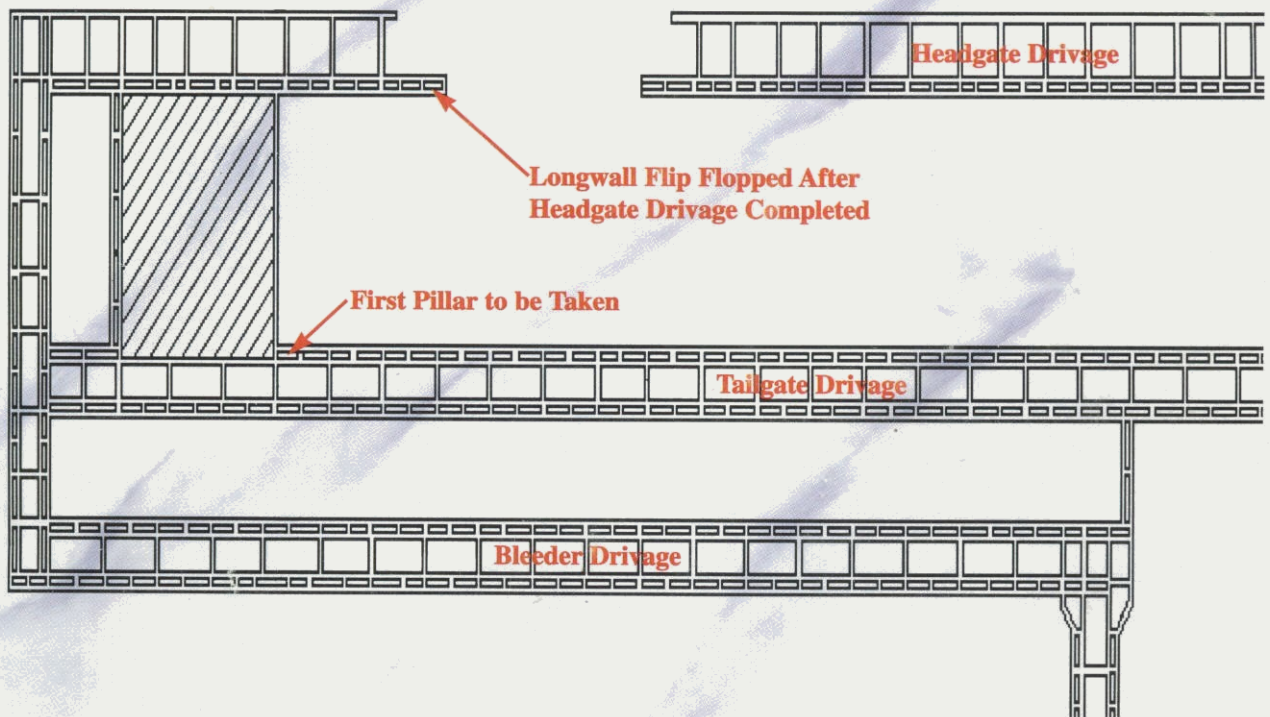




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Gateroad Pillar Extraction Experience at Jim Walter Resources

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ABSTRACT

Jim Walter Resources, Inc. (JWR), has successfully longwall mined for many years at depths ranging from 1200'-2500'. However, full pillar extraction has proven difficult and generally economically unfeasible. Overburden and redistributed stresses at these depths require relatively large pillars, which become unmanageable with most pillar extraction practices. The economic benefits of taking gateroad pillars as part of a longwall face have been recognized for some time, including increased life of mine recovery, elimination of belt moves, reduced overcast requirements, and improved longwall to continuous miner ratios. Until recently, the associated risks of pulling pillars prevented any extensive use of this concept at JWR.

Due to geological, economic, and longwall repeatability problems, JWR accepted the risks, and undertook pillar extractions with longwall faces in several different applications. The applications included taking a stable pillar (250' wide) as a longwall panel, taking yield pillars on the headgate and tailgate of panels, taking yield pillars and stable pillars in the middle of a panel, mining through chain pillars in the middle of a panel, and taking a support pillar on the tailgate end of a panel. This paper describes the background and experience of pillar extraction with longwalls at JWR.

INTRODUCTION

Jim Walter Resources, Inc. (JWR), operates seven longwalls at four different mines in the Warrior Coal Basin in central Alabama. These mines range in depth from

1200' to 2500'. Longwall mining is the primary means of extraction, with development entries utilizing continuous mining machines. The Blue Creek Seam is considered gassy, requiring multiple entries for mains and development drivage.

The geologic column above the Blue Creek Seam consists of the middleman rock (2'-8' of sandy shale), the Mary Lee Coal Seam (1.5'-2'), and the main roof (20'-60' of shaley sandstone). Two distinct mining conditions are encountered at JWR and are differentiated by the thickness and competence of the middleman. Where the middleman is relatively thick (greater than 3') and competent, only the Blue Creek Seam is taken (single seam mining). Where the middleman is thin or incompetent the middleman, Blue Creek and Mary Lee Seam are taken (twin seam mining). Twin seam mining generally results in a strong and competent roof compared to single seaming.

The primary pillar layout for longwall development is a four entry yield-stable-yield design. The yield pillar is 40'-50' center to center. The stable pillar ranges in width from 120' on single use gate roads to as much as 300' in extreme conditions. The normal stable pillar width is 200'x300', center to center. Until recently the development belt was installed as the section advanced in either #2 or #3 entry. After completing the development, the section belt was taken out and a longwall belt installed in one of the outside (#1 or #4) entries. This required two belt installations, two header constructions, and multiple overcasts at the mouth of the section. If the section belt could be used as the longwall belt, significant savings could be realized.

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Two alternatives were available to JWR for a single belt concept; converting drivages to three entries, or taking yield pillars on the headgate side as part of the longwall face. Three entry drivages are being used in some locations, but ventilation requirements do not allow it in some conditions. JWR recognized the advantages of taking the yield pillar, but until recently considered the risks too great. Several factors lead to experimenting with taking pillars including extensive underground monitoring of the yield-stable-yield pillar system, and most prominently, the possibility of extended longwall repeatability problems.

Two distinct concepts evolved at JWR regarding taking pillars, each with its own advantages and disadvantages. The first involved taking yield pillars and the second taking support, or chain pillars. Several unique situations presented the opportunity to gain experience and expand these ideas, some with great success and others with great difficulty.

TAKING YIELD PILLARS

Utilizing yield pillars on gateroad developments has been very successful at JWR (1). However, the concept of taking yield pillars with the face was initially met with resistance. Starting in 1992 an in depth rock mechanics study of the yield pillar/stable pillar system was conducted at JWR No. 7 Mine in conjunction with the Rock Mechanics Branch of British Coal (2). This extensive study provided key information about the reaction of the yield pillar and gave JWR the confidence to pursue extraction of yield pillars. The British Coal study showed that:

- 1) JWR yield pillars yield on development.
- 2) As the longwall retreated there was no additional loading on the yield pillar.
- 3) The stress in the spanning roof over the yield pillar decreased as the longwall passed.
- 4) The main roof formed a spanning beam across the yield pillar and distributed the longwall loads uniformly across the stable pillar.

From this information and the experience gained over 12 years of yield pillar use, it was believed that the yield pillar could be extracted with no adverse affect on ground conditions. As the concept was discussed and more confidence in the potential of the system was gained, two situations occurred at No. 7 Mine which lead to its implementation. These situations developed almost simultaneously so that two areas of yield pillars were designed for extraction before the first experience was gained.

Longwall 7-1

The initial experience involved a first panel in the south west area of No. 7 Mine on an 850' wide face in single seam mining (Figure 1). The tailgate drive was significantly ahead of the headgate, and it became obvious quite early in development that the headgate would not be completed in time for longwall continuity. A third section developed the bleeders and approximately 1500' of the headgate from the start line. The options at that time were to delay longwall start up, or start the face prior to connection of the headgate utilizing the existing drive. This would require using the intended tailgate as the headgate and ventilating the longwall around the 1500' developed on the headgate side.

Much discussion was given to the complications of using the tailgate as a headgate. Wrong handed equipment, belting requirements, and ventilation were the primary concerns. It was finally agreed to utilize the existing section belt for the longwall, taking the adjacent yield pillar. The longwall would retreat until the headgate was completed, then flip flop the face to the original design. It was also decided to reduce the risk by dropping the No. 4 entry in the tailgate drive immediately, reducing the number of yield pillars to be taken. This allowed the first 900' of panel to be longwall mined before the first yield pillar was extracted.

Standing support in the face entry and belt entry was initially substantial. A double row of pre-stressed Propsetters on 5' centers was set in the face entry, a row of timbers was set along the walkway side of the belt, a minimum of 5 Propsetters were set in the yield pillar cross cuts, and 4 cribs were set in the four way stable pillar junctions.

MSHA, UMWA, and JWR personnel were present as the face approached the yield pillar area. The Propsetters in the first intersection and in the face entry were closely examined during this phase. Some weight was seen on the Propsetters prior to the longwall approaching the area. No additional weight was seen as the face cut into and through the first intersection and yield pillar. The results were so encouraging the support density in the face entry was reduced. It was determined that the double row of Propsetters in the face entry could be safely reduced to a single row on 10' centers. Six yield pillars were successfully extracted before the face was flip flopped to the original design. The experience was considered successful and much confidence was gained toward taking pillars.

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Longwall 7-2

The second experience involved 22 yield pillars in the east of No. 7 Mine on longwall 7-2 (Figure 2). This panel was 850' wide and single seam. The initial headgate development started from the bleeders and progressed approximately 3000' on a four entry yield-stable-yield system. A second section was needed to maintain longwall repeatability and was started from the mains. A management decision was made to develop the new section driving a three entry yield-stable design. This created a match up problem for the connection of the two sections. Three options were discussed:

- 1) Increase the size of the stable pillar in the new three entry system by 40'. (not desirable)
- 2) Take the first part of the panel as normal and then extend the face at the transition area. (not desirable)
- 3) Take the yield pillar on the four entry system, leaving the stable pillar the original size and eliminating the extension of the face at mid-panel.

Option three was chosen.

Standing support for this area was begun prior to the experience on 7-1 and was initially substantial. A double row of fiber cribs in the face entry and a double row of wood cribs in the tailgate entry was installed. Normal standing support for tailgates include a double row of wood cribs on 7 1/2' centers. As experience was gained on the headgate pillar on 7-1, it was realized that the support on 7-2 tailgate could probably be reduced. The final support included a single row of Propsetters on 5' centers in the face entry and a double row of wood cribs in the tailgate entry. No adverse conditions were encountered and the experience was considered successful.

This situation presented two different conditions for tailgate loading. The first 1100' of the panel did not have an adjacent longwall, giving a first face stress condition in the tailgate. The remainder of the panel had an adjacent panel and the tailgate experienced the loading conditions of a second face. There was no observable difference in conditions from one area to the other.

No. 5 Mine West Barrier

The third experience taking yield pillars occurred at No. 5 Mine where a main line barrier pillar was to be extracted (Figure 3). The panel was 350' wide and single seam. The proposed headgate for this panel was originally

driven on a 6 entry yield-yield-stable-yield-yield system. It was desirable in this situation to set up the belt in the #2 entry to eliminate cross belts. This required pulling a yield pillar adjacent to a yield pillar, giving JWR another step forward in pillar extraction experience.

Based on the experience gained at No. 7 Mine, resupport at No. 5 Mine was reduced significantly from the initial efforts. A single row of propsetters on 7 1/2' spacing was installed in the face entry. The longwall retreated without complication with one exception. As the face approached an old major rock fall in #3 entry, the immediate headgate became unstable, requiring additional standing support. The area was secured by cribbing and the face continued to the stop line with no additional problems. The panel was considered successful.

TAKING STABLE PILLARS

The most difficult part of the yield-stable-yield development system is the continuous miner drivage of the large stable pillars. While investigating the idea of taking pillars, the concept of a face pillar was developed which would reduce the width of the support pillar. A face pillar was defined as a relatively small chain pillar developed with the intent of taking the pillar with the longwall face. The face pillar would be used in a three entry system and, by design, would be strong enough to control development and front abutment stresses, but small enough to allow for rapid section advance. By taking the support pillar with the face, the side abutment stresses would be redistributed to the next face panel. It was considered vital to gain significant CM advantages for the risk associated with the longwall phase.

Sizing the pillar took into consideration two ideas; the stress concentration along a longwall face and continuous miner advance rates. From extensive shield pressure investigations across numerous longwall faces it was determined that a protective stress shadow existed in the immediate headgate area and down the face for some distance. Pressure graphs consistently showed an area in the first 25-30 shields with lower leg pressures than the rest of the face. This area generally showed pressures approximately equal to setting pressures. It was believed that if the face pillar could be designed to stay within that stress shadow and remain stable on development, it could be successfully extracted. Given these parameters, a face pillar width of 100' was selected.

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The advantages of a face pillar design include all the advantages of taking a yield pillar, plus improving CM advance rates. Computer simulations indicated that this size face pillar would increase section advance rates by 30-50% over the standard design. A major concern of the concept is the potential for catastrophic failure as the longwall retreats into cross cuts. This potential for failure and other unknown risks of the system slowed development of the concept and hindered any serious consideration of implementing it. Once again, however, adverse circumstances lead to a proving of the system.

LW 4-2 South East Stable Pillar Panel

In the south east of No. 4 Mine, a single panel was developed with a 250' stable pillar on the tailgate and a 120' stable pillar on the headgate (Figure 4). As the start line was being driven, a small fault was encountered. Additional probes showed the fault running parallel to the gate roads for almost the entire panel length. After evaluating various methods to gain as much coal as possible from this disastrous situation, it was determined to set up a longwall the width of the stable pillar (250') on the tailgate development. Both single and twin seam mining were present in this panel.

