cannot be tolerated* (Peng, 2006). Sometimes, the gateroads have to be developed under a longwall mining face in order to mitigate mismatch between development and retreat mining. The gateroads developed under an active longwall mining panel are much more difficult to maintain due to the continuing disturbance of abutment pressures. Although many investigators have performed research on the deformations of roadways or the movement of the overburden strata subject to longwall mining (Matsui, Ichinose, and Uchino, 1987; Shen, Poulsen, and Qu, 2011; Summers and Jeffery, 1992; Donnelly, Culshaw, and Bell, 1989; Shen, King, and Guo, 2008; Lu, Liu, and Xu, 2007; Kim, Parizek, and Elsworth, 1997; Holla, 1997; Holla and Buizen, 1991; Chen, Mishra, and Beck, 2002; Huang, 2007; Unal, Ozkan, and Cakmakci, 2002; Trueman, Lyman, and Cocker, 2009; Juarez-Ferreras et al., 2008; Langosch and Ruppel, 2008; Singhand Singh, 2009; Torano et al., 2002), few of them conducted research on the entries developed under longwall mining in the immediate overlying seam.

The condition of rocks surrounding the entry and the state of stress of dynamic pressure of the entry in the closely spaced multiple-seam mining are the most complex among all the roadways. *Under such complex loading conditions, it is a challenge to design and utilize a proper support system, and yet ensure the safety and stability of these openings within economical limits* (Unal, Ozkan, and Cakmakci, 2002). In addition, because of the abutment pressures in the immediate overlying seam, roadway deformation in the underlying seam will be very serious after longwall mining in the overlying multiple seams (Xie, Jing, and Guo, 2012). Although numerous measures have been proposed and implemented, and improvements have been observed (Wu et al., 2011; Fang, He, and He, 2009; Tian and Gao, 2010; Xu, Zhang, and Cao, 2009; Yang, 2012; Han, Chen, and Li, 1998), the development of a dynamic pressure entry and its ground control is very difficult, and its deformation control problems still need to be resolved.

An example of gateroad lying under an active longwall mining panel 72 ft above is presented in this paper. Seam interaction was not considered in the gateroad support at first. Consequently, the gateroads were badly disturbed as the overlying longwall face was advancing. The unstable gateroads required supplemental measures, the implementation of which greatly slowed down the gateroad development.

In this research, the FLAC^{3D} numerical simulation program was used successfully to determine the influence of the immediate overlying longwall mining on the gateroads in the lower seam. The state of stress of the dynamic pressure gateroad was analyzed to provide guidance for properly designing the gateroad support subject to seam interaction in closely spaced multiple-seam mining.

MINE DESCRIPTION

The coal mine of interest is located in north eastern China. There are two minable seams: #16 (upper) and #17 (lower) that dip about 19°. The tailgate entry of the lower seam is 2,300ft (700m) deep. The #16 seam was 72ft (22m) above the #17 seam. When the longwall was retreating in the #16 seam, a headgate was being developed in the #17 seam, while the development of a tailgate had been completed. The two gateroads which underneath the gob of the #16 seam longwall on the #17 seam were 66ft (20m) apart and were the same size: 13ft (4m) wide by 15ft (4.7m) high, supported with roof bolts and steel sets. Gateroad deformation in the #17 seam was significant during the longwall mining because of the small interburden between #16 and #17 seams. Bolts failure and roof falls happened heavily. Under this condition, severe ground control problems occurred in the multi-seam interaction gateroads (i.e., the headgate and tailgate in the #17 seam).

The #17 seam, which is 12.1ft (3.7 m) to 12.8 ft (3.9 m) thick, consisted mainly of black clarain, interbedded with durain banding. The bedding planes are not well defined but joints are. The immediate roof consisted of carbonaceous mudstone, and the main roof consisted of grey fine sandstone with well-cemented large particles at the lower part. The floor consisted of dense and hard black siltstone, with fine sandstone bandings at the center of it.

The #16 seam, located 72ft (22m) above the #17 seam, is called the immediate overlying seam (IOS) in this research. It is 5ft (1.5m) thick with a dip angle of 19°. A fully mechanized longwall mining method with a panel width of 560ft (170m) was employed. Fig. 1 shows the relative position of roadways in the #17 seam and the longwall face in the #16 seam.

NUMERICAL MODELING

In order to analyze the stress distribution of the multi-seam interaction entry, the following two stages of simulations were performed:

- The two roadways in the #17 (lower) seam, 66ft (20m) apart, were developed before longwall mining began in the #16 (upper) seam.
- The effects of longwall mining in the #16 seam on the gateroads in the #17 seam were modeled.

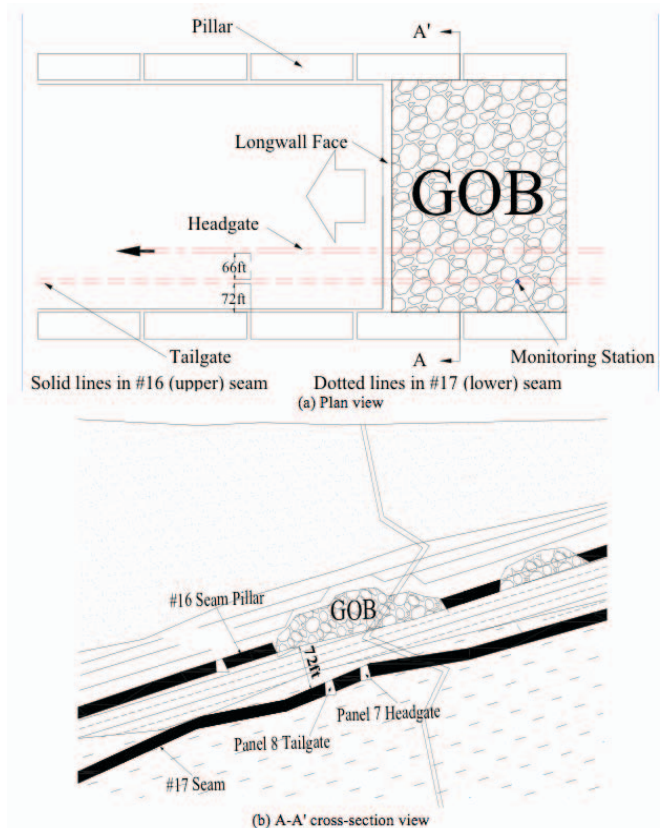


Figure 1. The relative position of roadways in the #17 seam and the longwall face in the #16 seam (not in scale).

Numerical model and simulation parameters

Based on the stratigraphic sequence of the colliery, a simplified Mohr-Coulomb model was constructed as shown in Figure 2 using FLAC^{3D} numerical simulation software. The model was 920ft (280m) in the X direction, 525ft (160m) in the Y direction, and 620ft (190m) in the Z direction. It consisted of 148,165 elements and 96,221 grid-points. The longwall face is 560ft (170m) wide and it is advanced a 390ft (120m) in the Y direction. It was excavated in six steps each of 65ft (20m) of face advance, leaving a 160ft (50m) wide protective coal pillar at both sides.

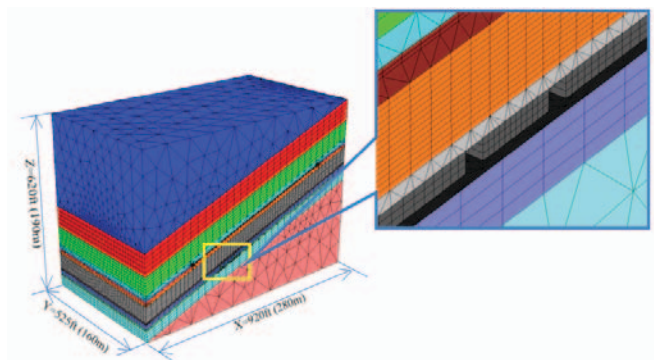


Figure 2. The 3D numerical model.

The overburden of panel 7 in #17 seam is 2,300ft (700m) thick. The distance from the top of the model to the surface was 1,800ft

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(560m). The horizontal stress was 1.2 times the vertical stress in the model. The bottom of the model was a fixed boundary, and the four vertical boundaries of the model were frictionless surfaces, i.e., they were roller-supported.

Table 1 shows the model stratigraphic sequence and the model parameters. Table 2 shows the sequence and their rock properties according to empirical values (obtained by the laboratory testing results).

Table 1. Simulated model stratigraphic sequence.

No.	Strata	Thickness of layer, ft (m)	Cumulative thickness, ft (m)
1	Sandstone	70.5(21.5)	278.9 (85)
2	Siltstone	59.1(18)	208.3 (63.5)
3	Sandy Mudstone	16.4(5)	149.3 (45.5)
4	#16 Seam	4.9(1.5)	132.9 (40.5)
5	Siltstone	13.7(4.17)	128.0 (39)
6	Fine Sandstone	49.2(15)	114.3 (34.83)
7	Carbonaceous Mudstone	9.3(2.83)	65.1 (19.83)
8	#17 Seam	12.5 (3.8)	55.8 (17)
9	Siltstone	7.2(2.2)	43.3 (13.2)
10	Medium Sandstone	36.1 (11)	36.1 (11)

Table 2. Rock mechanics properties used in the model.

No.	Strata	Density γ , kg·m ⁻³ (pound - ft ³)	Shear Modulus G ,MPa(Psi)	Bulk Modulus K, MPa (Psi)	Cohesion C,MPa (Psi)	Friction angle ϕ (°)
1	Sandstone	2,475 (154.5)	2,000 (290,075)	5,800 (841,219)	3.60 (522)	35
2	Siltstone	2,440 (152.3)	2,000 (290,075)	4,800 (696,181)	2.60 (377)	32
3	Sandy Mudstone	2,360 (147.3)	1,600 (232,060)	2,400 (348,091)	1.20 (174)	28
4	#16 Seam	1,400 (87.4)	960 (139,236)	2,200 (319,083)	0.80 (116)	25
5	Siltstone	2,300 (143.6)	2,100 (304,579)	2,600 (377,098)	2.20 (319)	32
6	Fine Sandstone	2,340 (146.1)	1,600 (232,060)	2,400 (348,091)	2.00 (290)	32
7	Carbonaceous Mudstone	2,290 (143.0)	2,100 (304,579)	2,600 (377,098)	1.00 (145)	28
8	#17Coal Seam	1,400 (87.4)	960 (139,236)	2,200 (319,083)	0.80 (116)	25
9	Siltstone	2,300 (143.6)	2,100 (304,579)	2,600 (377,098)	2.20 (319)	32
10	Medium Sandstone	2,600 (162.3)	2,600 (377,098)	4,800 (696,181)	3.00 (435)	35

CHARACTERISTICS OF THE STRESS DISTRIBUTION OF SURROUNDING ROCKS

Figure3 shows the vertical stress distribution contours in the surrounding rocks along a cross-section at Y=260 ft (80m) (at the middle of the mining direction) after the development of the entries. When the two roadways were developed, the induced

stresses that formed around them superimposed in the barrier pillar. The location of the peak pressure was located 15-30ft (5-10m) from the roadway ribs. The area of stress superposition in the pillar was 15-45ft (5-15m) inside the pillar. The magnitude of stress on the ribs of each roadway is smaller than that in the pillar area. The side induced pressures began to decrease from 30ft (10m) inside the panel and returned to the overburden pressure some distance away.

ABUTMENT PRESSURE DISTRIBUTION OF LONGWALL FACE

Stress redistribution in the surrounding rocks of the gob due to longwall mining not only causes stress concentration in the pillars but also induces stresses deep into the floor strata. Consequently, it is very important to know the state of stress in the floor strata or roadways that are affected by longwall mining so that the roadway can be properly designed, including location, dimensions, and supporting methods.

Figure 4 shows the vertical stress distribution in the floor of the longwall panel at #16 seam. Figure 5 shows the vertical stress distribution on the cross-section along the center axis of the tailgate. Figure 6(a-d) shows the vertical stress distribution on the cross-sections perpendicular to the mining direction at different distances from the longwall face.

Figure 6 shows four stress distributions on various cross-sections perpendicular to the mining direction. Figure 6a and Figure 6b show the front abutment pressure while Figure 6c and Figure 6d show the gob pressure. There is a larger blue area in Figure 6a than in Figure 6b. It shows that the front abutment pressure reaches the maximum value in the area 30ft (10m) outby the face

line. In the area 100ft (30m) behind the face, the gob pressure is not so high(orange area in Figure 6c).When it reaches 230ft (70m) behind the longwall face, the gob pressure begins to increase gradually(light green area under the gob in Figure 6d) because the gob was compacted.

Figure 4 and Figure 5 show that the influence area of the front abutment pressure ranges from 0 to 100ft (30m) outby the face (the

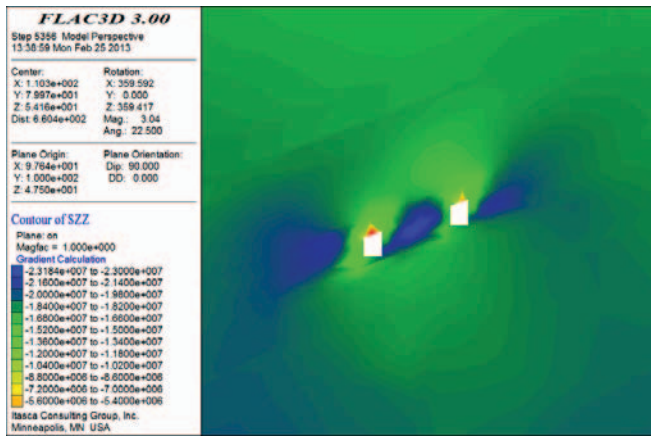


Figure 3. Stress distribution on the cross-section at Y=260 ft.

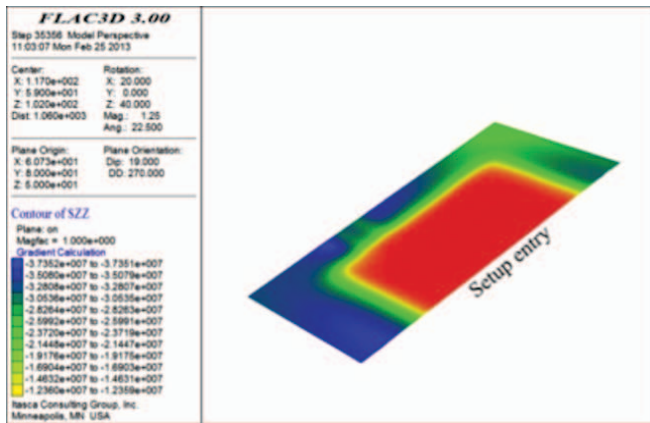


Figure 4. Stress distribution in the panel floor.

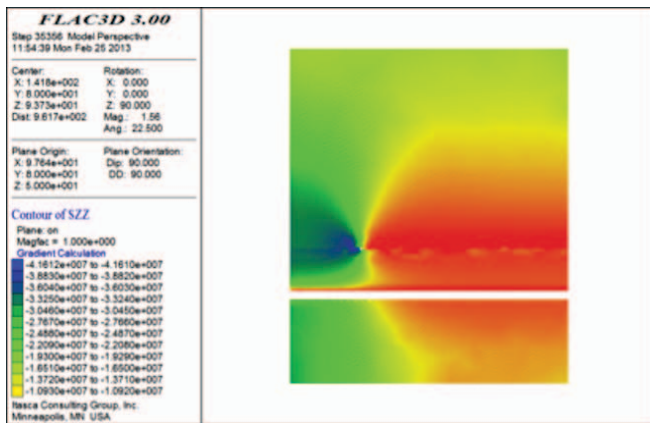


Figure 5. Stress distribution on the cross-section along the center axis of the tailgate.

range of dark blue area in the figure). It reaches the maximum value (i.e., maximum front pressure) when the face is 25–35ft (8–11m) inby (shuttle shape (the shape of the blue area in the figure) area in Figure 6a). The roadways are most affected between 25ft and 35ft (8 m and 11m) outby the face. The rocks surrounding the two roadways are not affected when the distance is more than 100ft (30m) outby the face (Figure 6b). The overburden pressure behind the face was not totally transferred to the floor within the range of

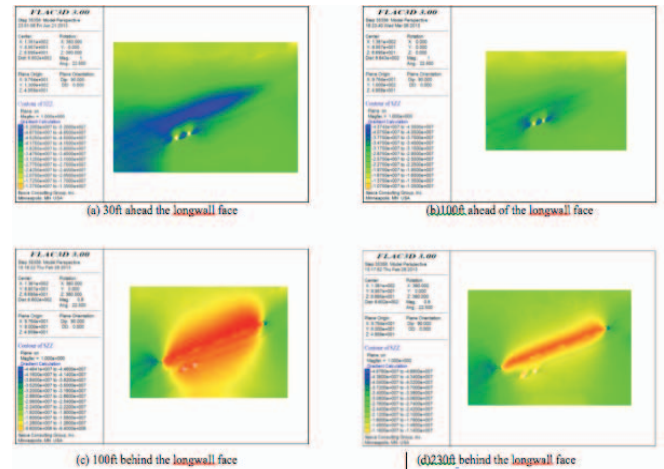


Figure 6. Stress distributions on various cross-sections perpendicular to the mining direction.