As the face retreated a significant elevation change from headgate to tailgate was encountered. This elevation change required the headgate to remain 20'-30' ahead of the tailgate in order to keep the face moving square. It was discovered that attacking the cross cuts at an angle, with one gate ahead of the other, resulted in better conditions in the cross cuts.

Supporting the cross cuts was a major consideration as the face retreated. Resupport was altered several times with varying success and failure. After several falls, it was determined that a single row of fiber cribs on 20' center set against the outby rib line and rebolting the roof in the cross cuts was required. Once this support system was in place, the longwall retreated with minimal complications. Eighteen 250' x 300' pillars were extracted over a 6 month period. This equated to approximately 300,000 clean tons, a significant amount of coal for a longwall that otherwise would have been idle. This difficult situation confirmed the technique of successfully crossing open entries with a longwall face, which is essential for successfully extracting face pillars.

Longwall 4-2 East Barrier Pillar Panel

Even after successfully taking the stable pillars, repeatability was a problem for this longwall. It was decided to move the longwall to a barrier pillar in the east mains. The panel would be 400' wide with both single and twin seam mining. As Figure 5 shows, this panel presented numerous strata control complications, including taking a yield pillar at the headgate, taking a support pillar and yield pillar at mid-face, and slabbing a pillar at the tailgate end. The alternative was to shut the longwall down for several months while a new longwall area was developed. JWR was gaining experience rapidly in supporting and extracting pillars, but this panel would be a major step in pillar extraction experience. Once again, adversity became the mother of innovation.

Because of local ground conditions and the location of an existing belt, this panel was set up to take a yield pillar on the headgate the entire length of the panel, approximately 3400'. Approximately 600' from the start line, probe entries had been driven across the panel from headgate to tailgate, forming yield pillars which would have to be taken. 1400' from the start line, the headgate entries expanded into the barrier pillar, giving a stable pillar and another yield pillar in the middle of the face. 1400' from the start line the barrier pillar narrowed on the tailgate end, requiring the longwall to slab approximately 25' off the tailgate support pillar. Although the panel was relatively narrow (450'), there were major concerns about the roadways at mid-face.

Taking the yield pillar on the headgate proved to be no problem. A single row of Propsetters on 10'-20' centers was installed and maintained to the stop line in the face entry. Several old header holes required false roofs to keep the headgate shields in stroke. Mining through the probe entries created no problems, with only minimal additional support set. The stable pillar cross cuts at mid-face were rebolted with 6' full column bolts and 4 Propsetters were set in each intersection. As the face approached the first stable pillar cross cut the roof began to work and the coal pillars sloughed. But, as the longwall came into the cross cut the area stabilized and no problems were encountered. Slabbing the pillar at the tailgate end also proved to be no problem.

The longwall retreated at a rapid rate and it was felt that this contributed to the success of the system. The face location at weekends was reviewed each

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week and the wall was never stopped in or at the edge of a big cross cut. The longwall was either stopped at least 20' short of the cross cut or continued until the face was through the cross cut.

Prior to the face reaching the stop line, which was twin seam mined, additional support (a double row of Propsetters on 5' centers) was installed in the pull out cross cuts. Based on success to that point, it was believed to be an adequate plan. As the face cut out the last coal in the last pillar a massive roof fall occurred at mid-face, extending from the yield pillar at mid-face to within 40' of the headgate yield pillar. Extensive resupport and clean up was required, delaying the longwall for approximately one week. It was felt that the slow down of the longwall to mesh allowed the gob to ride over the shields and cause the fall in the cross cuts. In retrospect, substantial standing support was needed at the pull out point where the face retreat rate was reduced significantly. Although the face was slowed considerably due to the fall, the panel was considered successful and valuable experience was gained with a longwall that would have otherwise been idle.

Longwall 4-2 Face Pillar Panel

With the success of the stable pillar longwall, it was decided to design a face pillar for the first tailgate development in the north east area of No. 4 Mine. This was a confined area requiring as much longwall coal extraction as possible to reduce repeatability problems for 4-2. The panel was developed totally twin seam as longwall 4-2 retreated in the east barrier.

As shown in Figure 6, the longwall started out 620' wide, and then was extended by 100' to take the face pillar. The face pillar cross cuts were developed on an angle to give a better angle of attack without having to keep one gate end ahead. The panel was also designed to take a yield pillar on the headgate side, incorporating both pillar extraction concepts. Because of the problems experienced at the stop line of the 4-2 barrier panel, the face entry was dropped prior to the stop line. This required the last 900' of drivage to be on a two entry design, but eliminated an entry at the pull out position.

A single row of Propsetters on 20' centers was installed in the headgate face entry adjacent to the belt. A single row of Propsetters on 20' centers was set in

the tailgate face entry. A single row of cribs on 10' centers was installed in the tailgate entry (the normal tailgate support for this area). The longwall retreated to the stop line with no problems in the headgate or the tailgate. The panel was considered successful.

No. 5 Mine North Barrier Pillar Panels

Just prior to No. 4 Mine starting the face pillar panel, No. 5 Mine began pulling another barrier pillar (Figure 7). Because of geological problems in the western portion of the mine, this longwall had been idled until a new panel could be developed in the east. The barrier pillar was being taken to improve mine production while the east side was being developed. To pull this barrier two sets of existing roadways would have to be negotiated. The first set included an intake shaft, and the decision was made to stop short of those entries. The longwall was mined to within 20' of the first roadway and then moved around the shaft. No adverse conditions were encountered.

Approximately 700' from the second start line, four entries driven on 110' chain pillars had been developed across the barrier. These roadways were driven 16 years earlier but were in relatively good condition. This longwall was retreating at a rapid pace and it was believed that the entries could be successfully mined through, particularly since it would not have to slow down to mesh in the chain pillar area. A double row of Propsetters was installed in the entries and cross cuts. The tailgate was supported with a single row of Propsetters on 7 1/2' centers. As the longwall approached the first entry there were no signs of problems. As the longwall cut into the first entry a major roof fall occurred, creating a massive loading on the shields.

An examination of the area revealed that the roof broke and sat down as one unit from the entry to the gob, overloading the shields and creating a squeeze on the face. The failure caused extensive longwall delays (approximately one month) and brought into question the viability of mining through the remaining entries. The two options were to idle the longwall for several months while the east side was developed, or continue on through the entries. The decision was made to significantly increase standing supports in the cross cuts and continue at least through the next entry. If conditions deteriorated the longwall would be stopped at mid pillar.

The additional support was added while the longwall was recovered from the

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squeeze. At least a double row of fiber cribs was added to the double row of Propsetters. Once the longwall resumed production no additional falls or delays were encountered. Although this was a difficult situation, additional experience was gained concerning the technique of taking pillars with a longwall and the conditions in which it is a viable risk.

OBSERVATIONS FROM THE MAJOR FALLS

Many attempts have been made in the industry in recent years to mine through existing roadways. Resupport costs have varied from thousands of dollars to hundreds of thousands of dollars using various support methods including standing support, cable bolts and slings, and complete back filling. Some experiences have been successful and some not. One of the more perplexing problems encountered by JWR was determining why, in similar conditions, some roadways failed and others showed no signs of deterioration. One uncontrollable aspect of mining through entries, which may explain this variability, is the magnitude of periodic loading at the time the face approaches an open area.

If the sequencing of periodic loading hits toward the end of a cycle, the overhang would be exaggerated due to the "soft" support given by the open entries, creating a major loading situation at the worst possible time. If the periodic loading was early in the cycle, the stress on the "soft" support would not be critical and the entries would see a manageable amount of load. The major falls at 4-2 East Barrier and No. 5 Mine North Barrier both appeared to carry over into the gob, which may have been directly related to the extent of periodic loading at the time. Designing for this worst case scenario would be expensive, but must be considered and may be a necessity to ensure success when mining through open entries.

FUTURE PLANS

JWR is currently designing all developments at No. 7 and No. 4 Mines to take yield pillars on the headgate. No. 5 Mine develops three entry drivages and does not plan to take yield pillars. No. 3 Mine has the worst main roof conditions and does not consider the benefits worth the risks associated with taking pillars as a standard design. Face pillars are considered a viable option under certain conditions. No plans have been made to incorporate face pillars as a standard design but the concept will be considered and utilized where viable.

CONCLUSIONS

A wide variety of situations have been encountered by JWR while pulling pillars. This experience has lead to the following conclusions:

- 1) Yield pillars can be successfully taken in most JWR conditions. The exception is in areas where the main roof is not competent.
- 2) The ability to safely extract yield pillars has lead to design changes which have saved considerable labor, material, and construction costs by giving the flexibility to use a single belt for development and retreat.
- 3) Face pillars can be designed successfully but require better overall conditions than when taking yield pillars.
- 4) The benefits of mining through open entries must be significant to justify the risks.
- 5) Substantial standing support is required when encountering open entries in the middle of a face.
- 6) Attacking cross cuts at an angle, either by design or face management, improves roof control in most circumstances.

It is the author's conclusion that excessive loading, which may lead to catastrophic failure, is unpredictable when mining through existing roadways due to periodic weighting cycles and changing ground conditions. Design parameters must take this into consideration and a comprehensive risk assessment made to determine acceptable applications of pillar extraction with longwalls. Many of the decisions to mine through pillars at JWR were only viable because of the negative alternative of idling that longwall for extended periods.

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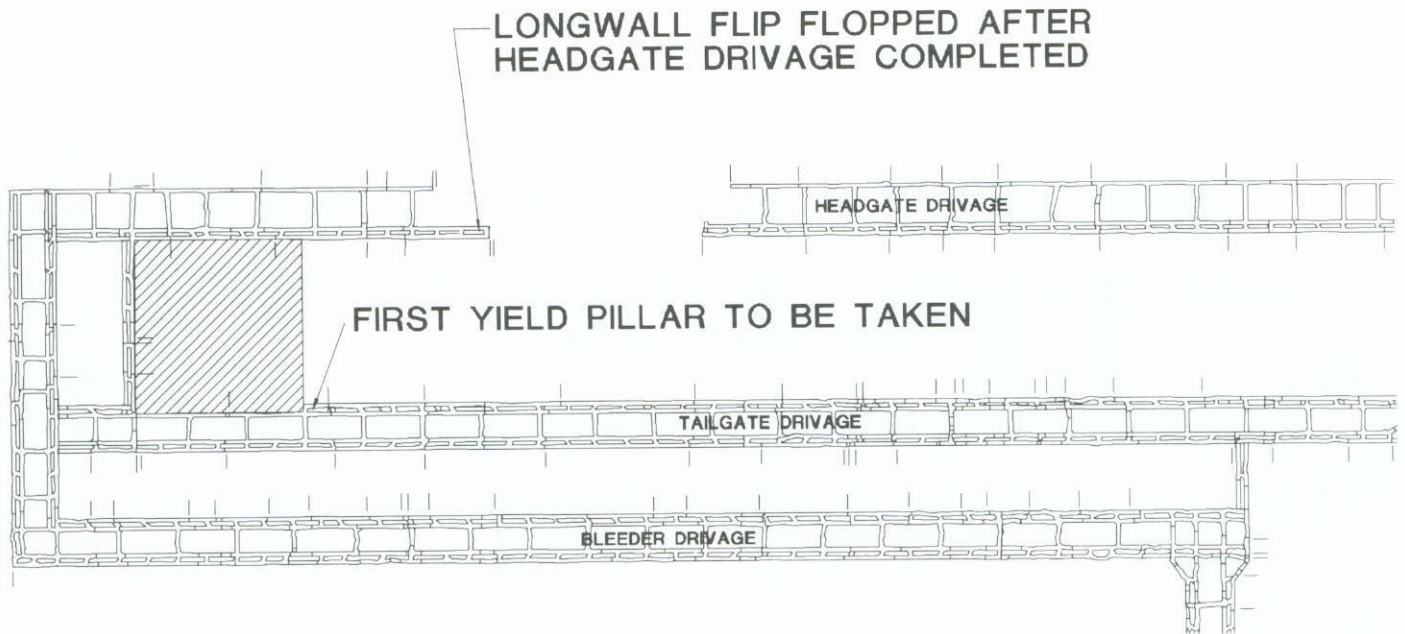


FIG. 1 LONGWALL 7-1

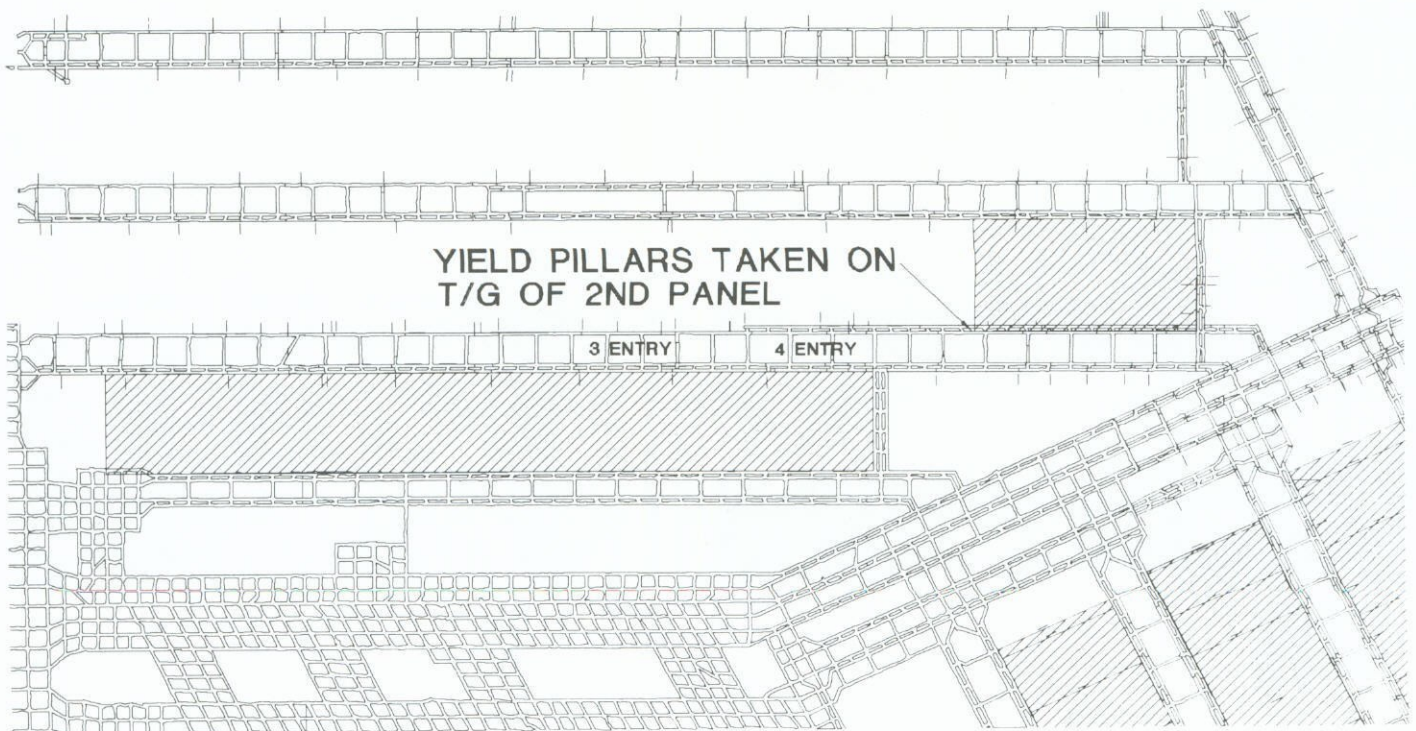


FIG. 2 LONGWALL 7-2

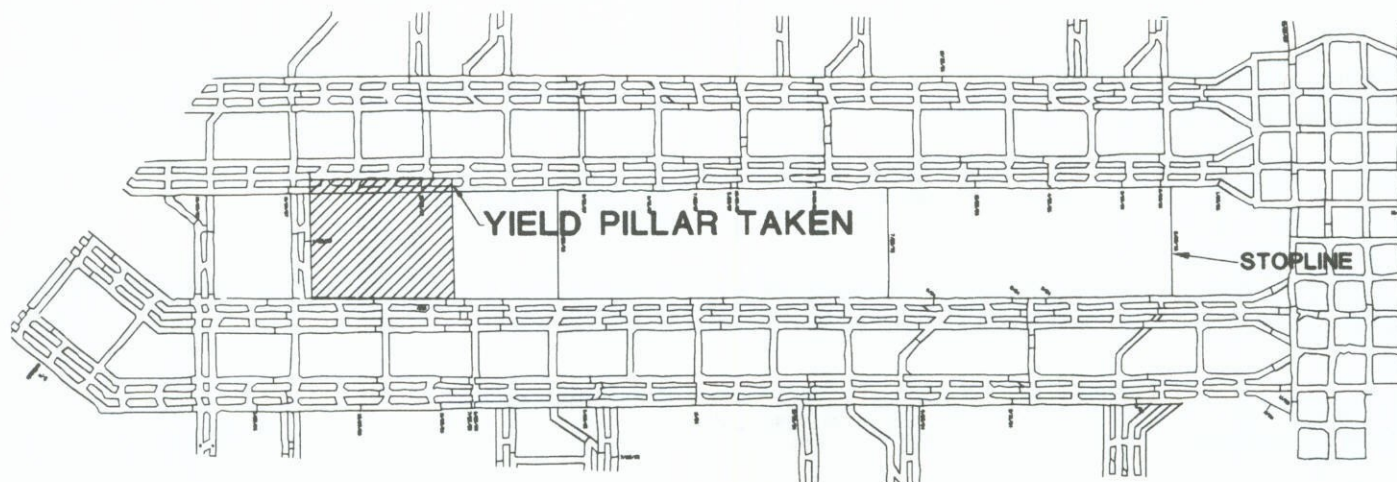


FIG. 3 NO. 5 MINE WEST BARRIER

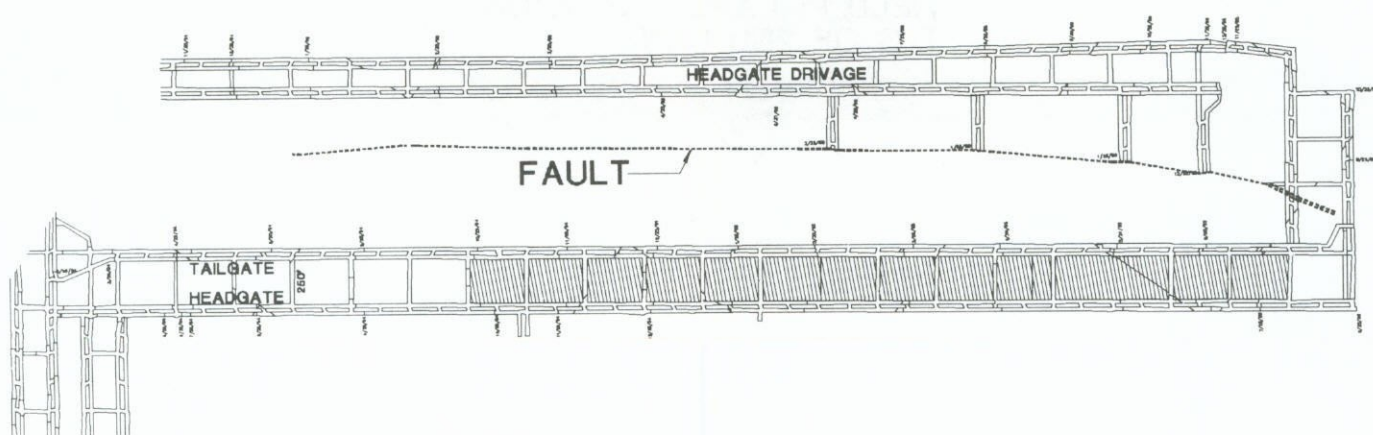


FIG. 4 LONGWALL 4-2 SOUTH EAST STABLE PILLAR

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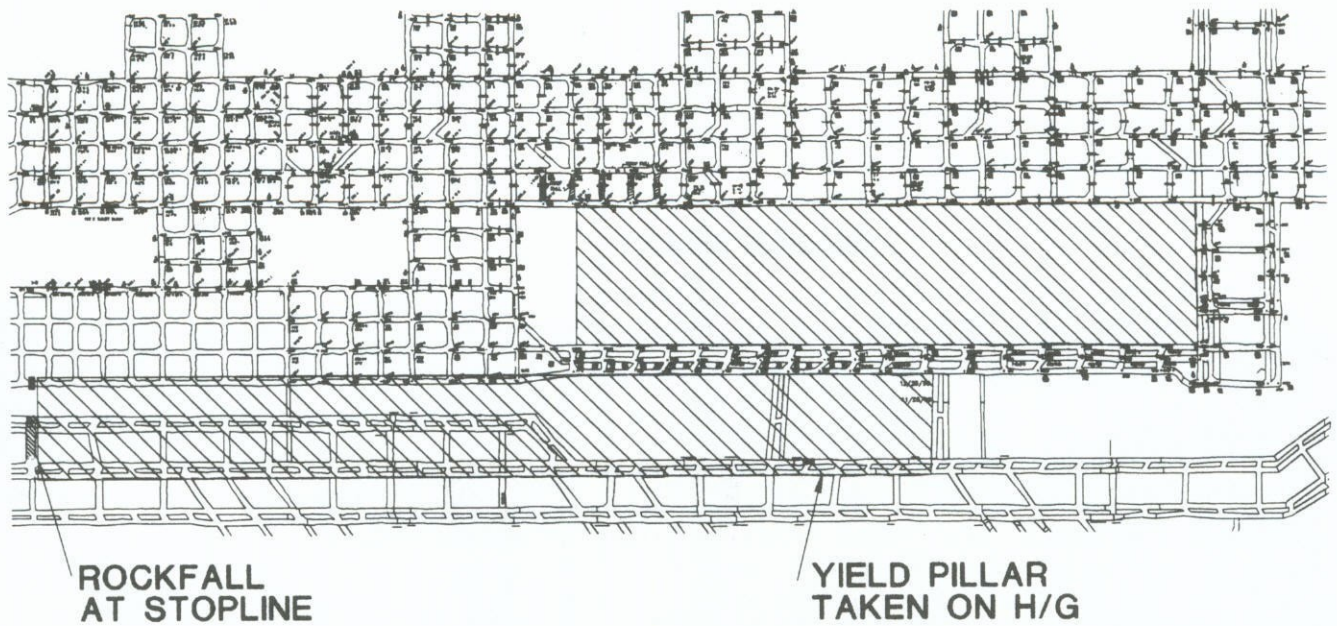


FIG. 5 LONGWALL 4-2 EAST BARRIER PILLAR

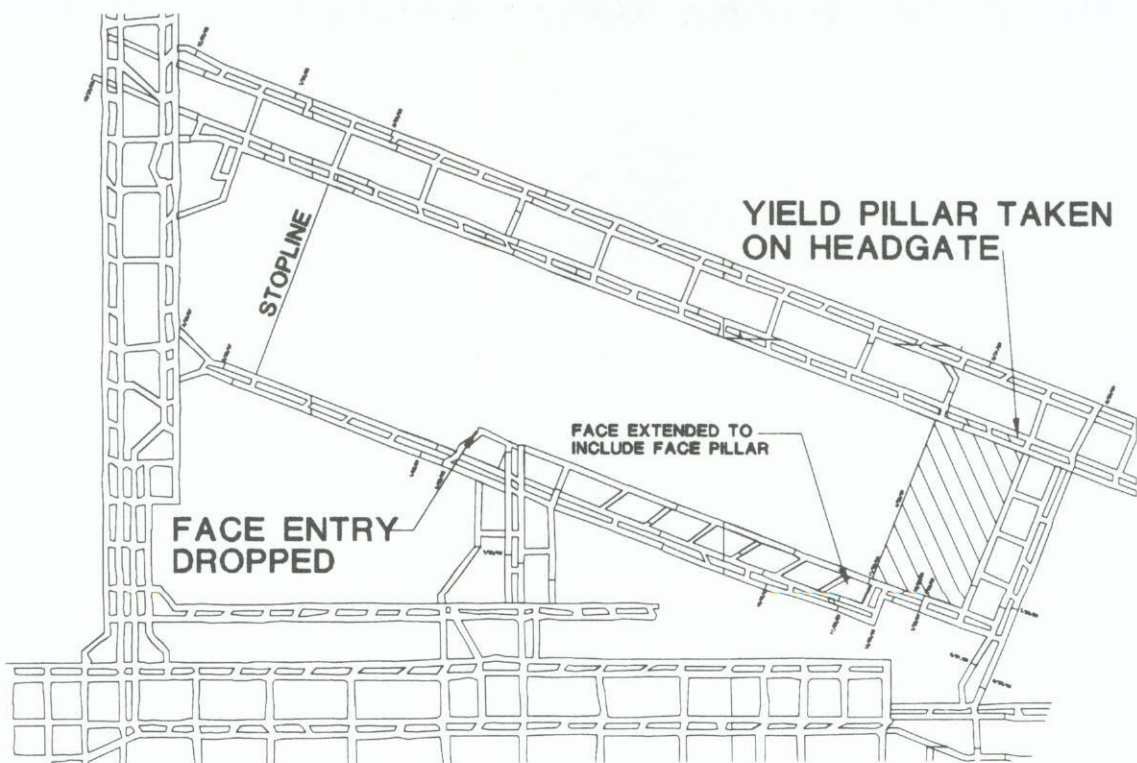


FIG. 6 LONGWALL 4-2 FACE PILLAR PANEL

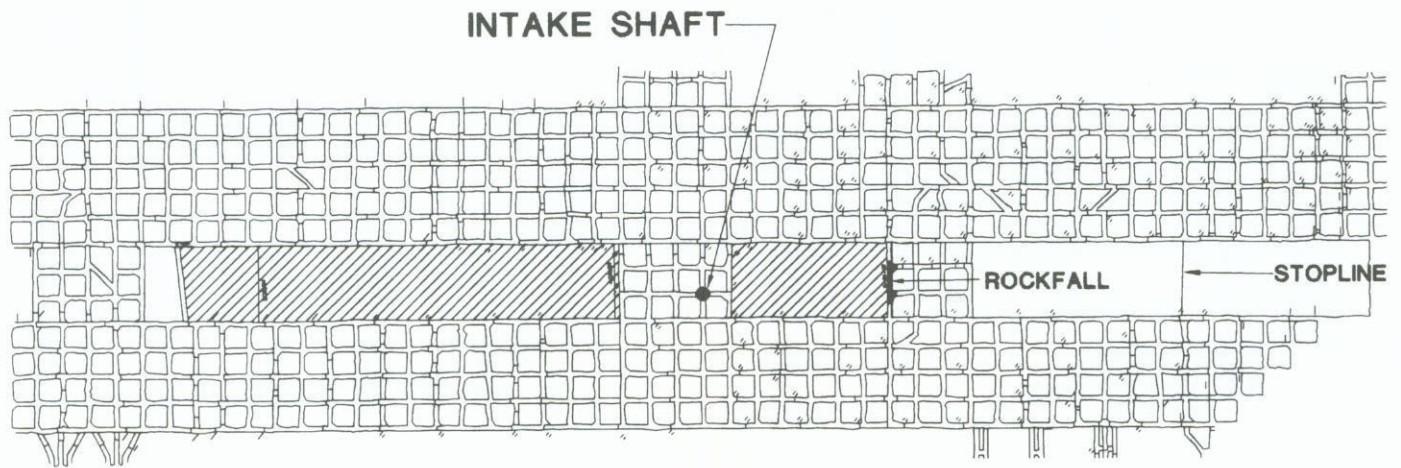


FIG. 7 NO. 5 MINE NORTH BARRIER PILLAR PANELS



Stability of Backfilled Cross-panel Entries During Longwall Mining

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ABSTRACT

In cooperation with Cyprus Twentymile Coal Co., researchers from the National Institute for Occupational Safety and Health (NIOSH), Spokane Research Laboratory, conducted a study at the Foidel Creek Mine, an underground coal mine near Oak Creek, CO, to evaluate the stability of underground working conditions as a longwall advanced through a series of backfilled cross-panel entries.