0–100ft (Figure 6c) behind the face. This range is called the stress-relieved zone. When it reaches 230ft (70m) behind the longwall face, the overburden pressure will be fully transferred to the floor (Figure 6d). When it is between 100ft and 230ft (30 m to 70m) behind the face, the two entries are affected by the multi-seam interaction pressure once again due to the compacting process of the gob. Thus, the two roadways should be reinforced with special supports and monitored until the face is 100–230ft (30–70m) outby.

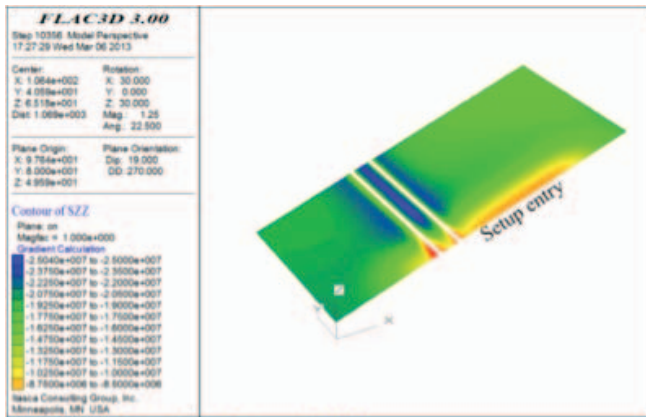
CHARACTERISTICS OF STRESS DISTRIBUTION OF ROCK STRATA SURROUNDING THE ENTRIES

Figure 7 shows the stress distributions in the coal seam on the cross-sections along the faceline direction when the face has advanced 65ft (20m) (Figure 7a), and 260ft (80m) (Figure 7b) from the set-up entry.

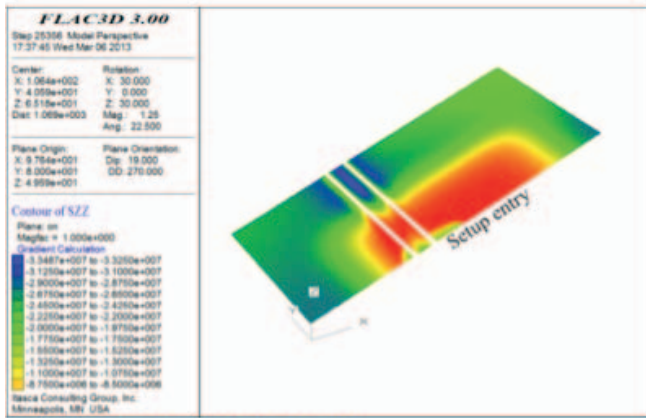
An analysis of Figure 7 in terms of the spatial relationship between the gateroads in #17 seam and the longwall face in #16 seam led to the following conclusions:

- When longwall mining begins at the set-up entry, parts of the roadways are located in the stress-relieved zone below the gob. Their pressures are gradually restored to the overburden pressure as the longwall face advances. In Figure 7a, where the face has advanced 65ft (20m), orange area of the pillar under the gob is located in the stress-relieved zone where the stress is low. The gob pressure began to increase when the face is 100ft (30m) outby (light green area inside the pillar near the setup entry in Figure 7b).
- As the face advances, the pressure in the pillar (between tailgate and headgate) is further increased, again 30ft (10m) ahead of the longwall face (dark blue area inside the pillar in Figure 7a) due to the arrival of the front abutment pressure.
- According to Figure 7b, each point in the pillar experiences a complete cycle of overburden pressure → stress superposition → stress relief → stress superposition → overburden pressure.

Consequently, implementation of secondary supports for the roadways is the most important task during longwall mining in the immediate overlying seam due to the cycle impact of the pressure.



(a) The face has advanced 65ft



(b) The face has advanced 260ft

Figure 7. Stress distributions in the floor on various cross-sections along the faceline direction when the face has advanced 65 ft and 260 ft from the set-up entry.

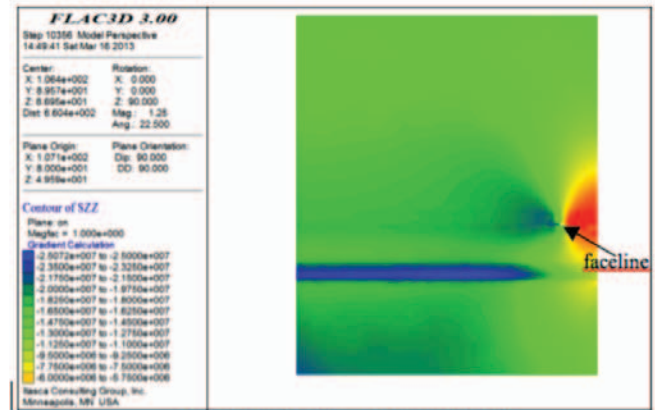
STRESS DISTRIBUTION IN THE PILLAR DURING LONGWALL MINING

Figure 8 shows the vertical stress distributions on the cross-sections along the center line of the pillar when the face has advanced 65 ft (20m) (Figure 8a) and 260 ft (80m) (Figure 8b) from the set-up entry.

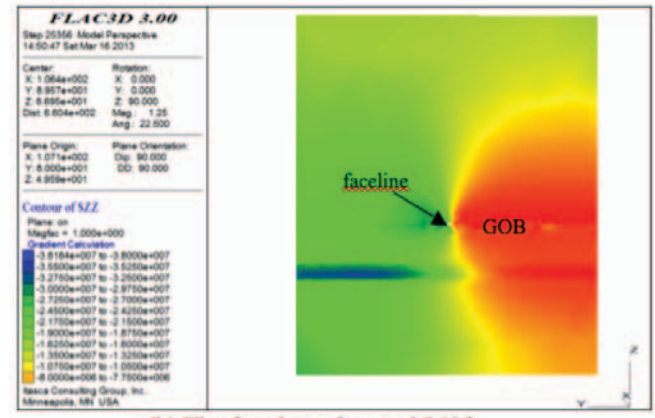
In order to analyze the effects of different stages of longwall mining in the #16 seam on the roadways in the #17 seam, the relative position between the face location in the #16 seam and the two roadways in the #17 seam are divided into four stages (Figure 8):

Stage I—the pillar development stage: This range is more than 100ft (30m) outby the face. At this stage, the longwall face was farther inby, so the roadways in the #17 seam were not affected by the front abutment pressure of the longwall face in the #16 seam. However, they were affected by the abutment pressures created due to the development of the two gateroads on both sides.

Stage II—the front abutment pressure stage: This range is between 0–100ft (30m) outby the face, Figure 8a. At this stage, the



(a) The face has advanced 65ft



(b) The face has advanced 260ft

Figure 8. Vertical stress distributions on the cross-sections along the center line of the pillars when the face has advanced 65 ft, 130 ft, 195 ft, and 260 ft from the set-up entry.

longwall face was close by, so the roadways in the #17 seam were affected not only by the superimposed abutment pressures due to gateroad development, but also by the front abutment pressure of longwall mining in the #16 seam. The influence area was within 100ft (30m) outby the face, as shown in Figure 8a. Because the stress concentration at this stage greatly affected the roadways in the #17 seam, supplemental supports should be installed at this stage.

Stage III—the unstable gob pressure stage: Unstable means that the pressure changes with the time in this stage. This range is between 100–230ft (30–70m) inby the face. The pressure below the gob changed continuously after the passage of the longwall face. As shown in Figure 8b, the red zones are under the unstable gob pressure stage. At the end of this stage, the stress was transferred down to the pillar and increased its stress concentration. Intensified monitoring and additional supports are required for the roadways in the #17 seam.

Stage IV—the stable gob pressure stage: Stable means that the pressure does not change obviously with the time in this stage. This range is more than 230ft (70m) outby the face. The stress acting on the roadways was stable at this stage. Therefore, the supports required would be fewer than in Stages II and III.

The four different stages stated above provide a guideline for implementing proper ground control measures at different sections of roadways subject to the dynamic pressures of longwall mining in the immediate overlying seam. In particular, Stages II and III require additional supports.

FIELD MONITORING AND ANALYSIS OF RESULTS

Layout of monitoring station

A set of monitoring devices (e.g., roof separation indicators and displacement meters) were installed to obtain the roof separations and convergence in the tailgate before the longwall mining in #16 seam. The displacement meters were setup horizontally and vertically so as to obtain the roof-to-floor convergence and rib-to-rib displacement. The roof separation indicator was a multipoint displacement meter, which was setup in the roof to obtain the displacement between the first (carbonaceous mudstone) and second (fine sandstone) strata. See Figure 1 for the location of the monitoring station (135ft (40m) from the set up room).

Analysis of results

The monitored data were recorded every 15ft (5m) of the face advance. Figure 9 shows the monitoring results when the face had advanced 260ft (80m) from the set up room. Figure 9a shows rib-to-rib displacement (red line) and roof-to-floor convergence (black line) where the 0 point on the X axis is the monitoring station and negative numbers mean the face is inby, while positive numbers mean the face is outby. Figure 9b shows the separation between the two strata (carbonaceous mudstone and fine sandstone) in the tailgate roof.

At the beginning, the monitoring station was 135ft (40m) outby the face. At this time, the rate of deformation of the gateroad was slow (see Figure 9a). When the face had advanced to point “a” on Figure 9a, the monitoring station was about 100ft (30m) outby the face line, and the rate of deformation of the gateroad began to increase steadily until the face reached point “b.” At this time the monitoring station was 80ft (25m) inby the face line (i.e., under the gob). As the face continued to advance, the monitoring station moved further and further inby the face. After point “b,” the rate of deformation decreased slightly up until point “c.” At this time, the gob was not totally compressed, so the stress that transferred to the lower seam was not high. When the face had advanced to point “c,” where the face was 190ft (60m) outby the monitoring station, the rate of deformation increased again. This is because the gob was compressed from this point on to point “d,” where the face was 230ft (70m) outby the monitoring station. From point “b” to point “d,” the stress that transferred to the lower seam was not steady. That’s why this stage is called the “unstable gob pressure stage” as stage III stated previous. After point “d,” the rate of deformation gradually becomes steady.

Similarly, there are four stages in the roof separation curve (see Figure 9b). From point “A,” the separation velocity began to increase until point “B.” From point “B” to point “C,” the gob was not totally compressed. That’s why the separation velocity decreased slightly. After point “C,” the gob was compressed and the separation velocity increased accordingly. The stage from point “B” to point “D” was the unstable gob pressure stage. After point “D,” the roof separation becomes steady. Between the first

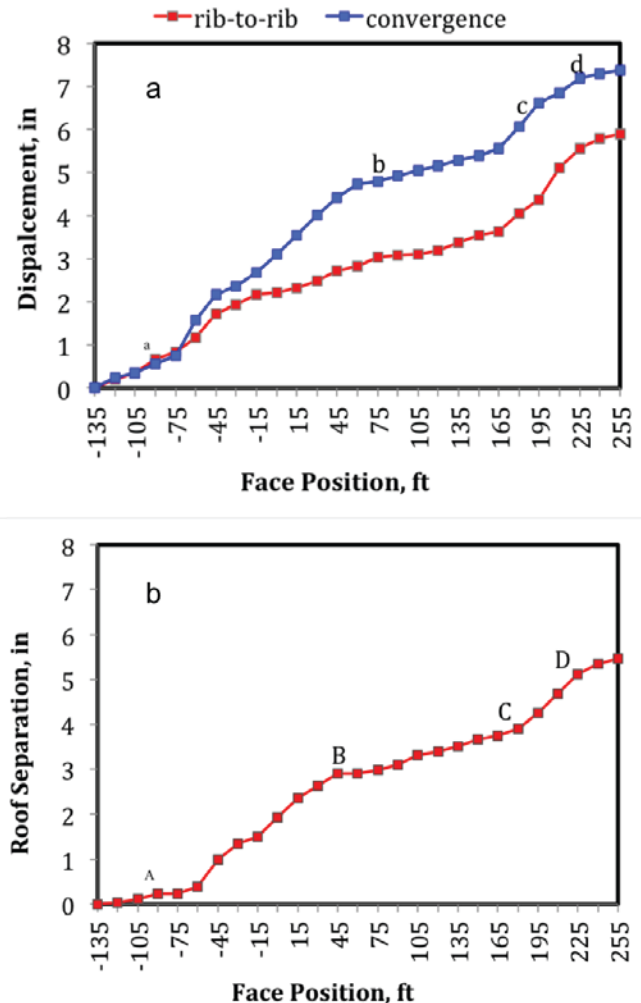


Figure 9. The results of instrumentation.

(carbonaceous mudstone) and second (fine sandstone) strata, there is a 6-inch roof separation. That’s because carbonaceous mudstone is soft and easily fractured when it is affected by the longwall mining in the #16seam.

According to the monitoring results, entry deformation was related to the distance between the face and the monitoring station. When the face was 80ft (25m) to 230ft (70m) inby the monitoring station, the entry was affected by the dynamic pressure due to roof caving in the gob. When the working face was 0ft to 100ft (30m) inby the monitoring station, entry deformation was significant due to the front abutment pressure. Referring to the former statement, the numerical modeling matches the monitoring results well.

Figure 10 shows the conditions of tailgate from outby to inby the face. Figure 10a was taken while the face was about 100ft (30m) inby the monitoring station, and Figure 10b was taken while the face was about 200ft (60m) outby the monitoring station. It can be clearly seen that the tailgate was subjected to heavy pressure and secondary supports are required.



(a) Photo taken 100ft outby the face



(b) Photo taken 200ft inby the face

Figure 10. Tailgate damage conditions.

CONCLUSIONS

In this paper, FLAC^{3D} was used to model the stress distribution of roadways subjected to multi-seam interaction from longwall mining in the immediate overlying seam in order to determine reasonable supporting measures needed at different stages of mining. The following conclusions were made:

At both sides of the gob, the influence area of the side abutment pressure is 0~100ft (30m) from the rib of the gateroad. The peak value of the front and side abutment pressures occurred 1530ft (510m) inside the longwall face and the rib.

The area from 0 to 100ft (30m) inby the longwall face is the stress-relieved zone. The gateroads are affected again by the dynamic pressure from 100 to 230ft (30 to 70m) behind the faceline in the gob. This area should be reinforced when it is located between 100 to 230ft (30 to 70m) behind the face. The rock strata surrounding the gateroads will be subjected to the overburden pressure when the gob is compacted 230ft (70m) behind the longwall face.

Along the face advancing direction, the influence zones of the gateroads in the lower seam that are subjected to the dynamic pressure from longwall mining in the immediate overlying seam can be divided into four stages: pillar stress, front abutment pressure, unstable gob, and stable gob.