Instruments installed in the cross-panel entry pillars, backfill, longwall panel, and adjacent headgate entry were continuously monitored by a computerized data acquisition system as the longwall approached and advanced through the backfilled section of the panel. Data collected from these instruments were analyzed to determine the magnitude and direction of the secondary principal stress changes ahead of the longwall face, deformation and closure of the headgate entry, and stability of the cross-panel entry pillars and backfill. A three-dimensional, boundary element model was calibrated to accurately predict mining-induced stress changes and the distance they occurred ahead of the longwall face.

As the longwall advanced through the backfilled entries, most of the mining-induced load was supported by the cross-panel entry pillars. The backfill provided stability for the roof and floor of the in-panel entries and also confined the entry pillars significantly improving their load-carrying capacity. Comprehensive information derived from backfill material property tests, instrument data, underground observations, and numeric modeling results document the safety and stability of the 8-Right minethrough and should provide beneficial case study information for other longwall backfilling applications in the future.

INTRODUCTION

The Foidel Creek Mine (figure 1), operated by the Cyprus Twentymile Coal Co., is located near Oak Creek, CO, approximately 20 miles southwest of Steamboat Springs. The mine is currently using state-of-the-art longwall equipment to mine low-sulfur, high-quality bituminous coal from an 8.5- to 10-ft- (2.6- to 3-m-) thick coal seam at a depth of about 1,300 ft (396 m). Stable ground conditions and innovative mining equipment, including a 72-in- (183-cm-) wide underground conveyor belt, allow the mine's longwall system to advance at an average rate of 100 ft/d (30 m/d), a rate that recently set world records for longwall production. Because this high-capacity longwall system requires that continuous miners develop gateroads at an average advance rate of 500 ft/day (152 m/d) (Carter, 1996), three parallel 20-ft- (6-m-) wide entries were driven across the 8-Right panel. The panel is 815 ft (248-m) wide by 18,000 ft (5,486 m) long. The cross-panel entries provide access for development of the gateroad entries, as well as escapeways for underground miners.

To maintain the entry stability during subsequent mining of the panel and to eliminate the need for a longwall face change, Cyprus decided to fully support the in-panel entries by backfilling them with an air-entrained mixture of fly ash and cement. The mine requested personnel from the Spokane Research Laboratory of the National Institute for Occupational Safety and Health (NIOSH) to assist in monitoring and evaluating the stability of the panel as the longwall advanced through the backfilled entries.

Prior to this study, Cyprus had obtained information about the local geology (figure 2) from a drill hole in the 8-Right panel. The mine floor is interbedded sandstones, while the immediate roof consists of sandstone and shale sequences. Longwall caving depends primarily on the 24.5-ft- (7.5-m-) thick C sandstone, which forms a cantilever behind the shields 80 to 120 ft (24 to 37 m) before caving (Schissler, 1997). The Wadge coal seam is part

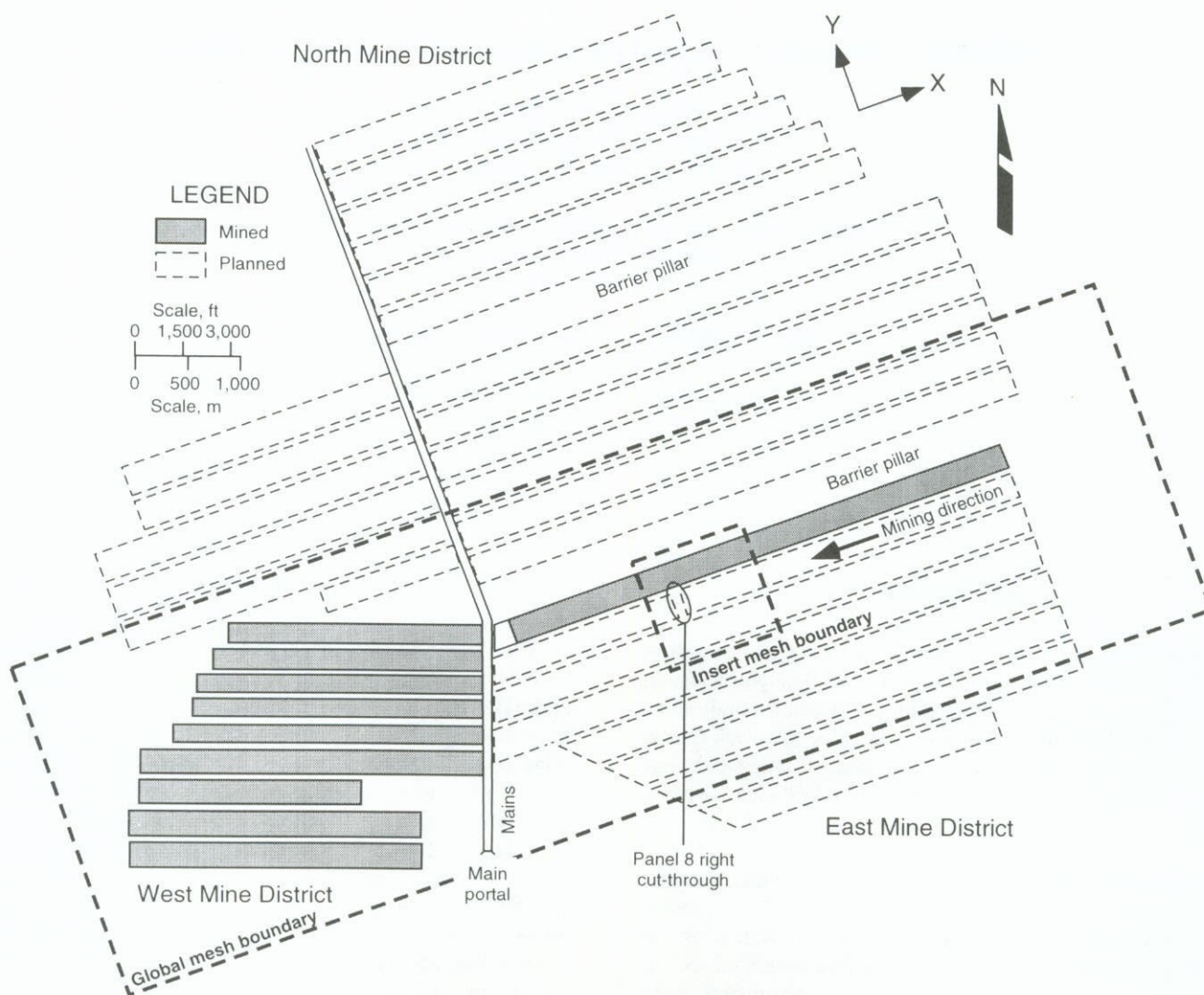


Figure 1.—Plan view of the Foidel Creek Mine.

of the Yampa Coalfield, which crops out along the Yampa River-Williams Fork Mountain area. This seam is a southeastern extension of the Sand Wash structural basin, formed of coal-bearing Upper Cretaceous, Paleocene, and Eocene rocks (Keystone Coal, 1998). The entire area is part of the Green River coal region, which extends from southwestern Wyoming into the northwest corner of Colorado.

BACKFILL PLACEMENT AND MATERIAL PROPERTIES

To maintain the stability of the cross-panel entries during minethrough, the in-panel entries were backfilled by Goodson & Associates, Inc., using a grout mix consisting of fly ash, 10 % cement, and a foaming agent. By entraining air in the mixture, the foaming agent reduced the amount of material required to fill the entries, improved the pumping characteristics of the mix, and reduced problems associated with excess bleed water underground. After bulkheads had been constructed across the

tailgate end of the cross-panel entries, the entries were sequentially backfilled over a 3-month period by pumping the grout mix from a surface batch plant down a borehole and through a pipeline set up on the headgate side of the entries. The 8-Right cross-panel entries dipped 5° below horizontal from the headgate to the tailgate, which permitted most of the length of the entries to be gravity-filled tight to the mine roof. Bulkheads were then constructed across the headgate end of the entries, and the remaining portions of the entries were pumped full of grout. The cross-panel entries appeared to be completely filled because no gaps were observed above the fill during mining.

In situ samples of the placed fill were collected in 3- by 6-in (7.6- by 15.2-cm) cylinders as the entries were grouted. The samples were tested to determine compressive strength, apparent modulus of deformation, and tensile strength after 28 and 90 days of curing. Because of inconsistencies in backfill materials, sampling procedures, and/or batching and placement processes, there was considerable variation in the density of the samples, which was reflected in the test results. The average density of the

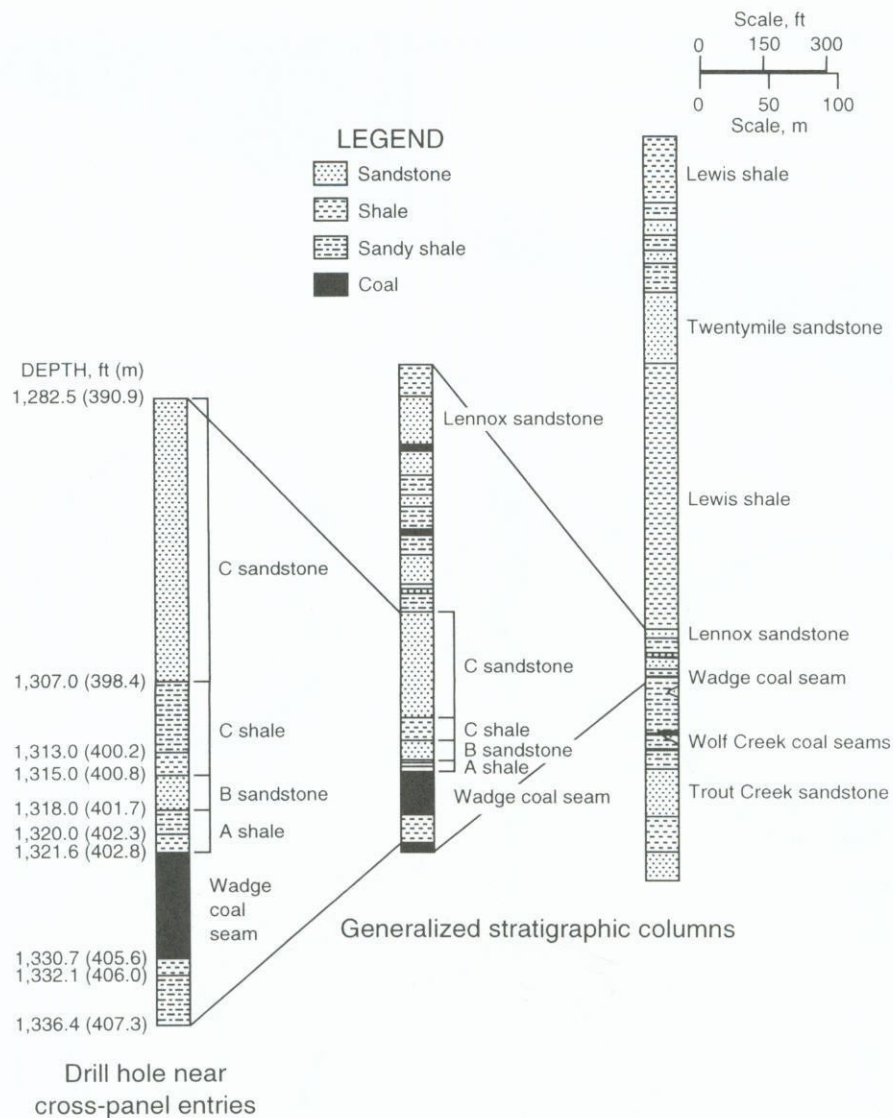


Figure 2.—Stratigraphic column

Table 1.—Summary of backfill test results.

Curing time, days	Compressive strength, psi	Modulus of deformation, psi	Tensile strength, psi
28	199	85,000	22
90	326	104,000	46

compression test specimens was 72.2 lb/ft³ (1,156 kg/m³), but the density of individual specimens ranged from 60.5 to 92.9 lb/ft³ (969 to 1,488 kg/m³). Generally, the more dense specimens exhibited substantially higher compressive and tensile strengths, as well as higher modulus of deformation values. A brief summary of the backfill test results is given in table 1.

INSTRUMENTS

To monitor stability as the longwall advanced through the backfilled in-panel entries, several kinds of instruments were installed in the cross-panel entry pillars, backfill, longwall panel,

and adjacent headgate entry (figure 3). The different types of instruments were clustered in groups to verify readings from individual instruments, to ensure that essential data would be collected if some of the instruments malfunctioned, and to provide additional information for interpreting ground behavior. Most of the instruments (table 2) were equipped with vibrating-wire transducers to minimize the effects of moisture, temperature fluctuations, and cable splices, and also to limit signal degradation resulting from long cable lengths. As the longwall approached and advanced through the backfilled section of the 8-Right panel, the electronic instruments were periodically monitored by a Campbell Scientific data acquisition system stationed in a nearby headgate entry.

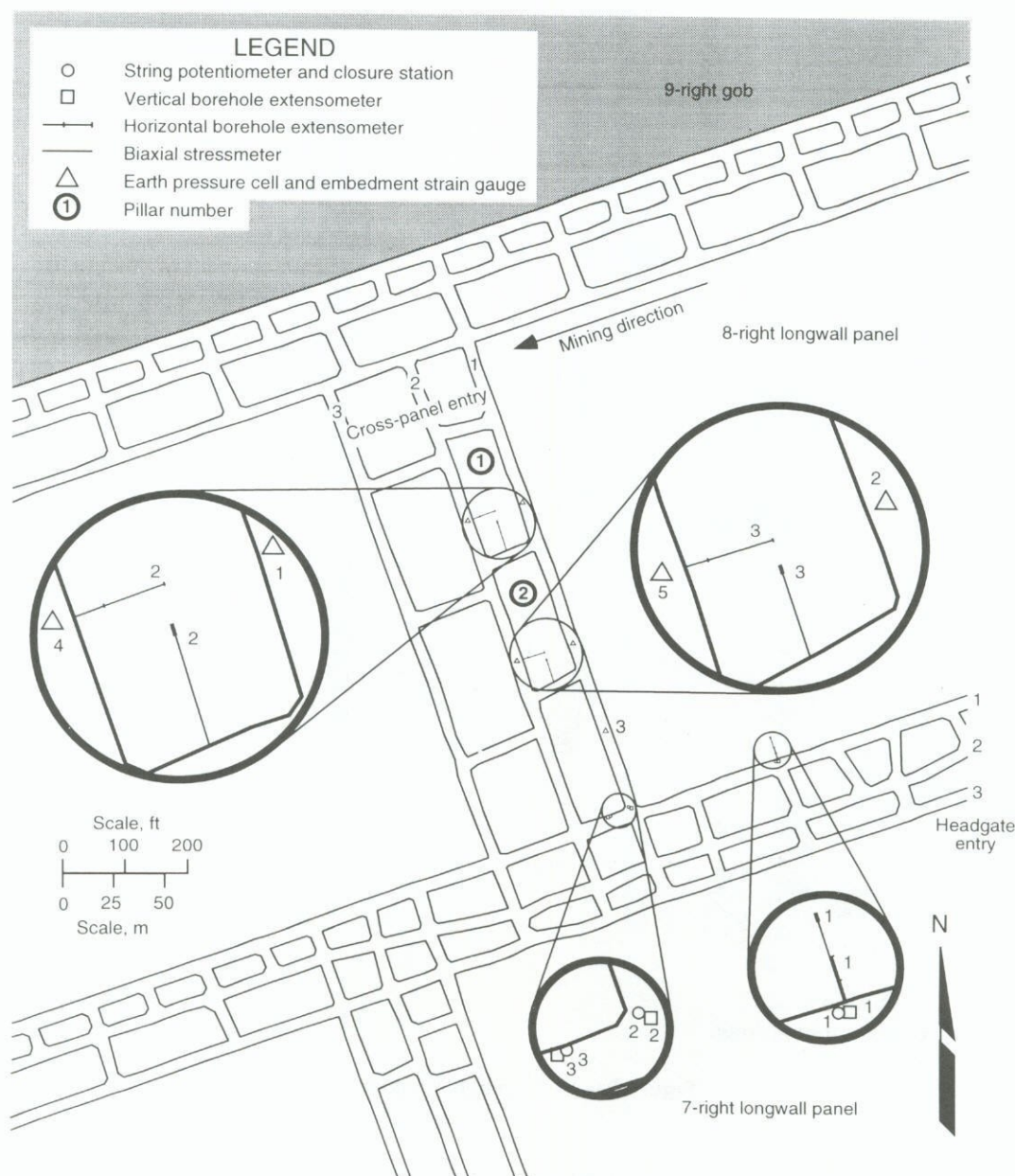


Figure 3.—Plan view of instrument locations.

STRESS REDISTRIBUTION

Longwall Panel and Cross-Panel Entry Pillars

To measure stress redistribution ahead of the advancing longwall, biaxial stressmeters were installed in the 8-Right panel as well as in two of the cross-panel entry pillars (figure 3). Biaxial stressmeters give magnitudes and orientations of the major and minor secondary principal stress changes in a plane perpendicular to the axis of a drill hole by measuring deformation of the drill hole. All stressmeters were equipped with a double set of vibrating-wire gages (two sets of three radial gages oriented 60° from one another, two longitudinal gages, and two temperature sensors) to provide additional data redundancy.

Mining-induced stress changes measured by biaxial stressmeters 1 and 2 are shown in figures 4 and 5 and table 3. Unfortunately, biaxial stressmeter 3, installed in cross-panel entry pillar 2, did not function correctly, probably because of installation problems. Theta, the angle measured clockwise from the major secondary principal stress change (p) to vertical, denotes the orientation of the stress change. For this study, a positive theta angle indicated that the stressmeters were being loaded from the direction of the main entries rather than from the direction of the advancing longwall. As the longwall approached, the orientation of the stress changes measured by biaxial stressmeter 1 was about 10° farther from vertical than the orientation measured by biaxial stressmeter 2. However, stress changes measured by both instruments became more vertical as the longwall approached.

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Table 2.—Description of instruments and location.

Instrument	Location	Anchor depth, ft		Dip, ¹ deg	Measurement	Sensor	Position
		1 point	2 point				
BSM 1	8-Right panel rib	39.5		-4.0	Panel stress change	Vibrating wire	Downhole
					Temperature	Vibrating wire	Downhole
BSM 2	Pillar 1	41.8		-5.0	Pillar stress change	Vibrating wire	Downhole
					Temperature	Vibrating wire	Downhole
BSM 3	Pillar 2	40.0		-3.5	Pillar stress change	Vibrating wire	Downhole
					Temperature	Vibrating wire	Downhole
HBX 1	8-Right panel rib	20.24	9.86	0.25	Panel rib dilation	Vibrating wire	Downhole
					Temperature	Thermistor	Panel rib
HBX 2	Pillar 1	37.79	9.88	2.0	Pillar dilation	Vibrating wire	Downhole
					Temperature	Thermistor	Pillar rib
HBX 3	Pillar 2	37.79	9.88	2.0	Pillar dilation	Vibrating wire	Downhole
					Temperature	Thermistor	Pillar rib
VBX 1-3	Headgate entry 1	16.91	4.91	90.0	Roof displacement	Vibrating wire	Downhole
					Temperature	Thermistor	Roof
SPOT 1-3	Headgate entry 1	NA		90.0	Roof-to-floor closure	Rotary potentiometer	NA
CP 1-3	Headgate entry 1	NA		90.0	Roof-to-floor closure	Manual	NA
EPC 1-3	Cross-panel entry 1	NA		0.0	Backfill vertical stress	Vibrating wire	Backfill
					Temperature	Thermistor	Backfill
EPC 4-5	Cross-panel entry 2	NA		0.0	Backfill vertical stress	Vibrating wire	Backfill
					Temperature	Thermistor	Backfill
ESG 1-3	Cross-panel entry 1	NA		90.0	Backfill vertical strain	Vibrating wire	Backfill
					Temperature	Thermistor	Backfill
ESG 4-5	Cross-panel entry 2	NA		90.0	Backfill vertical strain	Vibrating wire	Backfill
					Temperature	Thermistor	Backfill

BSM Biaxial stressmeter. HBX Two-point horizontal borehole extensometer. VBX Two-point vertical borehole gauge. SPOT String potentiometer.

CP Closure point station. EPC Earth pressure cell. ESG Embedment strain gauge.

¹The minus sign indicates negative angle from horizontal.

Table 3.—Stress change versus distance from longwall. Compressive stress is positive.

	Average principal stresses			Longwall distance, ft
	p, psi	q, psi	Theta, deg	
Initial stress change:				
Biaxial stressmeter 1	30	-57	41	991
Biaxial stressmeter 2	31	-51	30	666
Intermediate stress change:				
Biaxial stressmeter 1	131	-68	42	133
Biaxial stressmeter 2	172	-63	31	383
Maximum stress change:				
Biaxial stressmeter 1	790	174	27	7
Biaxial stressmeter 2	1,842	383	23	4
Final stress change:				
Biaxial stressmeter 1	902	258	17	4
Biaxial stressmeter 2	2,140	600	22	4

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Stress change values in table 3 were calculated for each instrument by averaging the response of the two sets of radial gages. Final readings obtained from a single set of radial gages.

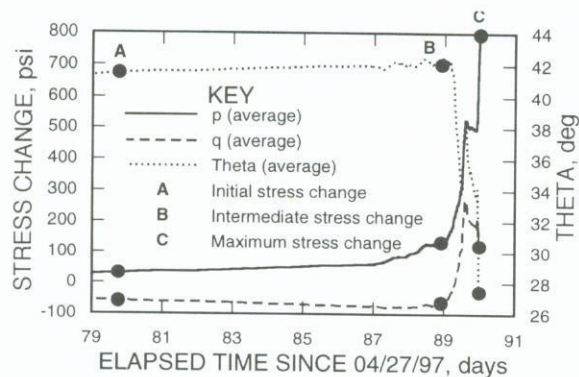


Figure 4.—Stress change in 8-Right panel

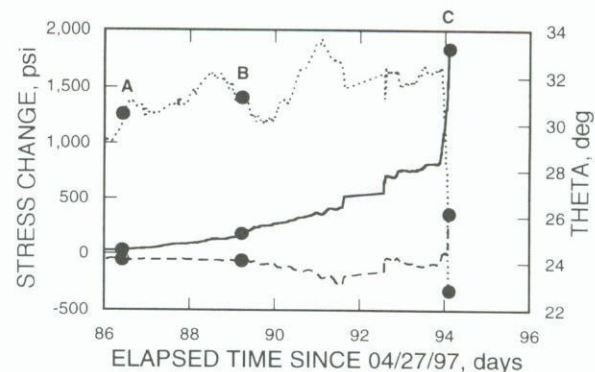


Figure 5.—Stress change in cross-panel entry pillar 1

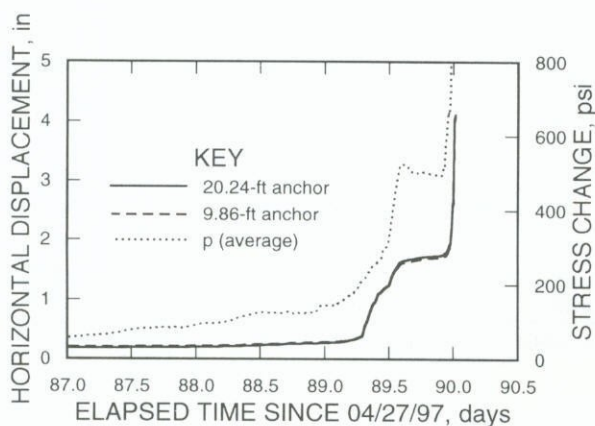


Figure 6.—Dilation and loading of 8-Right panel.

The stress increases measured by biaxial stressmeters 1 and 2 were verified by two-point horizontal extensometers installed to monitor dilation of the 8-Right panel rib and cross-panel entry pillar 1. Although the horizontal extensometers did not respond to the advancing longwall as early as the biaxial stressmeters, dilation followed the same general trends in stress changes (figures 6 and 7). Dilation measured in cross-panel entry pillar 2 was similar to dilation in entry pillar 1, but horizontal displacements were lower, indicating higher stress in the entry pillars near the tailgate.

Backfilled In-Panel Entries

While the cross-panel entries were being backfilled, earth pressure cells and embedment strain gauges (table 2) were installed at five locations to measure vertical stress and deformation induced within the fill by mining (figure 3). As a general rule, larger vertical stresses were measured in the fill near the tailgate, and much higher stresses were measured in cross-panel entry 2 than in entry 1. During the minethrough of the backfilled entries, maximum vertical stress averaged 182 psi (1.25 MPa) in cross-panel entry 1, compared to 500 psi (3.45 MPa) in entry 2. The

on each instrument indicated that the maximum stress change values for p and q were probably higher and oriented closer to vertical.

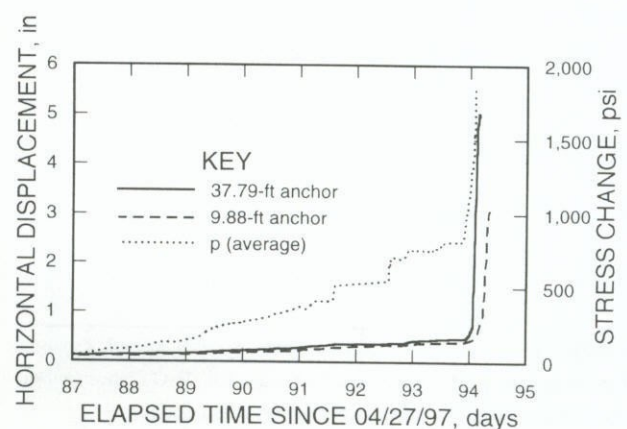


Figure 7.—Dilation and loading of cross-panel entry pillar 1

maximum vertical response of the earth pressure cells versus distance from the longwall face is shown in table 4.