The key stages of the gateroads affected by the dynamic pressure are located between 0–100ft (30m) outby the face and 100–230ft (30–70m) inby the face. Intensified monitoring and installation of secondary supports must be performed during this period.

ACKNOWLEDGEMENTS

This study is supported by the National Nature Science Foundation of China (No.51174192). The authors express great gratitude to the China Scholarship Council (CSC) for granting the scholarship (No. 2011642029) to Ximeng Li during his PhD joint training at the Department of Mining Engineering at West Virginia University (USA).

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Roadway Supporting Technology Applied to Very Soft Coal Seam at Yanlong Mining Area in China

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ABSTRACT

The compressive strength of coal in Yanlong mine area of China is less than 3 MPa. Basically, it is powdery. The roof and floor rocks are mudstone. Therefore this coal seam is a typical “three soft” coal seam. Roof bolting cannot be used due to the very soft strata. When the conventional U-steel support was used, roadways deformation exceeded 30% of its original dimensions in some sections one half month after development. The required repair work was huge, seriously affecting the daily mine production. This paper describes how to support the soft coal roadways. The deformation characteristics of soft coal roadway were investigated. The conventional U-steel support is not subjected to uniform load, so the whole supporting structure must be strengthened. In addition, drilling holes in the coal seam can transfer high stress from near the ribs to the deeper parts of the surrounding strata, which is beneficial to the stability of the roadway. This new support method is to combine the strengthened U-steel support with pressure-relieving drill holes, providing an economic and efficient way to support the very soft coal roadways.

INTRODUCTION

Dragon mining area is located in the geologically complicated tectonic zone of Songshan Mountain, having 72 mountain ridges. Therefore, coal seams in this area are extremely non-uniform and very weak with well-developed fractures, which is the typical characteristic of “tectonic coal”. Figure 1 shows the topographic map of Yonghua No.1 mine in Dragon mining area. The major coal seam mined in the Dragon mining dips $9^{\circ} \sim 20^{\circ}$. It is so soft, less than 3 MPa in compressive strength, that it can be crushed easily by hand. The immediate roofs are carbonaceous mudstone and shale, and the main roof is siltstone. The floor rock is carbonaceous mudstone having abundant plant fossils. Therefore, this coal seam is a typical “three soft” coal seam.

Sun (2003) studied the reliability theory of mine supporting systems and found that roof bolting has only limited applicability. When roof bolting is used in the roadways of very soft coal seam, the bolt holes becomes distorted soon after drilling, such that the resin and bolt are difficult to insert into the bottom of the hole. In addition, because the coal is so loose, there is no cohesiveness between the resin bolt and the hole wall. In the worst case, the bolt



Figure 1. Topographic map of Yonghua No.1 mine in the Dragon mining area, China.

will be pulled out, when the pre-tension is applied. Therefore, the bolting method cannot be used.

Xie (2010) analyzed the structural stability of U-steel support and its control technology. This type of support is used in many highly stressed roadways and known to have the greatest supporting strength. However it is a passive support. When the yielding steel support made of U36 steel was installed in the mine, it did not achieve the ideal supporting performance. Field observations showed that the largest cross-sectional deformation of coal roadway could exceed 30% about one half month after development. The whole support sank into the floor continuously without stabilizing after a long period of time. When the dimension of timber foot block was increased, the deformation of coal roadway did not improve much. On the contrary, the timber legs were bent and the joints of the timber legs were broken. In addition, it greatly increased the repair work because the large foot block was difficult to pull out of the floor. Usually, 70 ~ 90% of the coal roadways require repairs, and most of the roadways need repeated repairs and re-supports, which slowed down roadway development, and increases supporting cost and potential safety hazards.

The main reasons leading to the above-mentioned poor conditions were the extremely low bearing capacity and strong rheological properties of the surrounding rocks in soft coal roadways. Because the strong rheological properties made the roadways deform continuously without stabilizing. As a result, the stresses in the surrounding rocks were easily transferred to the artificial supports. Due to a lack of effective method to improve the coal seam strength, the key to solve this problem is to improve the strength of support materials or transfer the high pressure of surrounding rocks to other locations. Yao (2000) found the capacity of U-steel support could be enhanced. Luo (2009) increased the supporting capacity of U-steel supports through backfilling in the gob. However, his method was not suitable for soft coal seams. Through theoretical and finite element analyses, it was found that the conventional U-steel supports are subjected to uneven loading, and there are local stress concentration problems and that at the same time, drilling to relief pressures can improve the stress distribution around the roadways.

As for various types of steel supports in Chinese coal mines, the U36-steel support is a conventional support having the highest bearing capacity. But it is not strong enough for soft coal roadways. This paper provides guidelines for enhancing the supporting capacity of the whole U36-steel support through structural compensation and optimization. At the same time, a method of drilling holes for pressure-relief is adopted so that the high stresses in the surrounding rocks can be transferred to farther away. The combination of these two methods provides a way to ensure the stability of the soft coal roadways. This technology has been successfully employed in the Dragon mining area.

STRESS ANALYSIS OF STEEL SUPPROT

ANSYS was used to model the conventional U36-steel support in soft coal roadways. The model with boundary condition is shown in Figure 2, which is 75 m wide by 75 m high with the roadway at the center. The vertical displacement on the lower boundary was fixed, while the horizontal and the top boundary were free. The vertical stress was applied on the top boundary, and the horizontal stress was applied on the left-side and right side boundaries. The Mohr-Coulomb yield criterion was used. The model of U-steel support consisted of two nodes beam element, and the overlap parts of the steel support were simulated with equivalent material processing method, i.e., the lap part and clamps of the support were replaced by the equivalent cross section bar. The vertical stress σ_x , horizontal stress σ_y , and lateral pressure coefficient were 8.38 MPa, 12.56 MPa, and 1.5, respectively. Figure 3 shows the bending stress distribution of the U-steel support. The bending stresses of the top and side parts are larger. These parts deform easily due to lack of constraint.

Zhang (2012) investigated the locations of the steel support where its strength must be enhanced. Zhang (2010) proposed the factors for U-steel support design. In order to increase the bearing capacity of the whole U-steel support, horizontal and vertical beams can be used to limit the inward deformation of the U-steel support. The beams were both made of high strength U-shaped steel connected by a specially made weld head. Table 1 shows the four kinds of enhanced U-steel support.

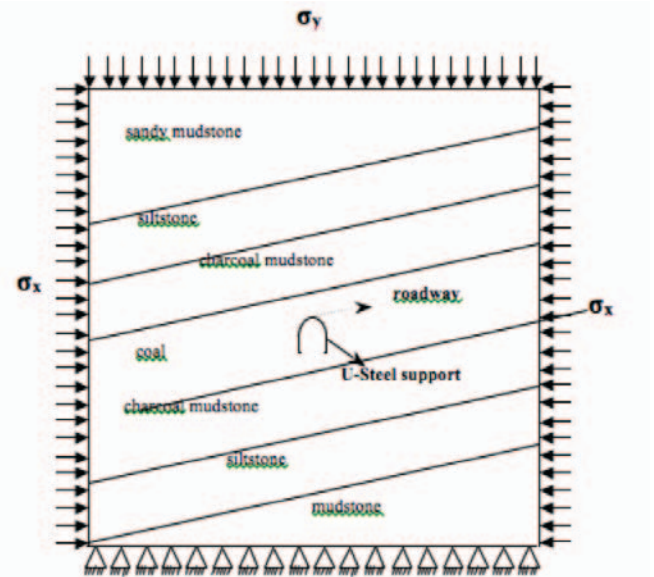


Figure 2. Model with boundary condition.

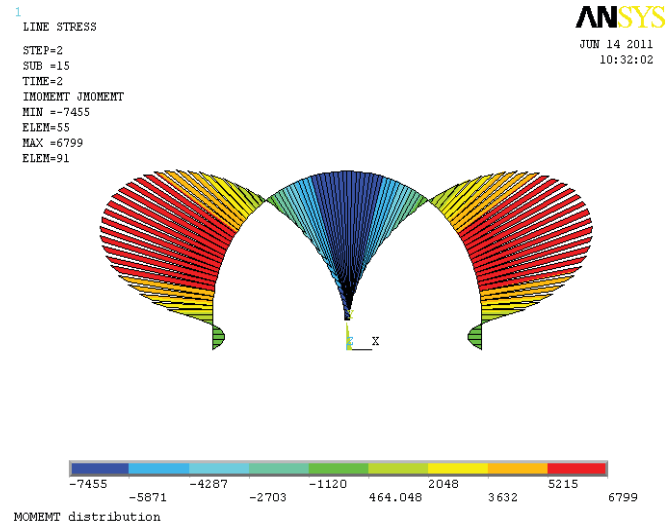


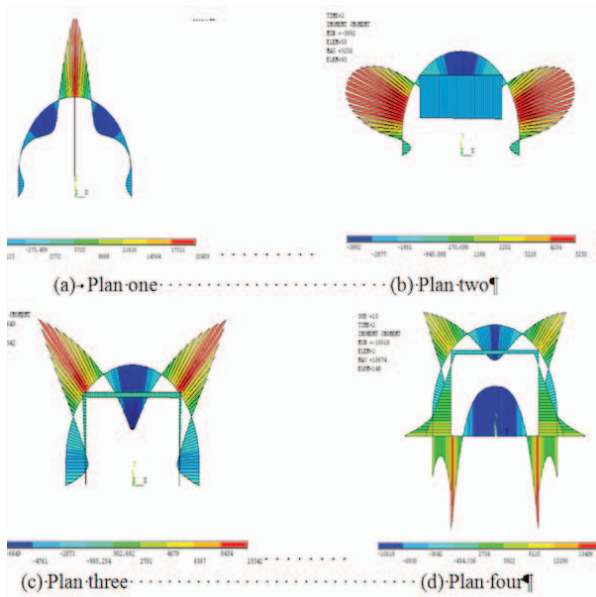
Figure 3. Bending stress distribution of conventional U-steel support.

The bending stress distributions of the four kinds of enhanced U-steel supports are shown in Figure 4. From Figure 4(a)–(d), the following conclusions can be made:

- When Plan 1 is adopted, the bending stress is the largest at the top center with respect to the other three plans. Consequently it is possible that the steel support will bend at the top center, resulting in the horizontal deformation of the side steel beams. Although the deformation at the top center can be constrained through installation of the vertical beam at the roadway center, it will easily punch into the soft floor and lose its function as an auxiliary support when the floor is soft.
- When Plan 2 is adopted, the bending stress at the top part of the U-steel support is reduced due to the horizontal beam. However the bending stress is still large, possibly inducing a large deformation.

Table 1. Four kinds of enhanced U-steel supports.

Plan	Support description	Schematic diagram
Plan one	U-steel support with a vertical beam	
Plan two	U-steel support with a horizontal beam	
Plan three	U-steel support with a horizontal beam and two vertical beams	
Plan four	U-steel support with two horizontal beams and two vertical beams	


Figure 4. Bending stress distribution of four kinds of enhanced U-steel supports.

- When Plan 3 is adopted, the bending stress at the top of the support is reduced relatively, while those at both sides of the support are reduced significantly. Conversely, the bending stress increases greatly at the shoulder, the deformation of which, however, is limited because of the erection of a horizontal beam and a vertical beam at each side.
- When Plan 4 is adopted, the bending stress at the top is relatively small, and decreases significantly at both sides of the support, but increases in relatively large amount at the shoulder, the deformation of which is limited due to the erection of a horizontal beam and a vertical beam at each side.

Therefore, through the addition of some vertical and horizontal beams to the U-steel support, its stress distribution can be altered and deformation limited to a certain extent. As a result, the bearing capacity of the overall support can be improved. In comparison, Plan 3 and Plan 4 have obvious advantages, and both can improve the stability of the steel support. However, plan 3 is easier to implement in the field because it has no lower horizontal beam.

NUMERICAL AND EXPERIMENTAL ANALYSES OF PRESSURE-RELIEF DRILL HOLES

Yi et al., (2011) studied the effect of pressure release of drilling boreholes in the soft and hard coal seams and found that its effect on pressure relief was better in soft coal seams. They used FLAC3D to analyze the stress distribution in the surrounding rocks of the coal roadway after pressure-relief holes are drilled. Figure 5 shows the model of pressure-relief drill holes, while Figure 6 shows the abutment pressures around the coal roadway before and after the holes are drilled.

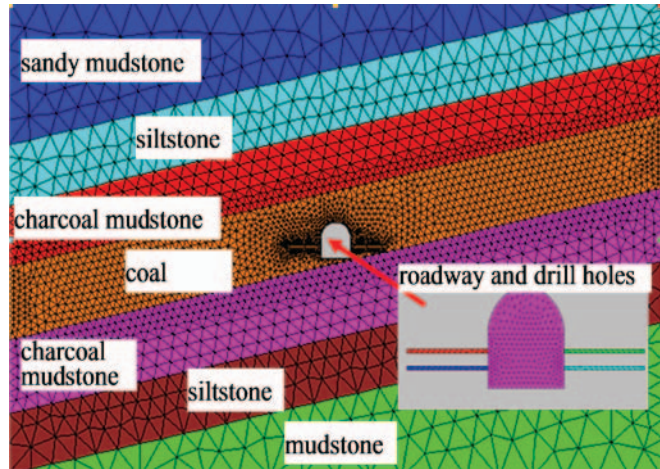
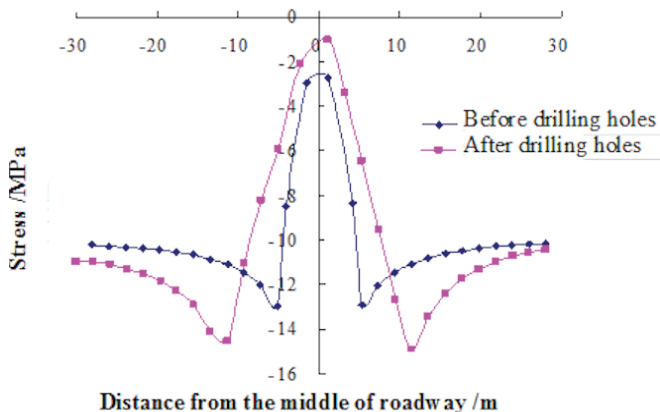

Figure 5. Pressure-relief drilling model.

Figure 6. Abutment pressure distribution of the roadway.

Figure 6 shows that there are high stress zones in the surrounding rocks before the pressure-relief holes are drilled. These high stresses are effectively released and significantly reduced after drill holes are completed. Consequently the coal roadway is located in the low stress area, while the highest stress is transferred to the deep portions of the surrounding rocks. As a result, the stability of roadway is enhanced.

Du et al., (2009) analyzed the factors affecting the pressure relief by drilling holes in the coal seam, including hole diameter, hole length, hole space etc. Table 2 shows the pressure-relieve drilling holes design used in this research.

Table 2. Parameters of pressure-relieve drilling hole.

Case	Diameter (mm)	Length (m)	Row Spacing (m)
1	89	3.0	1.5
2	89	3.0	1.0
3	89	3.0	0.5
4	110	3.0	0.5
5	110	5.0	0.5

Due to its softness, hole wall is rough. Some drill holes are closed up completely one month after drilling. In order to study the deformation characteristics of drill holes, two devices were employed to measure borehole deformation. The hole diameter deformation and the hole volume change meters are shown in Figures 7 and 8, respectively.

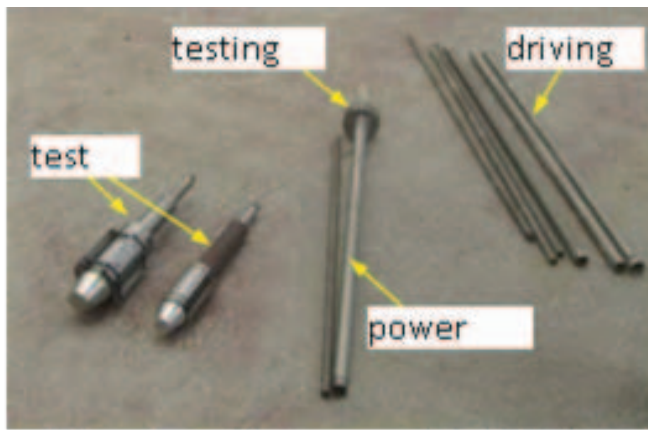


Figure 7. Hole diameter deformation meter.