As additional vertical stress was transferred to the backfilled entries during mining, the backfill strained in compression. Embedment strain gauges installed near each of the earth pressure cells measured these compressive displacements within the fill, confirming loading trends measured by the earth pressure cells. The maximum vertical strain averaged 1.1% in cross-panel entry 1 and 1.4% in entry 2, with larger vertical displacements measured toward the tailgate.

During mining, the backfill had sufficient strength and stiffness to maintain a vertical face and exhibited only minor spalling even when the backfilled entries were exposed along the entire length of the longwall face. Besides providing stability for the roof and floor of the cross-panel entries, the backfill also confined the entry pillars, improving their support characteristics. A comparison of figures 6 and 8 illustrates the importance of this confinement. Whereas the 8-Right panel rib dilated 0.5 in (1.3 cm) when a major secondary principal stress change of about 220 psi (1.52 MPa) occurred, cross-panel entry pillar 1 dilated 0.5 in (1.3 cm) at a stress change of 1,035 psi (7.14 MPa).



Table 4.—Backfill vertical stress versus distance from longwall. Compressive stress is positive.

Stress levels, psi	Earth pressure cells and longwall distance (in parentheses)			
	2	3	4	5
Initial	18 (386)	9 (224)	38 (483)	18 (490)
Intermediate	23 (108)	20 (115)	136 (78)	61 (82)
Maximum	202 ((6)	161 (8)	561 (3)	339 (6)

Because the coal is stiffer than the backfill, most of the mining-induced load was supported by the cross-panel entry pillars rather than the fill. Stress changes measured in the cross-panel entry pillars were much larger than the vertical stress levels measured in the fill (2,140 psi [14.75 MPa] compared to 561 psi [3.87 MPa]). As illustrated by the elapsed time of 92.8 to 93.5 days (figure 8), a stress increase of less than 100 psi (0.69 MPa) was measured in entry pillar 1 as the longwall advanced through the first cross-panel entry. Minimal stress changes were recorded between days 89.6 and 89.8 (figure 6) and 93.6 and 93.8 (figure 8) and corresponded to utility shifts when the longwall was idle. Although the in-panel entry pillars appeared to be less stable than the 8-Right panel, no alarming stability problems were observed.

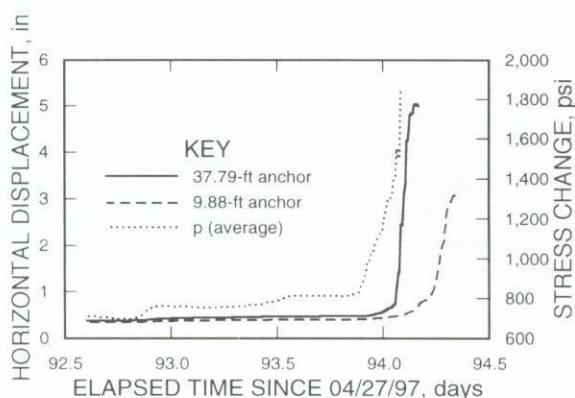


Figure 8.—Dilation and loading of cross-panel entry pillar 1 during minethrough.

HEADGATE ROOF BEHAVIOR

To monitor the stability of the adjacent headgate entry, closure stations were installed at three locations in headgate entry 1 between the conveyor belt and the 8-Right panel rib line (figure 3). Each closure station contained a string potentiometer and a set of closure points. The string potentiometer monitored roof-to-floor closure continuously, and the closure points allowed manual measurements to be taken periodically to verify the electronic readings. Near each closure station, a two-point vertical borehole extensometer was installed overhead to measure relative vertical displacements in the overlying B and C sandstones (figure 2). These instruments measured less than 1.65 in (4.19 cm) of roof-to-floor closure and less than 0.14 in (3.56 mm) of vertical displacement in the first 17 ft (5.18 m) of the mine roof, indicating that the headgate entry was very stable during

minethrough. The minimal movement could be explained as the result of the following: the instruments had to be located less than 4 ft (1.22 m) from the panel rib line; the headgate entry roof was well supported by roof bolts, mesh, and trusses; intersections with the in-panel entries were supported by wooden cribs prior to minethrough; and additional roof support was provided by the longwall gateroad shields. The instruments installed in the intersection of cross-panel entry 1 indicated that less than 8% of the measured roof-to-floor closure could be attributed to strata separation in the immediate mine roof. Therefore, most of the roof displacement appeared to occur above the C sandstone horizon.

UNDERGROUND TEMPERATURE

All electronic instruments except the string potentiometers were equipped with a temperature sensor (table 2). As expected, very little change in temperature was measured at depth within the coal. Downhole temperature sensors on the biaxial stressmeters indicated an average temperature change of less than 1.2°F (0.7°C). By comparison, the temperature sensors positioned near the roof and rib of headgate entry 1 were more exposed to the mine's ventilation circuit and, consequently, recorded larger fluctuations in temperature, particularly as the longwall passed by (figure 3). For example, the thermistors, which were mounted in the caps of horizontal extensometer 1 and vertical extensometers 1 and 2, measured an average temperature change of 5.2°F (2.9°C). Although all of the temperature sensors in the headgate entry displayed a similar response to fluctuations in the mine's ventilation system, the thermistor installed with vertical extensometer 3 measured an overall temperature change of 11.4°F (6.3°C).

The largest temperature changes were measured in the backfill and resulted from the heat of hydration as the fill cured. As the in-panel entries were backfilled, the thermistors on the earth pressure cells and embedment strain gauges recorded an average temperature increase of 49.5° and 68.1°F (27.5 and 37.8°C), respectively. Generally, the instruments installed adjacent to cross-panel entry pillar 2 near the center of the 8-Right panel measured greater temperature changes than instruments installed toward the tailgates or headgates, and the instruments installed in cross-panel entry 2 measured higher temperatures than those installed in cross-panel entry 1 (figure 3). While the thermistors on the earth pressure cells and embedment strain gauges measured an average maximum temperature of 112.0 and 131.1°F



(44.5 and 55.0°C), respectively, backfill temperatures may have reached as high as 136.2°F (57.9°C) as indicated by embedment strain gauge 5.

NUMERICAL MODELING

The three-dimensional, boundary-element program BESOL (Crouch Research, 1986) was used to model how mining in the West Mine District and the 9-Right panel affected the 8-Right panel. In plan view, this model represents a 36,000- by 10,800-ft (10,973- by 3292-m-) section of the mine that includes some of the North Mine District as well as most of the West and East Mine districts (figure 1). Material property inputs for BESOL (table 5) were based on laboratory (Schissler, 1997) and field (Beckett and Madrid, 1986) studies and consisted of determining elastic modulus and Poisson's ratio for the host rock, coal seam, backfill, and gob. To account for variations in the fly ash and other factors, such as settling of solids during backfilling, the modulus for the backfill was reduced to approximately 60% of the 28-day value listed in table 1.

Results from the global BESOL model indicate that there is a vertical stress gradient in the 8-Right panel caused by mining the 9-right panel. Stress redistribution from the West Mine District to the 8-Right panel extended from the mains 800 ft (243.8 m) inby or 5,800 ft (1767.8 m) outby 8-Right cross-panel entry 3.

To analyze the effects of mining the 8-Right panel on the backfilled cross-panel entries and in-panel entry pillars, a mesh was constructed that consisted of 180 by 180 elements and represents a 3,600- by 3,600-ft² (1097- by 1097-m²) area (figure 1). Adjustments were made in the material properties used in the model until the predicted stress changes were reasonably close to the stress changes measured by the biaxial stressmeters and earth pressure cells. This process involved reducing the coal modulus and using a nonlinear stress-strain curve to model the gob.

$$\sigma = \frac{a \epsilon}{(b - \epsilon)} \quad (1)$$

where σ = normal stress, psi

$a = 1500$,

ϵ = normal strain,

and $b = 0.02$.

Regression analyses on predicted versus measured stress changes produced correlation coefficients of 0.91 and 0.78 for the biaxial stressmeters and earth pressure cells, respectively. The stress path predicted for biaxial stressmeter 2 nearly duplicates the stress path produced from measured stress change values (figure 9). However, BESOL predicted higher stress changes for biaxial stressmeter 1 (figure 10). The data for both stressmeters indicated that the model accurately predicts the distance from the longwall face over which a stress increase occurs.

Stress changes measured by earth pressure cell 4 (figure 3) and stress changes predicted by the BESOL model were nearly identical until the longwall face was approximately 30 ft (10 m) from the instrument (figure 11). The two values diverged as the stress change exceeded the laboratory 90-day unconfined compressive strength of the backfill. Although the model generally predicted higher stresses than were measured in the backfill, it accurately predicted the distance from the longwall that stress increases occurred in the fill.

Secondary principal stress angles calculated from stress changes predicted by BESOL varied a maximum of 13° from theta angles measured by biaxial stressmeter 2. If excess horizontal stresses from earlier measurements (Bickel and Donato, 1988) are added to the initial stress state in the numerical model, this difference is reduced to 7°. Theta decreased in magnitude as the longwall approached the instrument.

Table 5. BESOL Material Properties.

	Mine-wide model	8-Right model
Rock modulus, psi	1.1×10^6	1.1×10^6
Rock Poisson's ratio	0.25	0.25
Backfill modulus, psi	50,000	40,000
Backfill Poisson's ratio	0.3	0.3
Gob modulus, psi	5,000	Nonlinear: from 75,757 psi at 8 psi applied load to 208,346 at 1,000 psi applied load.
Gob Poisson's ratio	0.3	0.3
Coal modulus, psi	400,000	150,000
Coal Poisson's ratio	0.3	0.3
Overburden weight, lb/ft ³	155	155

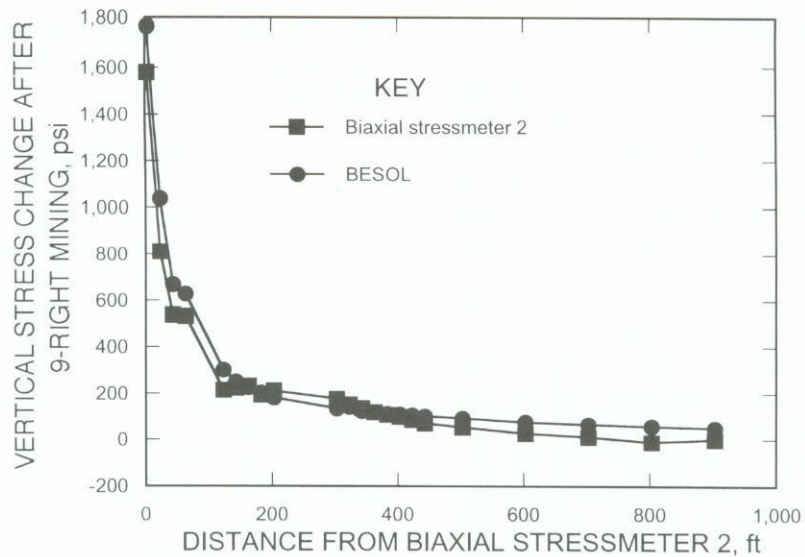


Figure 9.—Comparison of predicted and measured stress changes for cross-panel entry pillar 1.

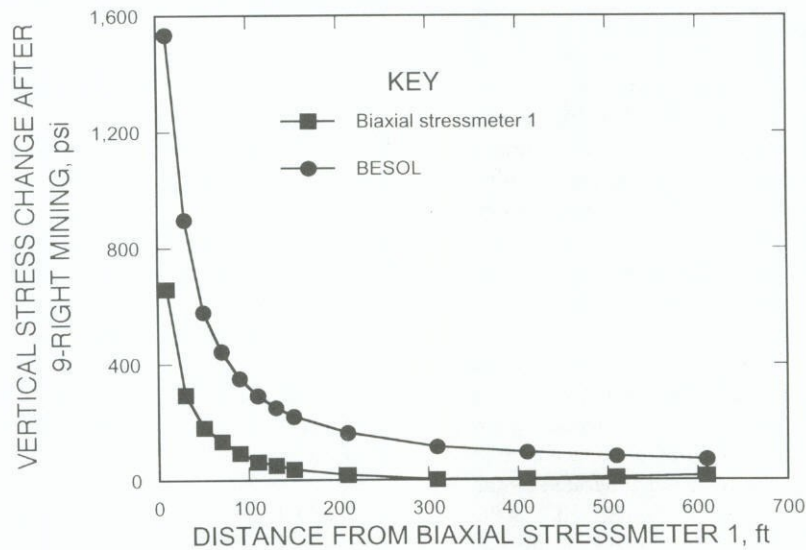


Figure 10.—Comparison of predicted and measured stress changes for the 8-Right panel.

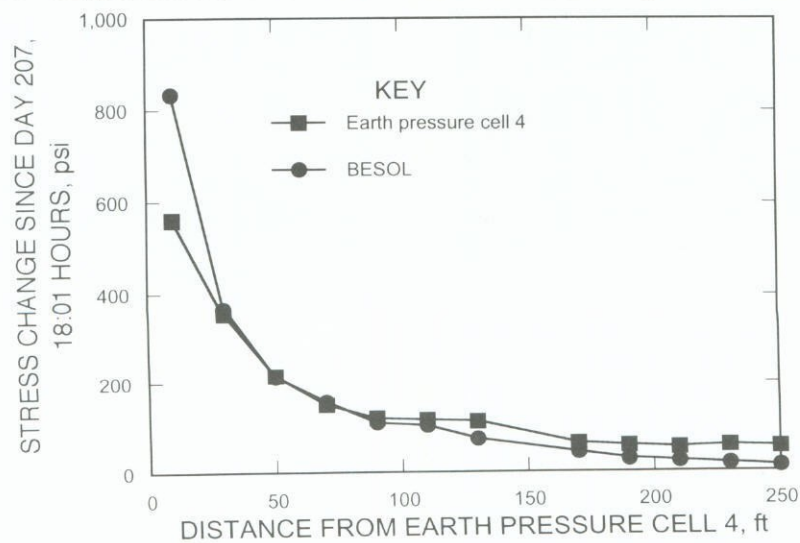


Figure 11.—Comparison of predicted and measured stress changes in cross-panel entry 2.