Figure 8. Hole volume change meter.

The hole diameter deformation meter is made of a test head, combination rods, driving rods, a testing unit, and power rotary, that can determine the change in drill hole diameter. The hole

volume change meter is made of bag, vent, control valves, water injection pipe, and water meter. When the hole diameter is reduced it squeezes the water out of the water bag and the amount of water squeezed out is a measure of the volume change of the drill hole. Multiple sets of experimental data show that the rate of volume changes of the drill hole basically decreases gradually from outside to inside. About forty days after drilling, the maximum rate of shrinkage in drill hole cross-section was 60% ~ 65%. By this time re-drilling of the hole should be considered.

SUPPORT DESIGN IN UNDERGROUND COAL ENVIRONMENTS

4.1 Structurally enhanced support design

The test site was located at the gateroads of Panel 2201, Yonghua No.1 mine in the Dragon mining area as shown in Figure 9. Due to difficulty in supporting the coal roadways, the roadways were developed in rock under the coal seam floor. A short panel width (or face length) was adopted in order to reduce the service time of coal roadways and support maintenance.

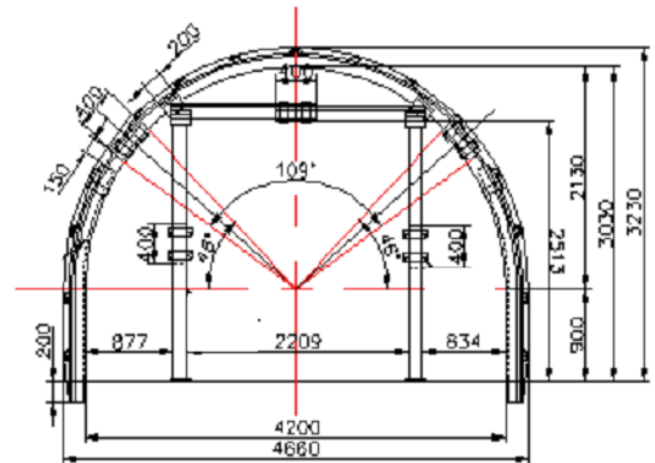


Figure 9. Test site details.

Originally, the conventional U36-steel support was used in the gateroads of this panel. The original gateroad support is shown in Figure 10. Based on the results of theoretical and finite element analyses, plus consideration of the on-site condition, the structurally enhanced U36-steel support was used in the gateroads. The location of the U-steel horizontal and vertical beams on both sides was considered from the point of view of effective use of space. The specific dimensions of the structurally enhanced support are shown in Figure 11. The roadway cross-section, 4.2 m wide by 3.23 m high, has been increased for the new support. The vertical and horizontal beams are connected to the conventional U-steel support by a section of U-shaped welded steel. This way it maintains stable through the use of both support and friction.

4.2 Design of pressure-relief drill holes

The holes were drilled between two steel supports spaced at 500 mm. In each row, the hole spacing was 500 mm. The angle of the

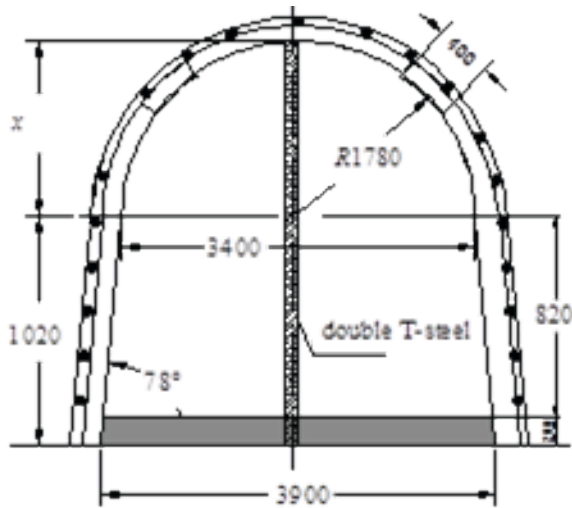


Figure 10. Original support design (mm).

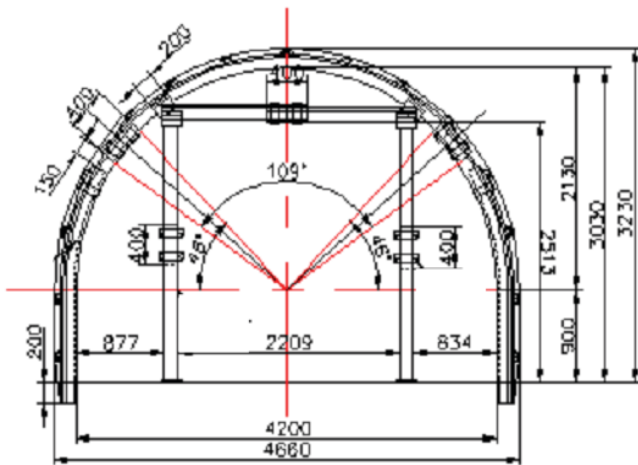


Figure 11. New support design (mm).

drill hole was 15° upward, and the depth was 3000~5000 mm. The layout of drill holes at the gateroads is shown in Figure 12.

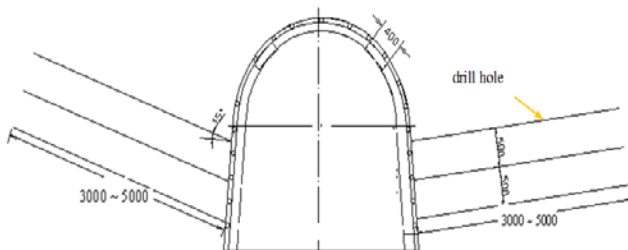


Figure 12. Layout of drill holes in the gate roads.

FIELD EXPERIMENT AND EVALUATION of SUPPORT EFFECTIVENESS

Figure 13 shows the gateroads supported with the conventional U36-steel supports one half month after development. From Figure

13, it can be seen that the cross-section of the gateroads supported by the original support design became smaller, and the whole U36-steel support sank into the floor, in particular the roof deformation is larger. There was some floor heave, and some broken coal rib was squeezed out into the gateroad. But the U-steel support has not damaged seriously, although gateroad deformation continued one month after development. In view of this situation, a new supporting method with the following installation sequence was developed: roadway development, installation of the conventional U36-steel support, drilling pressure-relief holes in the ribs, setting up the structurally enhanced auxiliary support, and finally re-drilling of pressure-relief holes.



(a) Roadway deformation



(b) Broken coal rib squeezed out into the gateroad

Figure 13. Original support with conventional U-steel tents, one month after development.

The hand-held pneumatic drill model ZQS - 65/2.5 was employed. The diameter and length of drill rod were 69 and 1000

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mm, respectively, while the diameter of drill head was 110 mm. The pressure-relief holes were drilled 20m outby the development face. Figure 14 shows a miner drilling a pressure relief hole, while Figure 15 shows a pressure-relief drill hole. The time lapse for re-drilling the hole was usually one or one and one half months later.



Figure 14. Miners drilling a pressure-relief hole.

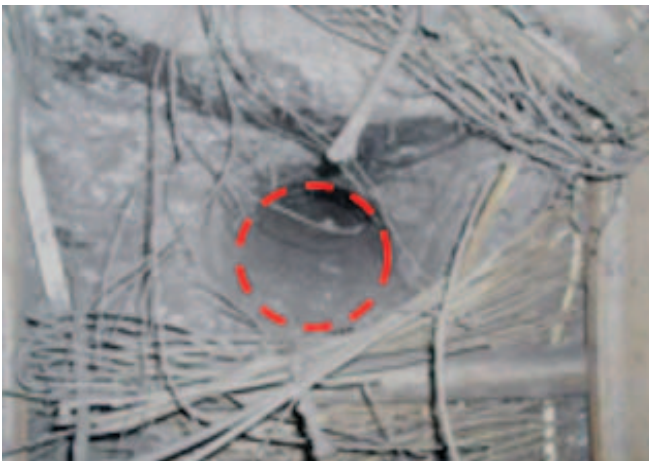


Figure 15. A pressure-relief hole.

The structurally enhanced auxiliary support is set up after the pressure-relief holes are completed. A special crew was arranged for drilling the pressure-relief holes and setting up the auxiliary support. Therefore, this crew can work simultaneously with those for roadway development, greatly improving the efficiency of roadway development. The structurally enhanced U36 steel support is shown in Figure 16. Some special U-steel connecting parts are welded in the vertical and horizontal beams in order to keep the auxiliary parts connected to the U steel shed arc. The connecting parts are shown in Figure 17. The horizontal and vertical beams are both made of two parts of U shaped steel, connected with two clamps with certain amount of retraction.

In order to monitor the roadway deformation, the “cross point” method was used to measure roadway deformations between roof and floor, and between two ribs. The results indicated that two months after development, the maximum deformation of



Figure 16. Structurally enhanced U36 steel support.



Figure 17. Connecting parts of the U-steel support.

roadway cross-section is less than 8%. Consequently the roadway deformation has effectively been controlled.

CONCLUSION

The bearing capacity of the surrounding rocks in soft coal roadways is so low that its stress is mostly born by the installed support. In a coal roadway, it is very difficult to improve the strength of coal seam. Therefore, the key to solve this problem is to improve the supporting strength of material or transfer the stress in the surrounding rock away from the support.

The conventional U-steel support is subjected to uneven load and thus its overall bearing capacity is limited. Localized stress concentration may damage the support or cause the support to punch into the floor. This research has demonstrated that the bearing capacity of the whole support can be improved by structural compensation, such as addition of horizontal and vertical beams made of U-shaped steel.

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By drilling pressure-relief holes in the ribs of a soft coal roadway, the stress distribution of surround rock in the roadway can be improved. This simple method can transfer the high stress in the surrounding rock to deeper parts and reduce the pressure on the artificial support.

A new supporting method developed in this research was applied successfully in the very soft coal seam in Dragon mining area.

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Influence of a Coal Mine Subsidence on An Adjacent Dam

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ABSTRACT

The WTZ coal mine is located adjacent to an auxiliary dam in P. R. China. The mining-induced ground movements are calculated using a 3-D numerical procedure based on the finite difference method (FDM) to judge whether the extraction of the coal seam will have a negative impact on the dam. First, the values of the rock mass mechanical parameters in the numerical model are estimated using the available literature that relates intact rock and discontinuity properties to rock mass parameters. Then, based on available surface subsidence monitoring data for an area in WTZ mine, the main mechanical parameters of coal and rock masses are calibrated by a back analysis procedure that combines an experimental design technique with numerical simulations. Then the reliability of these rock mass parameters is verified. Finally, according to the planned mining sequence, predictions of mining induced surface subsidence are computed in steps, and the boundary of the ground movement area of each step is determined. The nearest distance from the boundary of the surface movement area to the edge of the dam foundation was found to be about 1347 m. The maximum surface subsidence within the mining area turned out to be 2.7 m, and the maximum settlement point was found to be close to west-south corner of A2 mined area. These findings led to the conclusion that the coal seam mining would not cause dam damage.

Keywords: Mining; Subsidence prediction; Numerical modeling; Finite different method.

INTRODUCTION

The WTZ coal mine covers an area of approximately 37.36 km² and is located in Handan City, Hebei Province, the People's Republic of China. WTZ's southwest side is adjacent to the Yuecheng Reservoir (Figure 1). A part of an auxiliary dam for the reservoir is within the mine area. To prevent the dam from being damaged from mining-induced surface movements, two areas were set within the WTZ coal mine: a prohibited mining area and a restricted mining area. In the prohibited mining area, in which the shortest distance between any site and the dam foundation is less than 3 km, mining activities are strictly forbidden. In the restricted mining area, in which the shortest distance between any site and the dam foundation is longer than 3 km but shorter than 6 km, mining

activities are cautiously allowed according to the monitoring data of mining-caused surface movements. Therefore, it is essential to investigate the influence of WTZ coal mine surface movements on the auxiliary dam when mining in the restricted mining area.



Figure 1. Salient features of the WTZ coal mine (contour interval is 30 m).

Research on coal mining subsidence prediction has taken place for nearly two centuries. Empirical formulas, curves, graphs and influence functions have been developed based on numerous measurements of mining subsidence (Knothe, 1957; Ren et al., 1987; Álvarez et al., 2005; Luo & Cheng, 2009). These methodologies have not explicitly taken into account the effect of geologic structures, geomechanical properties and in situ stress of the rock masses at the mine site on subsidence prediction. The numerical methods predict subsidence through a mechanistic approach, using rock and coal mass geological, physical and mechanical properties. The suggested models from this group are random field theory based models (Toraño, 2000), elastic or visco-elastic theory based models (Astin, 1968), and continuum or discontinuum numerical models (Alejano et al., 1999; Tajduš, 2009). Both the finite element method (FEM) and finite difference method (FDM) can handle nonlinear and non-homogeneous characteristics of rock masses. Therefore, they have become important tools for coal mining subsidence prediction. The

FDM seems to be an appropriate method to perform subsidence modeling, especially because the Lagrangian formulation permits materials to yield and flow and the grid to deform in a large-strain mode (Alejano et al., 1999).

In this paper, the mining-induced subsidence of the WTZ coal mine is predicted by FDM. FLAC^{3D}4.0 software, developed by the Itasca Consulting Group, Inc. (2009), is used in this study. First, the computational model is constructed from available geologic information of the WTZ coal mine for FLAC^{3D}. For this model, first, the mechanical parameters of the rock masses in the strata are determined empirically. Then they are calibrated based on measured subsidence data resulting from a mined area using a back analysis method that combines an experimental design technique with a numerical simulation. The validity of these calibrated parameters is then verified through a comparison between the calculated and the measured surface subsidence. Finally, investigations of surface subsidence caused by mining are carried out in three mining steps, and the boundaries of the surface movement areas of these steps are determined.

GEOLOGY AND MINING CONTEXT

Geology

The WTZ coal mine is located east of the Fengfeng coalfield. Within the coal mine, there are seven major faults: F1, F2, F3, F5, F17, F25 and F31 (Figure 1). They are normal faults. The fault zones of these faults are approximately 10-30 m in width. They are composed of mudstone and mylonitic rocks as well as breccia.

The stratigraphic sequence in WTZ, in the order from oldest to newest, is Ordovician (O), Carboniferous (C), Permian (P), Tertiary (T) and Quaternary (Q) (Figure 2). The No. 2 coal seam, which is approximately 3.3 m thick and located in the Permian stratum, is the main minable seam in the WTZ mine. Within the study domain, the elevation of the coal seam roof varies widely, from the highest point of -200 m in the mid-west to the lowest of -1300 m in the southeast of the coalfield. The depth of the roof varies from 500 to 1400 m. For further details on the geology and the fault system, the reader is referred to Xu et al. (2013).

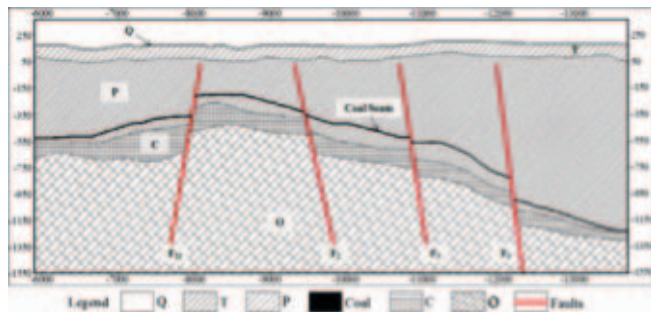


Figure 2. A typical geologic cross section of the mine field in Y-Y direction in A1 area.

Mining history and surface subsidence monitoring

From 2001 to 2007, seven mining panels in the No. 2 coal seam were excavated in the north-central WTZ mine, as shown in Figure 3 in the mined area A1 (Figure 1). Each panel is 150 m wide

and 500 – 1000 m long. The fully mechanized top-coal-caving technique was adopted to extract the coal. Table 1 lists the mining dates of the seven mining panels.

The surface subsidence was monitored during mining. A total of 79 surface subsidence monitoring points were set up in the mined area along the measuring line L shown in Figure 3 to record the values of surface subsidence during the mining period starting 2001.

From 2001 to 2008, the engineers had conducted subsidence measurements about 12 times. However, because some of the data have been lost, only five surface subsidence profiles along line L were available for this study (Figure 4). The horizontal locations of Panel 01 through Panel 07 are shown at the bottom of Figure 4. By August 28, 2008 the surface settlement caused by the excavation reached a stable state with a maximum settlement of 1.62 m (Figure 4).

COMPUTATIONAL MODEL

In this study, FLAC^{3D} was utilized to conduct numerical prediction of coal mining-induced rock movement. FLAC^{3D} is a 3-D finite difference computer program for solving rock and soil mechanics problems.

The rectangular region enclosed by the red lines shown in Figure 1 was selected as the computational model domain based on the spatial distribution of current and future areas for mining. As shown in Figure 1, the x-axis of the Cartesian coordinate system is N 20° E, and the z-axis points in the vertical upward direction. The simulated elevation of the model ranges from -1550 m to 250 m. The nearest horizontal distance from the mining boundary to the model boundary is 2 km.

The Ordovician, Carboniferous, Permian, Tertiary and Quaternary strata, as well as the coal seam in the Permian stratum, are simulated in the model. The model also contains all the 7 main faults. The geological model was meshed using a set of triangular prism elements. A mesh is formed with a total of 1,675,940 elements and 3,127,544 nodes. The incremental elastic-plastic constitutive model and Mohr-Coloumb yield criterion are used in this study. Displacement boundary conditions are used to apply in situ stress in this study. For details on the set up of the numerical model, the reader is referred to Xu et al. (2013).

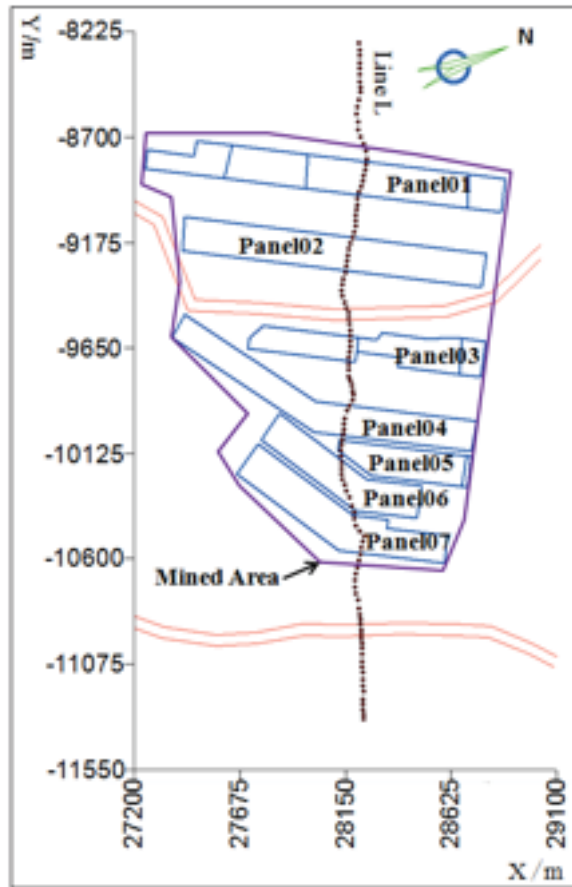
DETERMINING THE MECHANICAL PARAMETERS OF THE ROCK-MASSSES

Mechanical parameters of the rock-masses

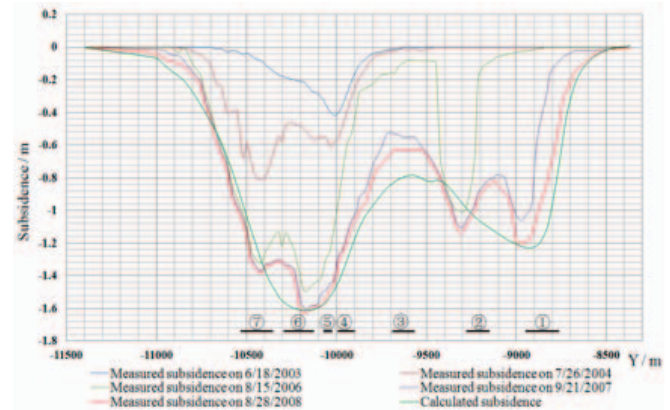
Rock mass is composed of intact rock and discontinuities. The mechanical parameters of intact rock obtained by laboratory experiments are quite different from those of rock masses (Hoek & Brown, 1997; Mohammad et al., 1997; Kulatilake et al., 1993, 2004; Wu & Kulatilake, 2012). Estimations by Mohammed et al. (1997) provide the following values for rock masses: a mean deformation modulus equal to 47% of the corresponding intact rock; a mean uniaxial compressive strength equal to 28% of the laboratory intact rock strength, and a mean tensile strength equal to 50% of the laboratory intact rock tensile strength. For a granitic gneiss rock, Kulatilake et al. (1993) obtained a mean rock mass

Table 1. Mining dates of the seven mining panels located in the mined area.

Date of mining	Panel 01	Panel 02	Panel 03	Panel 04	Panel 05	Panel 06	Panel 07
Start date	Jan. 1 st , 2006	Nov. 1 st , 2004	Oct. 1 st , 2006	Sep. 1 st , 2002	Aug. 1 st , 2005	May 1 st , 2001	Jan. 1 st , 2004
End date	July 30th, 2007	Jan. 31st, 2006	Oct. 30th, 2007	Nov. 19th, 2003	May 11th, 2006	June 28th, 2002	Mar. 31st, 2005

**Figure 3. Selected surface subsidence monitoring points along Line L in the mined area.**

deformation modulus equal to approximately 30% of the intact rock Young's modulus. For Aspo diorite rock located at a depth of 485 m at Aspo Hard Rock Laboratory, Sweden, Kulatilake et al. (2004) estimated the mean rock mass modulus to be 51% of the intact rock Young's modulus. Additionally, the mean rock mass Poisson's ratio was estimated to be 21% higher than the intact rock Poisson's ratio. In a recent study of a limestone rock mass, Wu and Kulatilake (2012) have obtained a mean rock mass strength equal to approximately 45% of intact rock strength. For the same rock mass, they have obtained a mean deformation modulus equal to approximately 50% of the intact rock Young's modulus. The mean shear and bulk moduli for the same rock mass were approximately 40% and 30% of the intact rock shear modulus and bulk modulus, respectively. The mean Poisson's ratio for the rock mass was approximately 14% higher than the intact rock Poisson's ratio.

**Figure 4. Measured and calculated surface subsidence profiles along L, shown in Figure 3.**

In this paper, the values of the deformation modulus and Poisson's ratio of rock masses in different strata were first estimated empirically based on the aforementioned literature as well as by comparisons with the mechanical parameter values of the coalfields adjacent to the WTZ mine (Table 3 of Xu et al., 2013). In this table, for each rock formation, the value of the Poisson's ratio for rock mass is approximately 11%-20% higher than that of the intact rock and the value of the deformation modulus is approximately 30%-50% of the intact rock Young's modulus. The values of C and ψ of rock masses were determined through comparisons with those collected from other mines in the Fengfeng coalfield. Then the rock mass parameters were calibrated based on measured subsidence data resulting from a mined area using a back analysis method that combines an experimental design technique with a numerical simulation (Xu et al., 2013). The calibrated rock mass parameter values are given in Table 2.

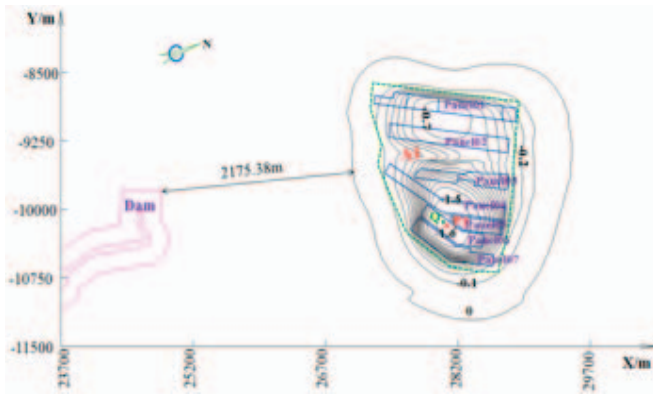
Verification test

The coal seam within the mined area A1 of WTZ had been completely extracted before 2008 and the resulted surface subsidence was monitored during mining. A simulation within the mined area was conducted using the calibrated rock mass parameter values shown in Table 2.

Figure 4 is a comparative map between the measured and calculated surface subsidence curves on line L shown in Figure 3. It can be observed that on line L, the subsidence values of the calculation tally closely with those measured on August 28, 2008. Figure 5 shows the surface subsidence contours produced by the results of FLAC3D. The measured maximum surface settlement point Q (the green point), with a subsidence of 1.62 m, and the calculated maximum surface settlement point P1 (the red point),

Table 2. Calibrated mechanical properties of the representative rock formations and faults.

NO.	Strata	Rock formation	$E(\text{GPa})$	μ	$C(\text{kPa})$	$\varphi(^{\circ})$
1	Q		0.03	0.37	53.34	17
2	T		0.85	0.36	196.34	15.05
3	P	Shiqianfeng	2.84	0.17	901.32	25.56
4		Shangshihezi	2.87	0.14	922.2	25.56
5		Xiashihezi	2.73	0.16	907.5	25.56
6		Shanxi	2.83	0.19	904.4	27.83
7	C	Taiyuan	2.32	0.22	809.3	20.31
8		Coal	0.23	0.32	160.78	15.05
9		Benxi	3.29	0.22	916.78	21.05
10	O		8.34	0.15	2683.1	30.1
11	Faults		0.04	0.35	48.7	18.94

**Figure 5. Calculated subsidence contours (at a 0.1 m interval) after finishing mining within A1.**

with a value of 1.6 m, are very close to each other. Therefore, the calibrated rock mass parameter values given in Table 2 are reliable.

As shown in Figure 5, the shortest distance from the boundary of the surface subsidence area to the edge of the dam foundation is about 2175 m. The largest movements in X and Y directions are 0.55 m and 0.5 m, respectively. The shortest distances from the X and Y-directions ground movement area to the dam foundation are 2128 m and 2162 m, respectively. Therefore, the coal seam mining in A1 has not caused dam damage.

INFLUENCE OF COAL MINE GROUND MOVEMENT ON ADJACENT DAM

According to the mining plan, the remainder coal seam in the WTZ coal mine outside of the prohibited mining area will be extracted in two steps. The first step is to extract the coal seam within the area A2, which is the restricted mining area except A1. The second step is to mine the coal seam within the area A3, which is an area of WTZ coal mine outside of both the restricted and prohibited mining areas. In this paper, according to the aforementioned plan, the mining sequence is simulated in two steps after simulating the mining within area A1 to calculate the surface subsidence. The areas A1, A2, and A3 are shown in Figure 1.

Mining simulation within the area A2

After simulating the extraction within the area A1, mining in the area A2 was simulated. There are 22 panels in A2 (Figure 6). Figure 6(a) shows the calculated surface subsidence contours after finishing mining in A2. The point P2 with the maximum surface settlement of 2.7 m is located at the west-south corner of the A2. The subsidence value at P1 increases from 1.6 m to 1.9 m; that indicates mining in A2 increases subsidence in the area A1. After finishing mining

in A2, the shortest distance between the subsidence area and the dam foundation turned out to be 1356 m. The maximum X-direction ground movement value is 0.7 m, and the nearest distance between the X-direction ground movement area and the dam foundation is 1364 m (Figure 6b). The largest Y-direction ground movement value is 0.9 m, and the shortest distance from the Y-direction ground movement area to the dam foundation is about 1358 m (Figure 6c). Although the ground movement area is enlarged, mining in A2 has not caused any damage to the dam.

Mining simulation within the area A3

Mining simulation within area A3, which is composed of 11 panels, was conducted after finishing extraction of A1 and A2. Figure 7 shows the calculated ground movements after finishing mining in A3. The place with the largest settlement of 2.7 m is still located at the west-south corner of A2 (Figure 7a). The subsidence value of P1 is basically the same as before; that indicates that mining in A3 results in only slight subsidence in the area A1. The shortest distance from the surface subsidence area to the dam foundation is about 1347 m. The maximum X-direction ground movement value is 0.7 m, and the nearest distance between the X-direction ground movement area and the dam foundation is about 1360 m (Figure 7b). The maximum Y-direction ground movement value is 0.9 m, and the shortest distance from the Y-direction ground movement area to the dam foundation is about 1353 m (Figure 7c). The abovementioned results indicate that no mining-induced damage is caused to the dam after finishing mining A1, A2 and A3.

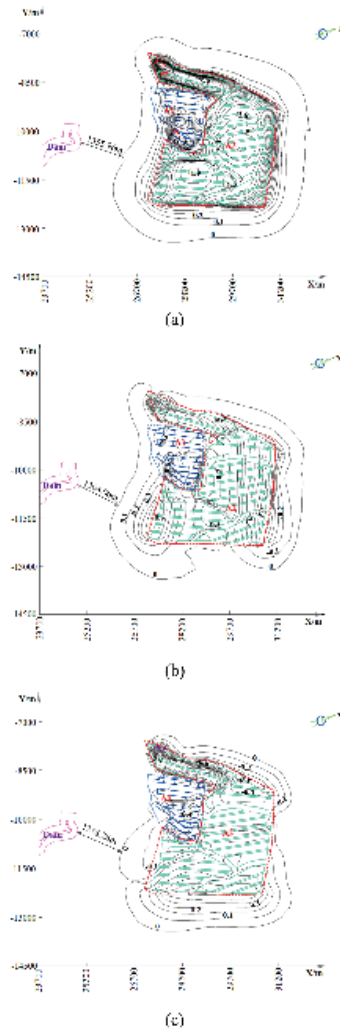


Figure 6. Calculated surface movement contours (at a 0.1 m interval) after finishing mining within A2: (a) surface movements along Z-direction; (b) surface movements along X-direction; and (c) surface movements along Y-direction.

CONCLUSIONS

To investigate the mining-induced negative influence to an auxiliary dam of Yuecheng reservoir of WTZ coal mine, FLAC^{3D} was used to calculate the ground movements resulting from mining within the areas A1, A2 and A3. First, the computational model included a simplified geologic model constructed from available geologic information for the site. Then, the values of the mechanical parameters of rock masses in different strata were determined empirically based on the literature as well as by comparisons with those of the coalfields adjacent to the WTZ mine. Then the rock mass parameters were calibrated based on measured subsidence data resulting from the mined area A1 using a back analysis method that combines an experimental design technique with a numerical simulation. These calibrated parameters were then verified by comparing calculated settlements with measured settlements due to mining in A1. Finally, mining was simulated within the areas A2 and A3 in steps according to the mining sequence. The contours of ground movements resulting from numerical modeling are shown in the paper. The shortest distance

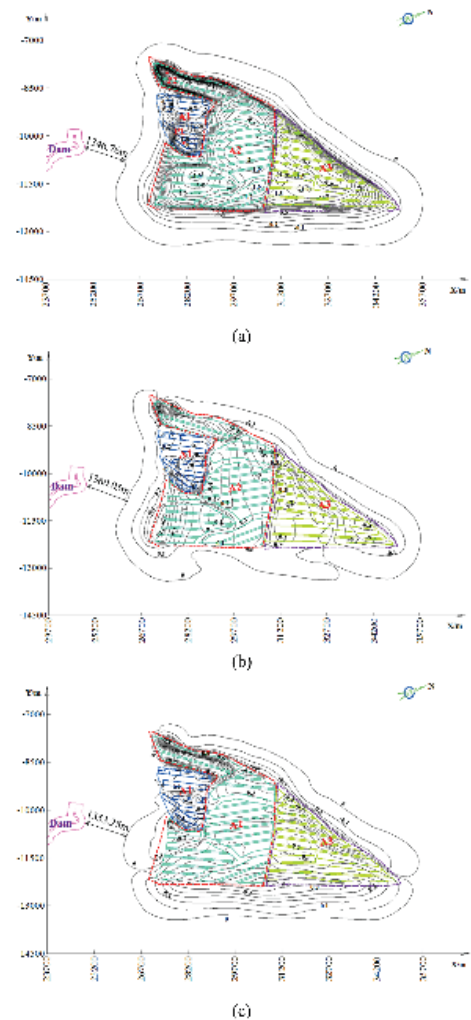


Figure 7. Calculated surface movement contours (at a 0.1 m interval) after finishing mining within A3: (a) surface movements along Z-direction; (b) surface movements along X-direction; and (c) surface movements along Y-direction.

from the areas with mining-induced ground movements to the dam foundation was found to be about 1347 m. The maximum surface subsidence resulting from mining within the areas A1, A2 and A3 turned out to be about 2.7 m. Therefore, mining in the WTZ coal mine outside of the prohibited mining area will not cause damage to the auxiliary dam of Yuecheng reservoir.