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CONCLUSIONS

Biaxial stressmeters proved to be reliable for measuring the magnitude and direction of mining-induced stress changes ahead of the longwall face. As the longwall approached and advanced through the backfilled section of the 8-Right panel, much higher stress increases were measured in the in-panel entry pillars than in the longwall panel. A stress increase of 2,140 psi (14.75 MPa) was measured in the center of the first cross-panel entry pillar compared with a stress increase of 902 psi (6.22 MPa) approximately 240 ft (73 m) in by the first backfilled in-panel entry. The major secondary principal stress changes were oriented from the direction of the main entries approximately 20° from vertical. The front abutment stress extended 133 ft (40 m) ahead of the longwall as it advanced through the 8-Right panel and 382 ft (116 m) as the longwall approached the in-panel entry pillars. Backfill vertical stresses did not increase substantially until the longwall was 101.5 ft (30.9 m) from the first cross-panel entry and 70 ft (21 m) from the second entry.

During the minethrough, vertical stresses in the backfilled entries ranged from 161 to 561 psi (1.11 to 3.87 MPa). In general, higher stresses were measured toward the tailgate of the entries, and higher stress levels were measured in the second cross-panel entry than in the first entry. These loading trends were confirmed by vertical displacement measurements within the fill. Although most of the mining-induced load was supported by the cross-panel entry pillars rather than by the fill, pillar dilation measurements verified that the backfill confined the entry pillars, significantly improving their load-carrying capacity.

The adjacent headgate entry was very stable during the minethrough, and most of the measured roof movement occurred above the C sandstone. While only small temperature changes were measured at depth within the coal and along the roof and rib of the headgate entry, higher temperatures were generated by the heat of hydration of the cement and fly ash in the backfill. As the placed fill cured, temperatures may have reached as high as 136°F (58°C).

Stress change measurements were used to validate a three-dimensional, boundary-element model representing the instrumented section of the 8-Right panel. The stress changes predicted by the model were consistent with stress changes measured by instruments installed in the longwall panel, cross-panel entry pillars, and backfilled entries. The model also accurately predicted the distance from the longwall face that mining-induced stress changes would occur in the coal and backfill.

The combined use of laboratory tests, instrument data, underground observations, and numerical modeling provided comprehensive information about ground behavior, rock and backfill properties, and performance of the fill, as well as the stability of the backfilled section of the 8-Right panel during minethrough. Stresses measured ahead of the longwall face are similar to the range of stresses reported for the panel, backfill, and in-panel entry pillars during a similar minethrough of backfilled entries at the Emerald Mine in Pennsylvania (Chen et al., 1997).

By documenting the safety and stability of the 8-Right panel minethrough, this study should lead to further successful longwall backfilling applications.

ACKNOWLEDGMENTS

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Mining Through In-panel Entries and Full-face Recovery Room Without Standing Support at U.S. Steel Mine 50

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ABSTRACT

Mining through in panel entries and the full face recovery method has been successfully practiced at U.S. Steel, Mine 50. Support of the cut-through entries and recovery room is critical. Traditionally, concrete crib or fly ash cement backfill has been utilized. However, these traditional methods are not feasible at Mine 50 due to the difficulties of the plow face mining through these types of support. Recent development of new cable support systems provides an option for cut-through entry support. Mine 50 and Jennmar Corporation personnel have worked together to design and apply the cable systems in the cut-through entries and full face recovery room to eliminate standing support. To date, a number of full face recovery rooms and cut-through in-panel entries have been successfully mined. The results indicate that the cable systems combined with high capacity shields, have provided sufficient capacity to support the roof.

Utilizing the Jennmar Finite Element Analysis (JFEA) computer program (Stankus and Guo, 1996), an analysis of the stress and roof deflection changes as the longwall gradually cut through was conducted. From the finite element analysis data, a supplemental support plan was designed. The history and experiences at Mine 50 are presented. Based on the case experiences and modifications made to the finite element analysis, an improved design criteria for future use has been formulated.

INTRODUCTION

At U.S. Steel Mining's Pinnacle Mine 50, the Pocahontas No. 3 seam with average mining height of

54" is longwall mined with a plow face. Previously, the longwall face was recovered using recovery chutes. To reduce the longwall move time, the full face recovery method with standing support was utilized. Currently, the full face recovery method and cutting through in-panel entries without standing support has been successfully applied. Longwall move time has been significantly reduced with associated cost savings.

The in-panel entries are developed under the following scenarios:

The in-panel entries are generally developed due to adverse geology. Since the plow is unable to cut rock, wherever a fault is present, an in-panel entry has to be pre-driven along the fault to mine out the rock. The lengths and directions of the in-panel entries depend on the fault. Usually, these entries extend partially across the panel.

Critical design parameters for the roof support are the type of support, support density, bolt length, and minimum required shield capacity. Further, prediction of entry convergence is important because of the low mining height. Mine 50 and Jennmar personnel have been working together to design and apply cable systems to successfully eliminate the standing support.

COMPARISONS OF DIFFERENT ROOF CONTROL PLANS

Recovery Room

Three options were compared for the recovery method. The first option consisted of using recovery

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chutes. This method has been successfully practiced. However, the method is time consuming and labor-intensive to bolt the recovery room. The second option was a full face recovery room with standing support (fiber crib or high capacity wood post). However, due to the difficulties of the plow cutting, the standing support had to be physically removed as the longwall cuts through. This practice is inconvenient and time-consuming. The third option was a full face recovery room without standing support. This is the most economical and fastest option, however it does have the highest risk.

Cutting Through In-Panel Entries

Three possibilities were considered and compared for cutting through three in-panel entries. The first option was to move the longwall from the inby to outby entry. This plan is most costly in terms of the labor and longwall idle time. The second option was backfilling the entries with grouted material and mining through them. This procedure has been tried at Mine 50. However, it slows down the plow's rate of advancement significantly because of the difficulty cutting the grout material. The third option was to mine straight through the in-panel entries with appropriate roof support. Comparisons of these options are listed in Table 1.

Table 1 Comparisons of Three Plans

Description	Positive	Negative
Plan 1: Face move	Low risk	Labor: 585 man shift Cost: \$405,113 7 day idle period for LW
Plan 2: Fill entries with a grout material and mine through them	Labor: 80 man shift Tonnage: 35,000 raw	Cost: \$728,735 Grout material to plan Slower advancement
Plan 3: Support in-panel entries and mine through them	Cost: \$275,231 Tonnage: 35,000 raw	Labor: 408 man shift

By comparison, economically, Option 3 is the best but it has the highest risk factor.

It was believed that with the current cable system technology, if the roof support is properly designed, this method could be successful. The key factors that had to be considered:

- **Critical Time:** As the face approaches the entry, the pillar between the entry and face will gradually yield and the residual strength of the pillar will decrease. As a result, the roof deflection will increase rapidly. The duration between the roof deflection abrupt increase and face cut through is defined as the critical time. During this period, the supplemental

support will take the roof load and transfer this load to the shield. In turn, the shield load will increase. The maximum load on the shields will occur at the time when the remaining pillar has completely lost its residual strength.

- **Minimum Shield Capacity:** The shield must have the capacity to handle the transferred load from the recovery room.

- **Roof Convergence:** The roof convergence cannot exceed an amount lower than the minimum shield working height.

- **Support Density:** The effective density must be high enough to support a certain height of entry roof and transfer the load to the outby pillar, and the shields.

- **Vertical and Angled Bolt Length:** This is a function of the roof strata properties.

THE SUPPLEMENTAL SUPPORT DESIGN

Finite Element Analysis

The finite element analysis method was utilized to determine the roof deflection, critical time, and stress distribution. Six 2-dimensional finite element models were constructed to analyze the stress on the pillar between the face and the recovery room as the face gradually approaches the recovery room. Shown in Fig. 1 is the finite element model.

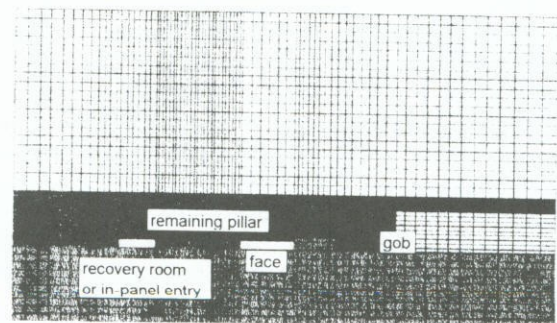


Fig. 1 Finite Element Model

Pillar Yielding Process

Fig. 2 shows the average stress change on the pillar and strength change as the face approaches the

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recovery room. It can be seen that the pillar starts to yield when the face is 25' inby the recovery room. When the face is 20' inby, most of the pillar is yielded. The stress on the pillar will not follow the stress curve obtained from an elastic solution due to continuing yielding. Instead, the pillar will retain some residual strength. It is known that residual pillar strength is related to the width/height ratio of the remaining pillar, which is plotted in Fig. 2. Therefore, the dotted line in Fig. 2 represents the actual pillar stress,

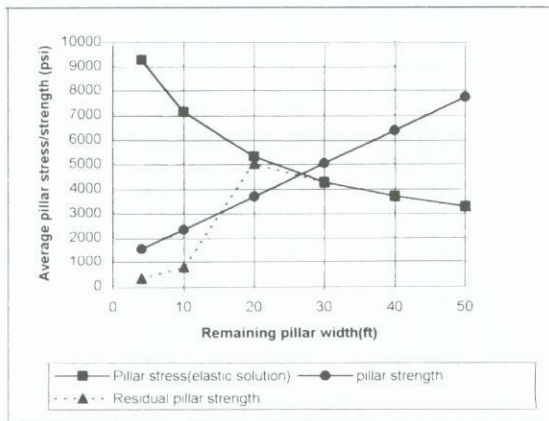


Fig. 2 Modeled Average Stress Change on Pillar

Roof Deflection and Critical Time

After the pillar is yielded, the residual strength is used in the finite element model. Three additional models were run to simulate the stress and roof deflection after pillar yielding. Fig. 3 shows the roof deflections predicted by finite element analysis. It can be seen that when the pillar yielded, roof deflection starts to increase. The critical time is when the face is 0'-20' inby. During this period, the face must be continuously advanced as fast as possible to minimize the time-dependent roof deflection.

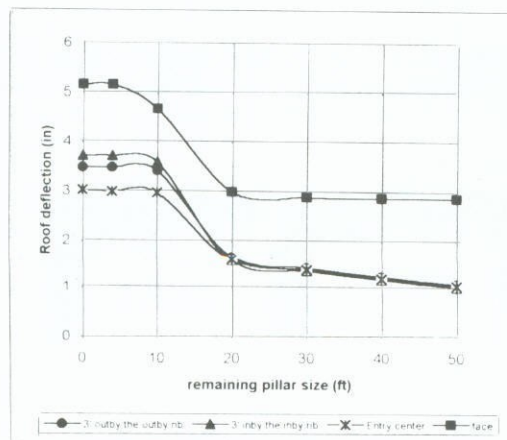


Fig. 3 Modeled Roof Deflections

The maximum roof deflection predicted by the models is shown in Table 2.

Table 2 Maximum Roof Deflection Predicted

POINTS	1	2	3	4
Roof deflection (in)	3.00	3.50	3.75	5.2

Note: Point 1 is 3' inby the outby rib, point 2 is at entry center, point 3 is 3' outby the inby rib, and point 4 is 8' inby point 3.

Minimum Shield Capacity

As the face approaches the recovery room, the shields will be subjected to transferred load from the recovery room roof. When the face is 5' inby, the yielded pillar has little residual strength. The load from the recovery room will be transferred to the shields. Fig. 4 shows the load transfer mechanism. Load generated by twenty feet of the immediate roof, which is five times of the mining height, will be transferred to the outby pillar and the shield. Assuming half of this load is transferred to the shield and a 1.5 safety factor is considered, the minimum required shield capacity should be higher than 524 tons. The current shield capacity is rated 650 tons, which meets the requirement.

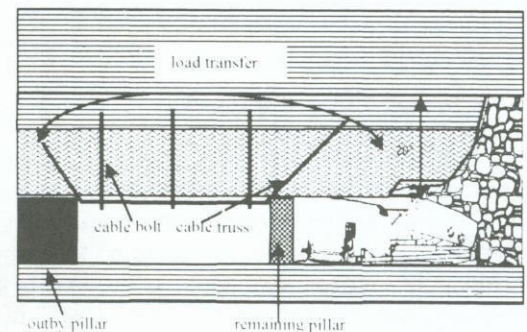


Fig. 4 Load Transfer Mechanism

Determination of Supplemental Support Parameters

The cable bolt and tensioned cable truss will be utilized as the supplemental support system to support the recovery room and transfer load to the outby pillar and shield. The parameters include the basic support plan, support density, and the length of vertical cable and angle bolt of the truss.

Support Plan

Fig. 5 shows the modeled shear stress changes at the outby and inby roof corner and tensile stress changes at mid-roof.