ACKNOWLEDGMENTS

This research was supported by the Natural Science Foundation of China (Grant Numbers 40602037 and 40872183) and the Fundamental Research Funds for the Central Universities.

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Research on the Characteristics of Mine Pressure and Overlying Strata Movement in Downward Mining Longwall Panels in Steeply Dipping Seams

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ABSTRACT

Many longwall panels in China are laid out along the dip direction such that the direction of longwall retreat mining is along the strike direction of coal seams. Downward mining longwall panels in steeply inclined seams refers to those that are laid out in a non-uniformly developed fold structure where the seam is not only steeply dipping, but also highly undulated along the strike direction, causing longwall mining extremely difficult because it has to deal with large dip angles that vary both in the mining and faceline directions. Obviously, the characteristics of mine pressure and overlying strata movement in downward mining longwall panels in steeply dipping seams have unique patterns that are very much different from those cases of mining the flat or gently inclined coal seams. The effects of those unique patterns presented a big challenge to production and safety.

Almost all previous and current researches on ground control have been concentrated on issues related to the flat and gently inclined coal seams. Conversely, research on steeply inclined coal seams, especially those with large angle variation both along the seam dip and strike directions is almost none. Therefore, it is very important to initiate a study to better understand the characteristics of mine pressure and overburden strata movement for this special type of geological condition.

The objective of this paper is to monitor and analyze the characteristics of mine pressure and overlying strata movement in downward mining longwall panels in steeply inclined seams, the angle of which varies greatly along both the seam dip and strike directions.

The measured data has conclusively demonstrated that the characteristics of mine pressure and overlying strata movement in downward mining longwall panels in steeply inclined seams are different from those for the flat and gently inclined coal seams. The large downward angles of coal seam undulation along the strike (mining) direction can obviously affect the roof movement and reduce strata pressure. Study in this research showed that appropriately increasing the rate of mining (or face advance) is positive to roof control.

INTRODUCTION

The steeply inclined coal seam is usually referred to coal seams with dip angle between 35° and 55° . The inclined coal seams are commonly found in many countries, including China, Germany, Russia and Poland. Many studies have been performed to improve mine safety and production for this type of coal seams. However, most of those researches in these countries have been concentrating on the equipment of mechanized longwall mining (Mathur et al., 1993).

In China, the reserves and annual production of the steeply inclined seams account for roughly 15%~20% of the total coal reserves and 10% of the total annual coal production, respectively (Wu, 2006). Since the 1980s, many studies on theories and mining methods for steeply inclined coal seams have been performed in China, and most of them focused on non-mechanized mining methods. In recent years, with the development of fully mechanized longwall mining in the flat and gently inclined coal seams in China, some progresses have been made in the technology of steep seam mining. Yin et al., (2001) summarized the basic behavior of strata movement in inclined coal seams based on the results of physical simulation and FLAC computer modeling. Zhang et al., (2010) built a thin plate mechanical model to study the mechanism of roof fractures on the up-dip & down-dip longwall mining panels in steeply inclined seams. Furthermore, Wu et al., (2013), Wu (2006), and Wu et al., (2007) conducted a series of studies and tests on longwall mining in steep coal seams and established a dynamic model of a roof-support-floor (R-S-F) system under asymmetrical constraints in mining steep seams. They also developed a theory of strata control and hazard prevention. Regardless, research on steeply inclined coal seams with large variation in dip angle both along the seam dip and strike directions is almost none. Therefore, it is necessary to initiate a study to better understand the characteristics of mine pressure and overburden strata movement for this special type of geological condition.

Description of Longwall Mine

The longwall panel for this research was laid out in the east wing mining area of a coal mine located in Anhui Province, eastern China. The average seam thickness was 2.83 m (9.3 ft), with

overburden depth varying from 465 m (1496 ft) to 607 m (1991.5 ft). The immediate roof was mudstone, and the main roof consisted of in ascending order fine sandstone, coal rider, mudstone, and sandy mudstone. The immediate floor and main floor were mudstone and fine sandstone, respectively.

The panel of interest, 118 m (387 ft) wide and 940 m (3084 ft) long, was laid out in a non-uniformly developed syncline structure where the seam was not only steeply dipping, but also highly undulated along the strike direction. In this panel the coal seam first dips downward (and thus requiring mining downward) in the set-up room side and then dip upward (and thus requiring mining upward) on the recovery room side (Fig. 1). However, only the characteristics of mine pressure and overlying strata movement during downward mining period will be discussed in this paper. The dip angles in the mining and faceline directions varied from 10° to 42° (average 27°) and from 13° to 31° (average 25°), respectively. The distribution of dip angle in the faceline and mining directions during the observation period is shown in Figs. 2 and 3, respectively.

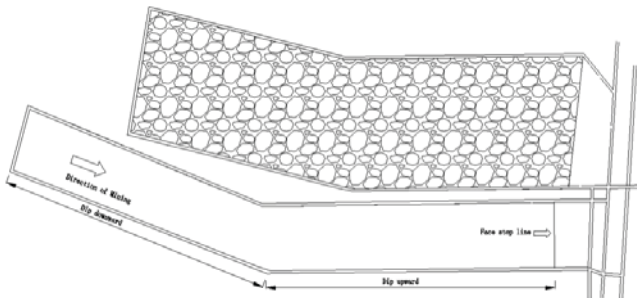


Figure 1. Panel layout.

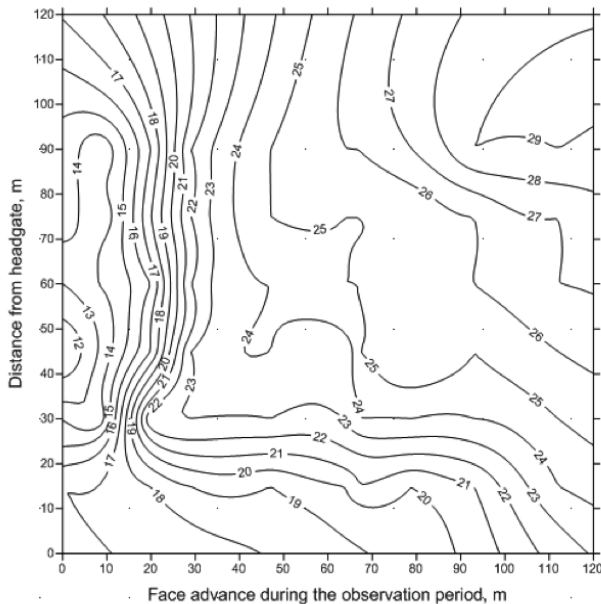


Figure 2. Distribution of dip angle (°) in the faceline direction during the observation period.

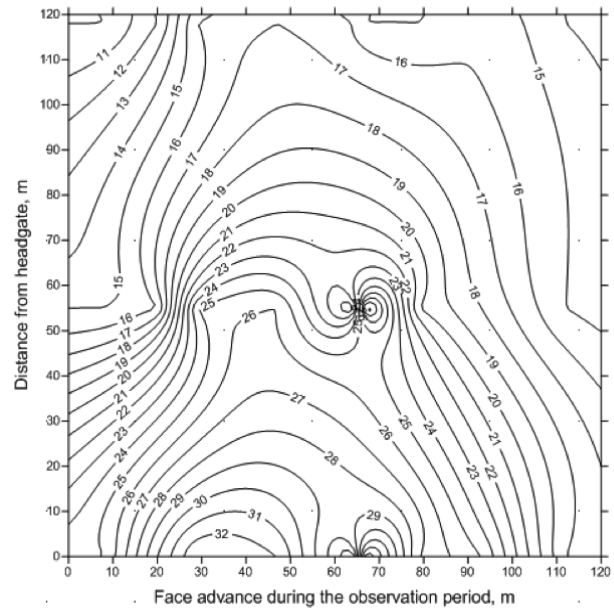


Figure 3. Distribution of dip angle (°) in the mining direction during the observation period.

This panel employed the fully mechanized longwall mining. It was surrounded on three sides by solid coal, while the 4th (or west) side was adjacent to a previously mined-out panel. The model ZZ7600/18/38 4-leg shields with yield capacity 7600KN were used.

FIELD MEASUREMENTS AND ANALYSIS OF RESULTS

Mine pressure monitoring plan

The main parameter of underground monitoring was support resistances. The shield leg pressure data were obtained by the support resistance automatic recording system (Fig. 4), which was composed of a KBJ-60-1 continuous pressure recorder (Fig. 5), a data collector, a wireless communication adapter, and a computer. The model KBJ-60-1 continuous pressure recorder was mounted on the hydraulic leg of the shield in each observation station to record continuously the hydraulic leg resistance at 5 minutes interval. Pressure data were collected from the continuous pressure recorder by the data collector weekly and then transmitted to the computer via the wireless communication adapter. Six observation stations along the faceline were set up by placing a pressure recorder in every 10 shields (i.e., the station spacing was 15 m (49.2 ft)) from the eighth shield on the headgate side (i.e., bottom of the face) to the tailgate side (i.e., top of the face) (Fig. 6).

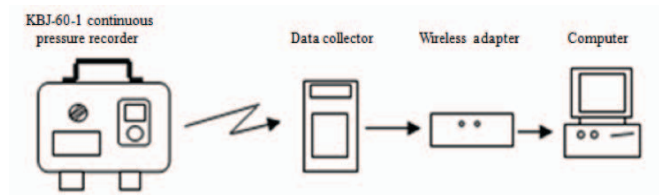


Figure 4. Support resistance automatic recording system.



Figure 5. Model KBJ-60-1 continuous pressure recorder.

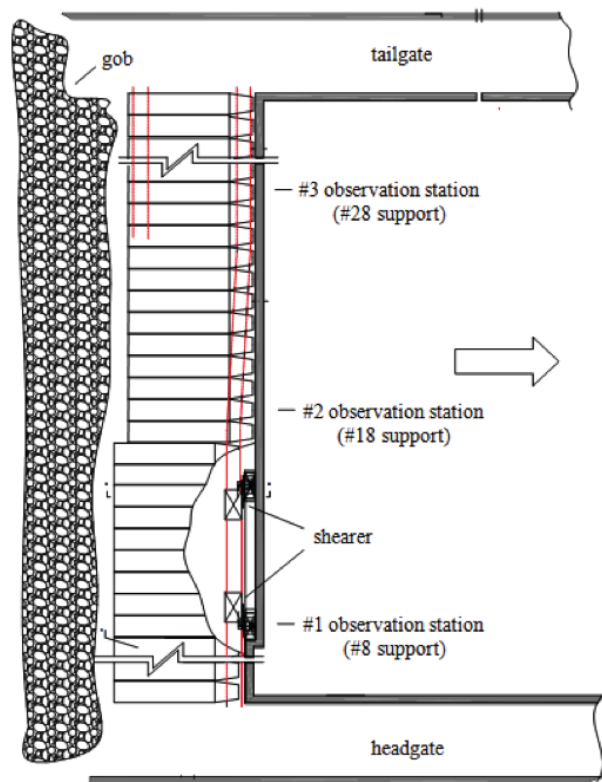


Figure 6. Layout of the observation stations.

In order to determine the abutment pressures, a pressure monitoring system was set up at both the headgate and tailgate. It consisted of mounting pressure recorders on a group of 10 single

hydraulic props (about 30m in length) and monitored the variation of prop resistance for each of the 10 props.

Distribution of shield support resistance

Support resistances from April 10, 2009 to June 1, 2009 were monitored and used for analysis in this paper. The time-weighted average resistances (TWAR) of the shields were determined and shown in Fig. 7. It moved up and down periodically in each cycle as the longwall face advanced. The average TWAR for all periodic roof weightings was 18 MPa (2609 psi) while that for non-periodic roof weighting period was 13.6 MPa (1976.9 psi). The periodic weighting characteristics were listed in Table 1. The weighting interval varied from 14.3 m (47 ft) to 20.2 m (66.3 ft), with an average of 17.6 m (57.8 ft). The periodic weighting leg pressure data of all intensity factor varied from 1.16 to 1.59 with an average of 1.32.

The distribution pattern of support resistances for the monitored panel in the mining direction was similar to that of the adjacent panel located in a moderately inclined region in the same coal seam. Those two panels employed the same mining techniques and under the same geological conditions, except that the seam dip angle was no more than 15° in the adjacent panel both in the mining and faceline directions.

Compared with the weighting characteristics of the adjacent panel (Table 2), the weighting interval of the monitored panel was larger and the duration was longer, but the weighting intensity was smaller. Generally, the weighting along the faceline direction occurred at the center part first, and then in the other parts. The leg resistance varied greatly during the whole observation period.

Support resistance along the faceline (seam dip) direction also exhibited some trends. Fig.8 shows the distribution of average TWAR along the faceline direction. As the face advanced, the support resistance was high in the upper center region and decreased gradually to both sides.

Abutment pressure distribution

As shown in Fig. 9, the front abutment pressure was approximately 12 MPa (1740 psi) at a distance of 30 m (98 ft) outby the face. It was stable until it began to increase rapidly when the face approached to within 12 m (39 ft). It reached the maximum front abutment pressure (21 MPa (3046 psi) at the tailgate side and 22.9 MPa (3321 psi) at the headgate side) when the face was 4m (13 ft) inby. Thereafter the pressure dropped drastically. Depending on the characteristic of variation, the significant influence zone of front abutment pressure was located from 0 to 12m (0 to 39 ft) outby the face.

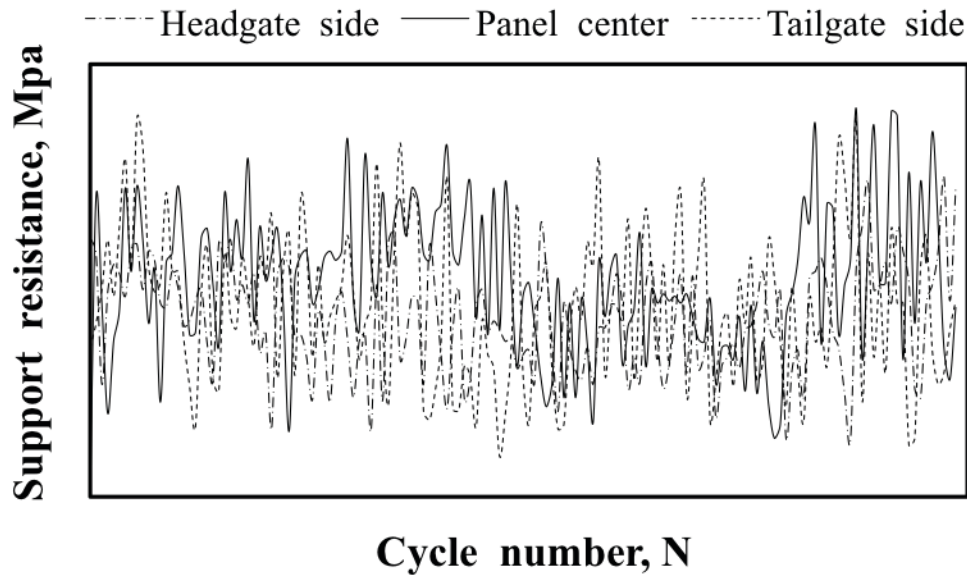
Correlation between the seam dip angle during downward mining and characteristics of mine pressure

During the observation period, the dip angle of the coal seam changed considerably, while all other geological conditions were relatively stable. This condition makes it ideal to analyze whether or not the seam dip angle affects the strata behavior.

The TWAR at different seam dip angles was analyzed (Fig. 10). It shows that when the seam dip angle increases, the shield leg

Table 1. Weighting characteristics of the monitored panel for this research.

Weighting No.	Weighting interval , m(ft)	Distance traveled during periodic weighting, m(ft)	Time-Weighted Average Resistance (TWAR), Mpa(psi)		Periodic weighting intensity factor, I
			Periodic roof weightings period	Non-periodic roof weighting period	
1	17.4(57.1)	2.83(9.3)	19.6(2836.9)	16.9(2449.7)	1.16
2	16.3(53.5)	2.33(7.6)	18.2(2638.2)	13.9(2013.1)	1.31
3	18(59.1)	4.43(14.5)	17.6(2545.4)	13.5(1956.6)	1.3
4	19.5(64)	5.7(18.7)	15.7(2280)	12.1(1755)	1.3
5	14.3(47)	3.2(10.5)	17.6(2545.4)	13.4(1943.5)	1.31
6	20.2(66)	5.63(18.5)	19.3(2802.1)	12.2(1765.1)	1.59
Average	17.63(57.8)	4(13.1)	18(2609.2)	13.6(1976.9)	1.32


Figure 7. Relationship between time-weighted average resistance (TWAR) of the shield and mining cycle.

pressure will decrease quickly. The linear relationship between the seam dip angle in the panel and TWAR can be represented by

$$p = -0.5803\alpha + 27.834 \quad (1)$$

where P is the Time-weighted Average Resistance of shield and α is the seam dip angle in the panel along the mining direction.

Fig. 11 shows the roof strata movement in mining direction of longwall panel with steeply dip angle. In order to maintain equilibrium condition of the broken roof blocks, the roof strata quickly become a stable structure under the horizontal force. Therefore, as compared to that in the flat and gently inclined coal seams, a “voussoir beam” is more easily formed when strata in the overburden are squeezed by horizontal force in longwall panels with large varying dip angles in the mining direction. The

immediate roof carries parts of the overburden weight, thereby reducing the support resistance.

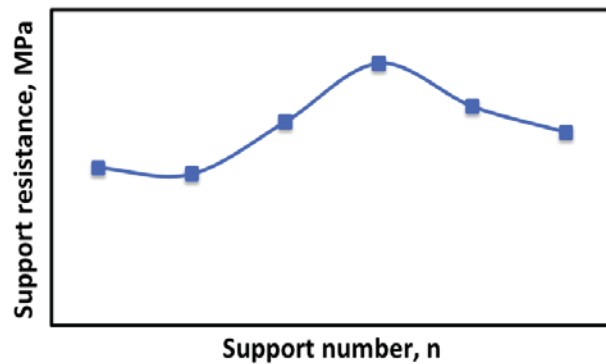
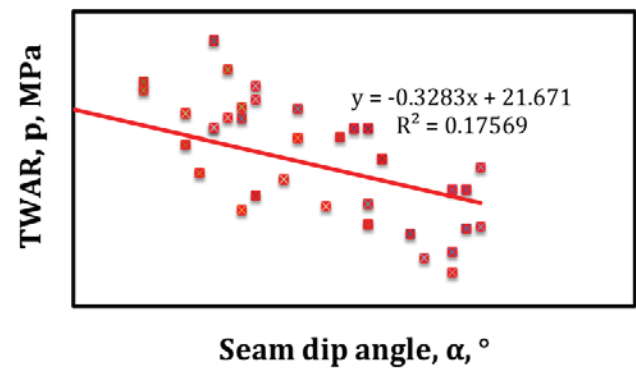
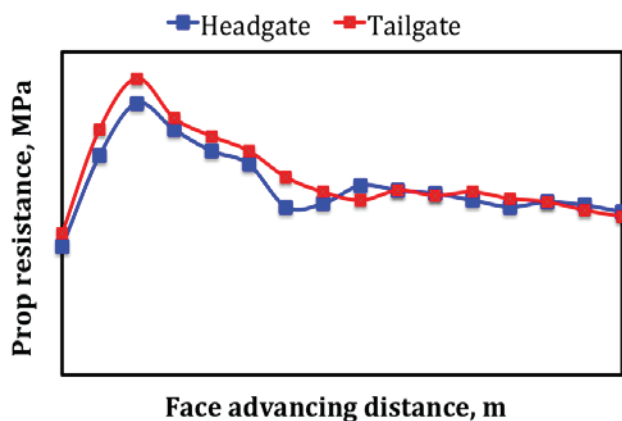
Correlation between the rate of advance of mining and overlying strata movement

The TWAR's of #8 shield at different advance rates of mining were analyzed (Fig. 12). In order to reduce the influence of periodic weighting and dip angle of the coal seam, the TWAR data obtained during the non-periodic roof weighting period and used in Fig.12 was selected. The dip angles during this observation period varied from 25° to 32° and from 20° to 25°, respectively, in the mining and faceline directions.

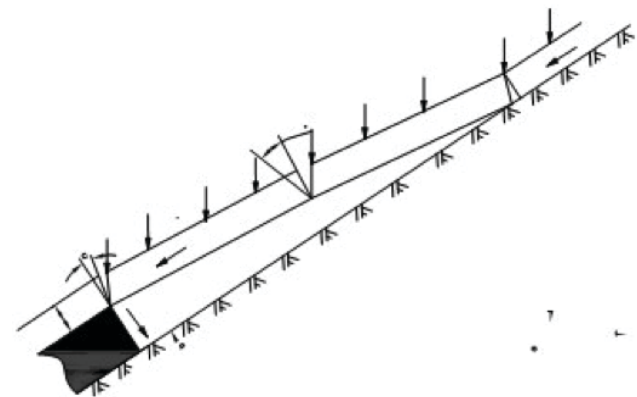
It can be seen from Fig. 12 that the longwall advance rate greatly influenced the support resistance of the shield. When the advance rate was low, the it was generally large. With the increase of the advance rate, the support resistance of the shield

Table 2. Weighting characteristics of the adjacent panel.

Weighting No.	Weighting interval , m(ft)	Distance traveled during periodic weighting, m(ft)	Time-Weighted Average Resistance (TWAR), Mpa(psi)		Periodic weighting intensity factor, I
			Periodic roof weightings period	Non-periodic roof weighting period	
1	25.2(82.6)	3.4(11.2)	23.4(3399.4)	15.7(2281.4)	1.49
2	18.6(61.)	1.9(6.2)	21.7(3150.4)	16.1(2333.7)	1.35
3	22.3(73.2)	3.1(10)	20.3(2945.1)	14.2(2059.5)	1.43
4	20.5(67.4)	3.8(12.5)	23.5(3412.6)	15.2(2201.7)	1.55
5	17.7(57.9)	2.9(9.5)	18.9(2742.1)	13.7(1987)	1.38
6	23.4(76.9)	4.3(14.2)	22.5(3260.5)	14.1(2037.8)	1.60
7	15(49.2)	4.7(15.3)	17.9(2598.8)	13.9(2014.6)	1.29
Average	20.4(66.9)	3.4(11.3)	21.7(3153.6)	14.8(2150.2)	1.47


Figure 8. Distribution of average TWAR along the faceline direction.

Figure 10. Relationship between the seam dip angle and TWAR.

Figure 9. The variation of front abutment pressure as the face advances.

decreased correspondingly. In other words, within a certain range of advance rates, increase in advance rate can effectively relieve the subsidence of the overburden strata and reduce supporting resistance. However, when the advance rate increases to certain level (3-4m/d in this example), the interaction will be much less.


Figure 11. Schematic of roof rock blocks in downward mining.

The relationship between advance rate and TWAR can be expressed by

$$p = 22.092 v^{-0.437} \quad (2)$$

where P is the Time-weighted Average Resistance of shield support and v is the longwall advance rate.

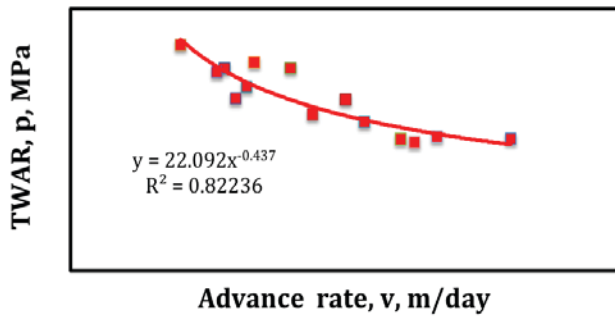


Figure 12. Relationship between advance rate of mining and TWAR of the shield support.

Fig. 13 shows the load increment of the shield support after the face stopped due to various reasons. The panel stood idle for more than 40 hours, causing the shield leg pressures to increase much more than that under normal advance rate. Within the first 2 hours, support resistance increased rapidly. Thereafter, the rate of load increase decreased gradually with time. After 15 hours, the rate of load increase approached a constant value. In other words, as time went on, the time-weighted average resistance exhibited a slow but steady increase. These measurement trends are also consistent with the research performed by the junior author of this paper for US longwall mining (Peng, 2008; Peng, 2006).

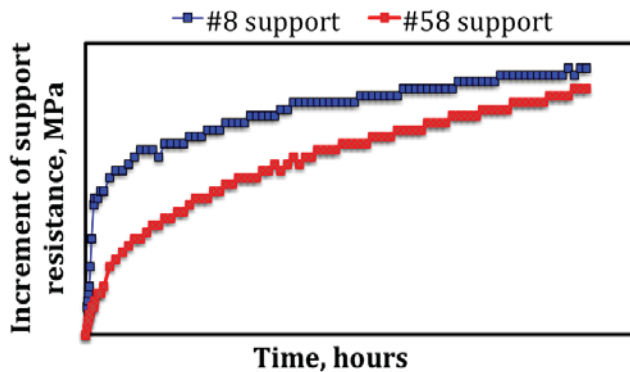


Figure 13. Load increment of shield support during idle period.

CONCLUSIONS

Compared with longwall panels in the flat and gently inclined coal seams, the measured data showed that the characteristics of mine pressure and overlying strata movement in downward mining longwall panels in steeply inclined seams have some unique patterns: the weighting interval is larger and duration is longer, but the weighting intensity is smaller. In addition, the support resistance along the faceline direction is higher in the upper center region than both sides of the panel.

The large downward angles of coal seam undulation along the strike (mining) direction can obviously affect the roof movement and reduce strata pressure. The relationship between seam dip angle in the panel and support resistance is linear in that support resistance decreases greatly with increase in the magnitude of

seam undulation. Moreover, within a certain range of mining rate of advance, the increase in the rate of advance can effectively relieve the subsidence of the overburden strata and reduce the support resistance.

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Gateroads Layout and Entry Support in a Western Longwall Coal Mine in China

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ABSTRACT

Comparing to the coal seams in Eastern China, the coal seams in Western China usually have better geological conditions. Therefore, changes in the conventional gateroads layout are required for the Western longwalls. A case study based on a Western coal mine (LiMin) was presented to discuss the feasibility of an innovative gateroads layout. Based on the geological conditions, a 5m wide pillar between the adjacent two longwall panels, 09112 and 09113, was proposed in order to increase the recovery rate. More importantly, the entry supporting strategy of panel 09113 headgate needed to be determined and verified carefully since only 5m wide pillar was left to separate the entry from the panel 09112 gob. Three strategies with different supporting intensities in the panel 09113 headgate were evaluated by numerical modeling. The model outputs indicated that roof and rib boltings plus single hydraulic props were essential to maintain the gateroads stable. Field observation showed that under the proposed entry supporting strategy, the total roof-to-floor and rib-to-rib convergences were only 360mm and 460mm, respectively, during the service life of the panel 09113 headgate, which indicated that the innovative gateroad layout proposed in this paper is suitable for longwalls with similar geological conditions in Western China.

INTRODUCTION

Since the past decade, the extraction of coal seams in the eastern China has become more and more difficult and many safety hazards have been reported because many longwall panels extracted the bump-prone, deep and highly gassy seams (Qian and Shi, 2003).

However, comparing to the eastern coal seams, the western coal seams usually have a simpler geological condition (Qian and Shi, 2003), such as less overburden depth, less in-situ stresses, less methane content, and more uniform thicknesses of coal seams. Due to these advantages, changes in gateroads layout in Western longwalls are required to make the extraction of the reserves more efficient and economic. In this paper, the design and verification of gateroads layout and the roof and rib supports in a Western coal mine (LiMin) were discussed.

GEOLOGICAL AND MINING CONDITIONS

LiMin coal mine is located in WuHai, the Western Inner Mongolia. The 091 District of LiMin coal mine extracted the No.9 coal seam, which was flat and 3m thick. Geological survey showed that the No. 9 coal seam was uniform and its average overburden depth was 300m. According to the core logs of exploration in this area as shown in Table 1 the rock layers above the coal seam consisted of shale, sandy shale, fine sandstone, and medium sandstone in the ascending order, while the floor layers below the coal seam consisted of shale and sandy shale in the descending order. During the development of this coal mine, the government evaluated the methane content of No.9 coal seam and concluded that it was the mildly gassy seam in accordance with the regulations (Lan and Bai, 2010).

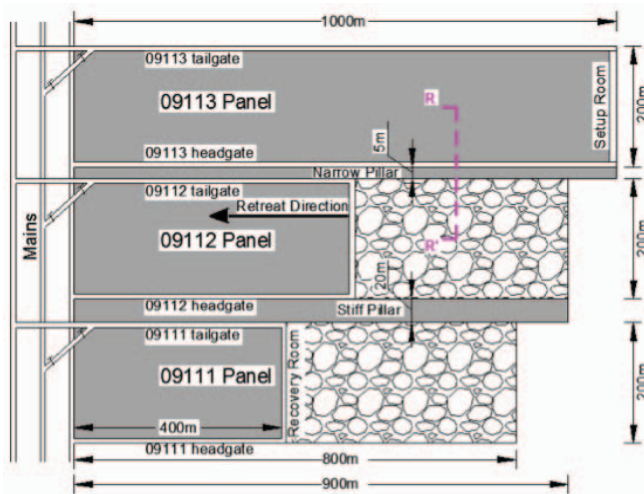
The panel layout in the 091 District is shown in Figure 1. The sizes of panels 09111, 09112, and 09113 were 800m×200m, 900m×200m, and 1000m×200m, respectively. All of them employed the single-entry system since the geological conditions was as mentioned earlier very simple. The cross-sections of all the gateroads were 4.5m wide by 3m high. The headgate of panel 09112 was developed during the retreat of panel 09111. In order to cope with the large abutment pressure from the panel 09111 gob and make the development of panel 09112 headgate safe and successful, a stiff pillar of 20m wide between panels 09111 and 09112 was selected as the preliminary design. In addition, a bolting pattern of 1m×1m was employed for the roof and ribs in the panel 09112 headgate.

It must be noted that panel 09111 was stopped 400m in by the right edge of the mains to avoid damage to the surface structures.

After panel 09111 was mined out, a field observation in the panel 09112 headgate was conducted. The results showed that the 20m wide stiff pillar was sufficient because only minor deformation occurred in the panel 09112 headgate. However, management was concerned with the large pillar that reduced coal recovery. After a lengthy debate on the proper barrier pillar width between panels 09112 and 09113, a 5m wide narrow pillar was proposed as shown in Figure 1. Hence the heart of this new design was how to keep the panel 09113 headgate stable during the retreat of panel 09112.

Table 1. Stratigraphic column for the No.9 coal seam.

	Rock Strata	Overburden Depth, m	Thickness, m
Roof Fine sandstone Sandy shale Shale	Medium sandstone	279.68	9.72
	289.4	1.69	
	291.09	3.60	
	294.69	5.31	
Coal Seam	#9 seam	300	3.00
Floor Sandy shale	Shale	303.00	0.33
	303.33	15.3	

**Figure 1. Panel layout in 091 district.**

ENTRY SUPPORT

When longwall mining in panel 09112 is in progress, the abutment pressures occur around the edges of the gob. Normally, the magnitudes of the maximum side abutment pressure may reach up to 3.5 times of the overburden stress (Qian and Shi, 2003; Peng, 2008; Morsy and Peng, 2002; Morsy and Peng, 2006). Hence, it may endanger the stability of panel 09113 headgate since the pillar was only 5m pillar wide. Based on the geological conditions and mining activities, three roof and rib supporting strategies were proposed as shown in Table 2.

Table 2. Three strategies of the entry support.

	Roof/rib bolting pattern	Roof cable bolting pattern	Hydraulic props
Strategy I	1.0×1.0m	1.5×2.4m	No
Strategy II	0.8×0.8m	1.5×2.4m	No
Strategy III	0.8×0.8m	1.5×2.4m	1.5×1.5m pattern

The resin-grouted tensioned rebar bolts were used in all the three strategies. The length of resin between the rebar and the wall of the bolt holes was 1.2m. At least three metric tons of tension was required during installation and an anti-friction washer between the bearing plate and the tension nut was used to make the installation more efficient. The thickness of the bearing plate was 10mm. The diameter and the length of the rebar bolts were 22mm and 2.4m, respectively. On the other hand, the cable bolts installed in the roof consisted of a 7-strand wire made of one straight central king wire surrounded by 6 outer wires wrapping around the king wire. The installed tension of the cable bolts was 10-12 metric tons. The diameter and the length of the cable bolts were 17.8mm and 7.2m, respectively.

In strategy I, the roof/rib bolts and roof cable bolts are installed in the entry after development following the pattern as shown in Table 2. In strategy II the supporting density is higher with more roof/rib bolts. While in strategy III it combines the roof/rib bolting patterns of strategy II with the hydraulic props. The setting pressure for the hydraulic props was 20MPa (16 metric tons) with the installation patterns for props shown in Table 2.

A 2D numerical model employed FLAC^{3D} subjected to plane-strain condition was generated to simulate a thin slice at cross section R-R' in Figure 1. The 2D model consisted of one half each of panels 09113 and 09112 and the gateroad system between them. The dimensions of the global model were 205m×1.0m×138.95m as shown in Fig. 2. Since the cover depth between the surface and the top of the global model was 229.68m, the vertical stress of 5.75MPa was applied to the top of the model, while the horizontal stress was assumed to be 1.2 times of the vertical stress. The Mohr-Coulomb criterion was used to simulate the rock strata except for the gob, which was assumed to behave as a strain hardening material and calibrated as the double-yield constitutive model in FLAC^{3D} following the Salamon's theoretical gob-model (Salamon, 1990; Pappas and Mark, 1993; Peng, 2008). In addition, a portion around the panel 09113 tailgate in the global model was highlighted as the sub-model (Fig. 2) to facilitate more in-depth analysis.

There is a practical trade-off between getting a realistic stress distribution and a realistic convergences in the numerical models related to ground control issues (Heasley, 2008; Heasley and Su, 2006). In general, it is not possible to accurately model both stress and displacement with the same set of material parameters. If the user optimizes the input parameters to produce realistic

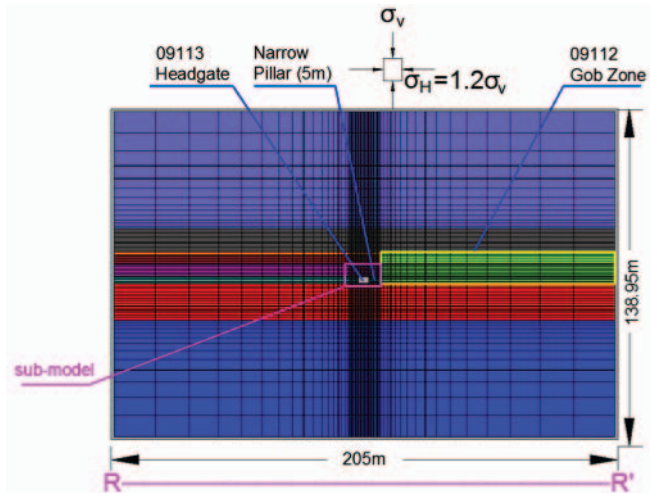


Figure 2. Global model for evaluation of the three stages of entry support.

displacement values, then the calculated stress values will be inaccurate, vice versa. So this would be a case-dependent choice that the user needs to make. On the other hand, a lot of narrow pillar application in Eastern China (Chen and Lu, 1994; Hou et al., 1995; Jiang et al., 2005) indicated that the narrow pillar could be used successfully in worse geological and mining conditions than those in this study. The only problem is how much entry convergence is allowed for the specific entry of interest. In this case, a better understanding of entry displacement is the primary objective. Hence, the inputs need to be calibrated with field observation of entry convergence during the retreat mining of previous longwall panels. Fortunately, the authors have the data from previous research (Li et al., 2010) that can provide proper inputs for the rock strata and coal seams (Table 3) in the global model.

Table 3. Rock/Coal Properties for the numerical model.

Rock Strata	Thickness, m	Density, Kg/m ³	Shear Modulus, GPa	Bulk Modulus, GPa	Friction Angle, degree	Cohesion, MPa	Tensile Strength, MPa
Top Rock Layer	50	2600	9.31	1.1	42	7.1	1.43
Medium Sandstone	9.72	2600	8.47	1.71	41	5.1	0.85
Fine sandstone	1.69	2600	8.67	1.68	41	5.4	1.16
Sandy shale	3.6	2600	8.26	1.83	41	4.3	0.73
Shale	5.31	2500	7.62	1.97	38	3.5	0.65
Coal	3.00	1400	1.22	2.59	34	2.51	0.59
Shale	0.33	2500	7.62	1.97	38	3.5	0.65
Sandy shale	15.3	2600	8.26	1.83	41	4.3	0.73
Bottom Rock Layer	50	2600	9.31	1.1	42	7.1	1.43

The gob stiffness is another key factor which may affect the abutment pressure significantly. A previous study showed that it is adequate to use FLAC^{3D} double-yield model to simulate the compaction of the gob material (Peng 2008). Based on the Salamon's theoretical model (Salamon, 1990), the numerical gob-model was calibrated and the input parameters for the double-yield model are shown in Tables 4 and 5.

RESULTS OF MODELING

The three strategies of entry supports were performed in the global-model separately. Figure 3 shows the contour of the grid displacements of the three strategies in the sub-model after panel 09112 retreat had completed. For the detailed analysis of the numerical models, Figure 3a is used to illustrate the model outputs as an example when the supporting strategy I was performed: the short red lines represent the roof/rib bolts, while the longer red lines in the roof denote the roof cable bolts; the color column on the right shows the intervals of the magnitude of displacements. In addition, the three black cylinders standing in the entry in Fig. 3c represent the hydraulic props in strategy III.

When using strategy I to design the entry support (Fig. 3a), the roofline in the left half will sag down by 250 to 300mm, while that in the other half will be 300 to 350mm; the maximum displacements in the pillar rib is 480mm and the whole floor heaves by 50 to 100mm.

On the other hand, the use of supporting strategy II may keep all the roofline sag less than 300mm (Fig. 3b), which indicates it can control the roof more reliably, comparing to the use of strategy I. Unfortunately, with respect to pillar rib displacement and floor heave, there is only a small observable difference.

However, there is a significantly decrease in entry convergences when supporting strategy III is employed according to Fig. 3c. The roofline sag, maximum displacements of pillar rib, and the floor heave are 200 to 250mm, 384mm, and 50 to 100mm, respectively.

Therefore, it is quite obvious that the use of supporting strategy III can reduce the entry convergences considerably, comparing to the use of the other two strategies. Meanwhile, it must be

noted that the headgate of panel 09113 must remain stable by artificial controls due to the 5m wide barrier pillar, and the convergences in the headgate of panel 09113 must be as small as possible throughout the active mining period since plenty of large equipment will be moving in it as panel 09113 is retreating. In this respect, strategy III will be the primary option to support the panel 09113 headgate.

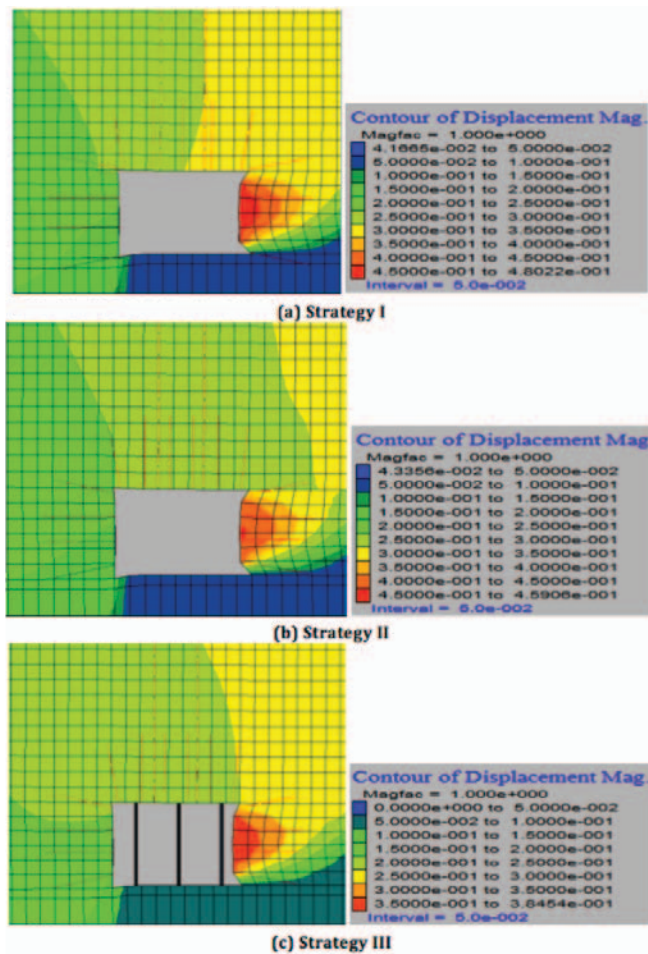


Figure 3. Grid displacements under the three stages in the sub-model.

Table 4. Material parameters in the numerical gob-model.

Property	Density, kg/m ³	Bulk modulus, GPa	Shear modulus, GPa	Friction angle, degree	Dilation angle, degree
Value	1000	8.69	6.35	20	5

Table 5. “Cap Pressure” in the numerical gob-model.

Strain, mm/mm	0.0	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.1
Stress, MPa	0	0.13	0.27	0.43	0.61	0.81	1.03	1.27	1.56	1.89	2.28
Strain, mm/mm	0.11	0.12	0.13	0.14	0.15	0.16	0.17	0.18	0.19	0.2	0.21
Stress, MPa	2.73	3.28	3.94	4.77	5.84	7.25	9.23	12.08	17.05	26.67	54.45

Nevertheless, the hydraulic props at the entry center need to be removed before the retreat of panel 09113 to make room for the large equipment.

FIELD MEASUREMENTS

As discussed previously, the strategy III was adopted to support panel 09113 headgate. The roof-to-floor and rib-to-rib convergences were monitored during the retreat of panels 09112 and 09113. Their average values are shown in Figs. 4 and 5.

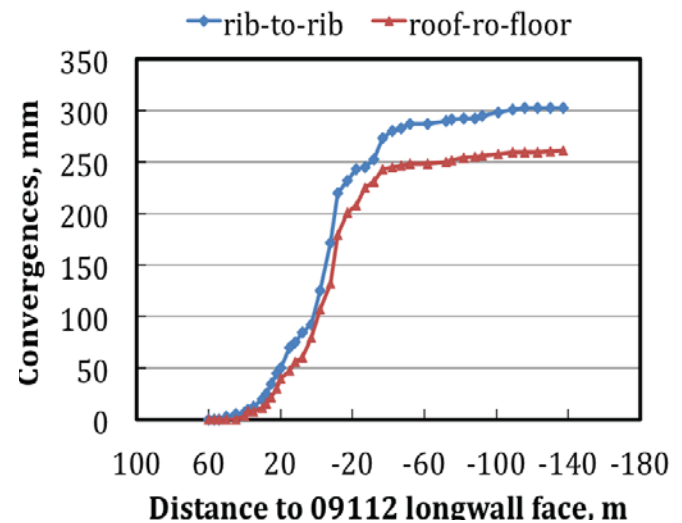


Figure 4. Convergence of the 09113 headgate during the retreat of the 09112 panel.

According to the field measurements, the maximum roof-to-floor and rib-to-rib convergences in panel 09113 headgate during the retreat of panel 09112 were 260mm and 300mm, respectively. And more than 90% of the entry convergences occurred in the area 40m outby and 60m inby the longwall face.

Please note that the model results stated above show the entry convergences located in the cross-section R-R' when the strategy III is used (Fig. 3c), while Fig. 4 shows the average value of the measured entry convergences along the mining direction. The discrepancy between the model results and the field observation may be attributed to change in geology along the mining direction.

During the retreat of panel 09113, field observations showed that entry convergences started when the longwall face was 60m inby. The roof-to-floor and rib-to-rib convergences were 100mm and 160mm, respectively, when the longwall face passed. Note that Fig 5 shows only the entry convergences occurred during the retreat of panel 09113 longwall face.

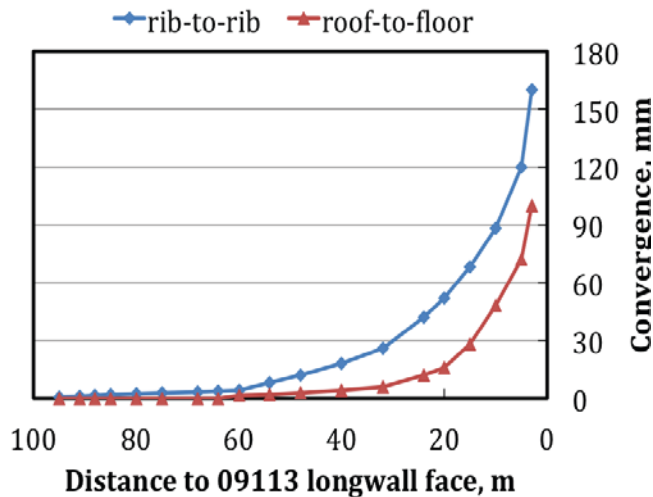


Figure 5. Panel 09112 headgate convergences during retreat of panel 09113.,

CONCLUSIONS

The coal seams in the Western China usually have a better geological condition, as compared to those in the Eastern China. Hence, an innovative gateroads layout of longwall in the Western China is required to make its extraction more efficient and economic. Innovation refers to the new designs of pillar width and roof and rib supports that are different from those used in the eastern coalfield of China.

A case study was presented in this paper as an example of the evolution of the gateroad layout. The proposed innovative changes were evaluated by numerical modeling and then implemented in the field. Field observation showed that the total roof-to-floor and rib-to-rib convergences were only 360mm and 460mm, respectively.

In addition, the proposed designs provides an alternative gateroad layout for the longwall panels in Western China that can greatly increase the recovery rate.

ACKNOWLEDGEMENTS

This research was sponsored by the National Natural Science Foundation of China through Contract No. 51174195. Also, the authors would like to express their sincere appreciation and gratitude to Dr. Syd Peng for his review of this paper.

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