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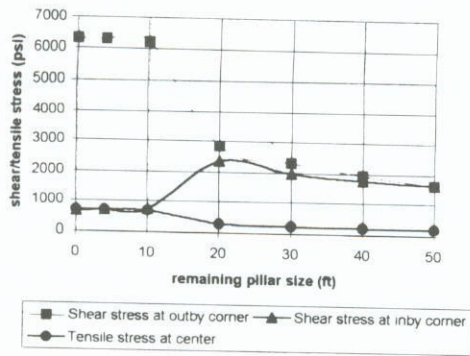


Fig. 5 Modeled Shear and Tensile Stress Changes

As the face advances from 50' to 20' inby, shear stresses at both roof corners and the tensile stress at mid-roof slightly increases. From 20' inby till cut through, the shear stress decreases at the inby corner and abruptly increases at the outby corner with a maximum stress of 6300 psi. This indicates that after the pillar yielded, roof failure in the form of cutter roof may occur at the outby corner. Also the tensile stress increases at mid-roof, with a maximum value of 780 psi. In order to address the stress conditions, three rows of cable bolts and tensioned cable trusses are required in conjunction with a T-5 roof channel.

Lengths of Cable Bolt and Truss

The cable bolt and truss system functions not only to control the possible immediate roof failure, but also to transfer the load to the outby pillar and shield as shown in Fig. 4. The cable should be long enough to suspend the immediate roof from a strong roof layer. The stratascope data indicated that the first 7' of roof consisted of laminated sandy shale or shale. From 7' up, the roof is a strong sandy shale with few laminations. The length of cable bolt should be no less than 12' with 4' minimum resin encapsulation. The length of the angle bolt of the truss should be enough to reach over the shield when the pillar has completely lost its residual strength. As shown in Fig. 4, the angle bolt is installed at an angle of 45°, the pillar is 5' wide (no residual strength), and the distance between the shield tip and coal face is 2'. Therefore, the minimum length of the angle bolt should be no less than 10'. The outby angle bolt should be at least 8' in length.

Spacing of The Cable Bolt and Truss

The spacing of the cable bolt and truss is 4'. The load density is 17 tons/ft. of recovery room. Therefore, the total load for 4' of recovery room is 68

tons. The total support capacity for 4' recovery room is 132 tons, which equates to a safety factor of 1.9.

The finalized supplemental support plan is shown in Fig. 6.

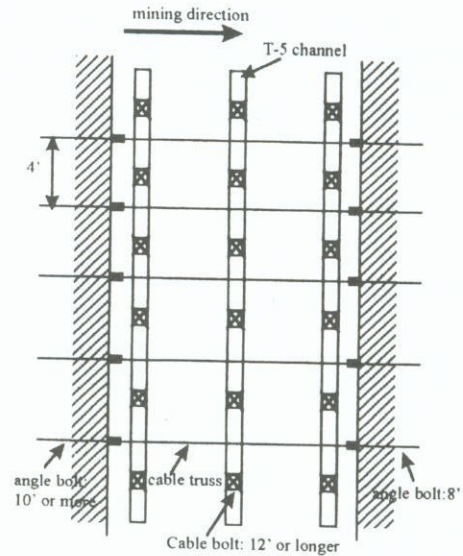


Fig. 6 Finalized Supplemental Support Plan

HISTORY AND EXPERIENCES

The longwall face recovery methods have evolved in four stages as new support technology also developed:

Stage 1 - Regular Recovery Method: Before 1992, all the longwall faces were recovered by using the regular method (roof and rib bolting) because adequate support technology was not available.

Stage 2 - Recovery Chutes: In 1992, the first recovery chutes were utilized in the 6N and 7N panels. The recovery chutes were successful and recovery time was reduced.

Stage 3 - Full Face Recovery Room with Standing Support: In 1993, a partial full-face recovery room was tried in the 5N and 4N panels as shown in Fig. 7.

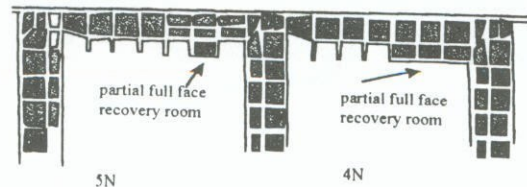


Fig. 7 Partial Full-Face Recovery Room - 4N and 5N Panels

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The roof was supplemented with rigid rod bolts and trusses combined with fiber cribs as shown in Fig. 8.

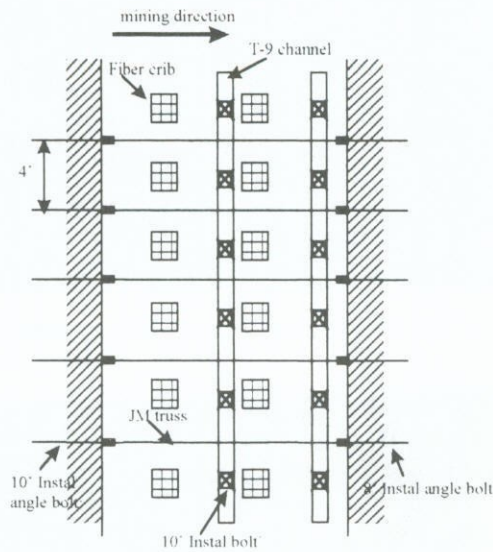


Fig. 8 Support Plan for 4N and 5N Recovery Room

In 1994, based on the experience from 5N and 4N panels, the first full face recovery room was employed in 1RT panel with the same support plan as shown in Fig. 9.

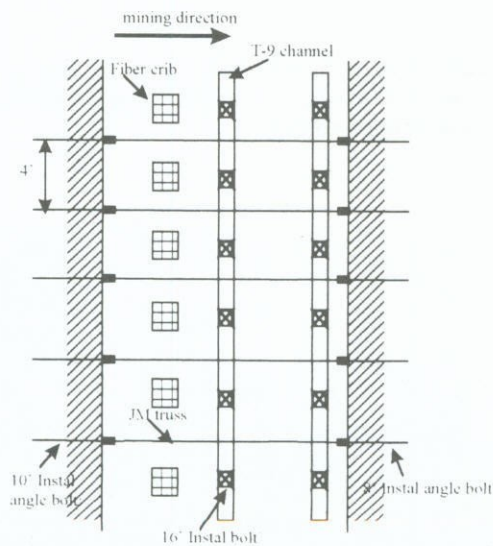


Fig. 9 Support Plan for 1RT Full-Face Recovery Room

However, since the plow was unable to cut the cribs, they had to be removed manually as the face cut

through. This was time-consuming and labor-intensive. To expedite the face recovery, the fiber crib was replaced by a high capacity wood prop in 7I and 2RT panels in the fall of 1995. The full-face recovery room with standing support was successful. However, the manual removing of the standing support, whether it was fiber cribs or wood prop, was always time-consuming and labor intensive.

Stage 4 - Full Face Recovery Room without Standing Support: The high capacity and flexibility of the cable bolt and tensioned cable truss made this possible. Since 1995, using the supplemental support plan shown in Fig. 6, five panels (7K, 3RT, 4RT, 8N, and 9N) have been successfully recovered utilizing this method. An in-panel entry system (three entries in 9N panel) has also been successfully cut through. A number of partial in-panel entries, which were created to eliminate faults, have also been successfully mined through using the same support plan.

Three typical cases are presented in detail as follows:

Case 1: 4RT Panel Full Face Recovery Room and In-Panel Entry

Shown in Fig. 10 is the 4RT panel recovery room. During development of the recovery room, a fault was found 160' from the tailgate. A 50' long and 20' wide in-panel entry had to be driven almost perpendicular to the recovery room to cut the rock out along the fault. The supplemental support plan shown in Fig. 6 was utilized in both the recovery room and in-panel entry.

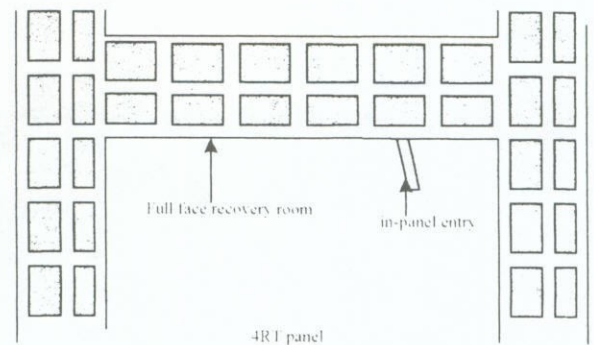


Fig. 10 4RT Recovery Room with Fault Cut Out

The longwall face successfully cut through the in-panel entry and into the recovery room without any problem. The total convergence in the recovery room was 1-3". No roof failure or cutter roof occurred. One

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year later, this area was revisited. The recovery room was still standing and in good shape.

Case 2: 7K Panel Full-Face Recovery room

Fig. 11 shows the 7K panel recovery room with the same supplemental support as shown in Fig. 6. In order to evaluate the supplemental support, an instrumentation program was designed as shown in Fig. 11, which included two load cells, a convergence station and stratascope borehole.

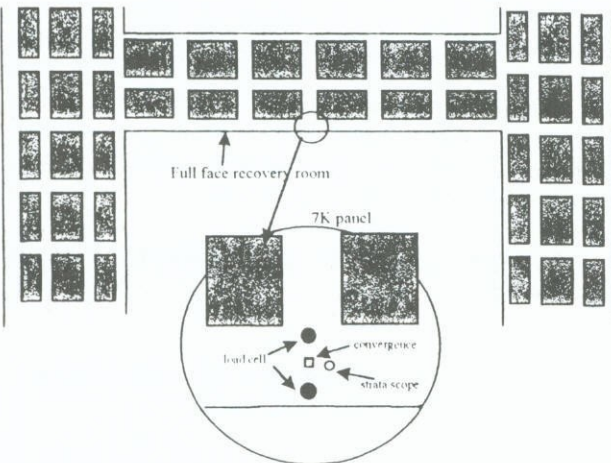


Fig. 11 Instrumentation Plan - 7K Panel Recovery Room

The longwall face successfully cut into the recovery room and was recovered without any problem. Generally, the inby rib incurred minimal sloughage as shown in Fig. 12. However, at mid-face, the rib sloughage was 1-1.5' (Fig. 13). The outby rib remained in good condition as shown in Fig. 14.

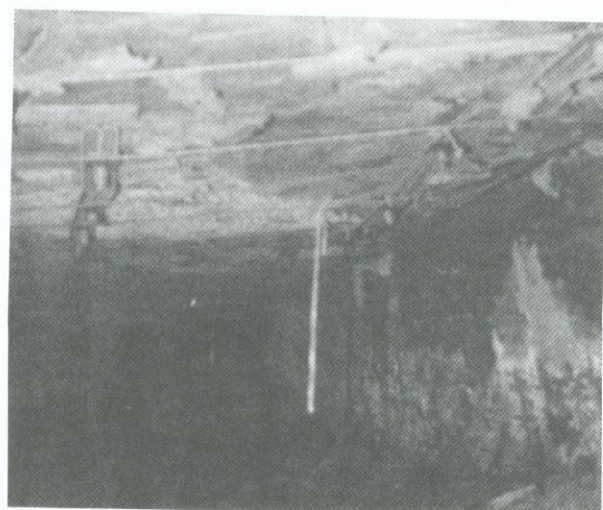


Fig. 12 7K Recovery Room - Inby Rib

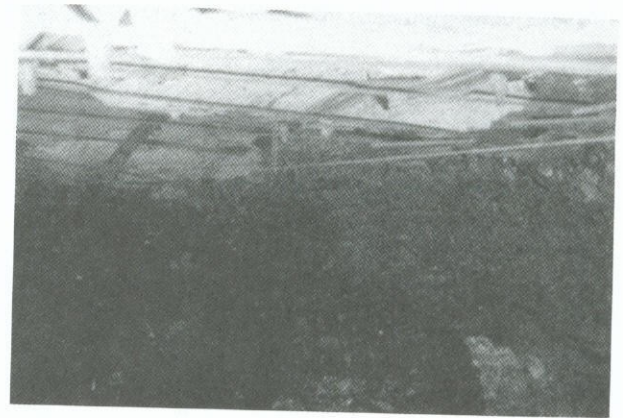


Fig. 13 7K Recovery Room - Inby Rib Mid-Face



Fig. 14 7K Recovery Room - Outby Rib

A visual record of the recovery room as the face cut in, is shown in Figs. 15 through 20. Fig. 15 is looking into the face from headgate.



Fig. 15 7K Recovery Room - Face From Headgate

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Fig. 16 is the recovery room at the tailgate side as the plow was cutting into the recovery room.

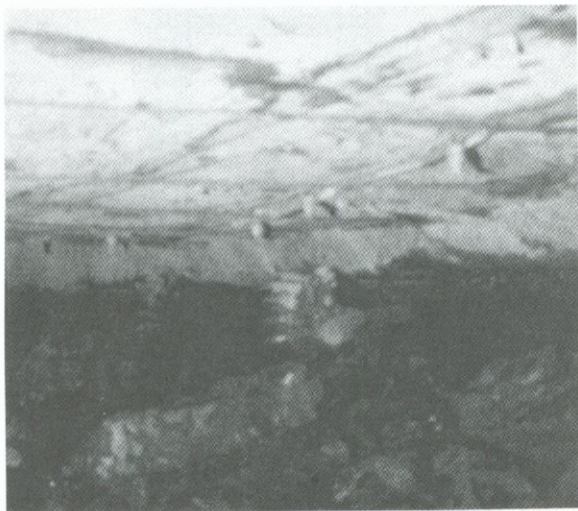


Fig. 16 7K Recovery Room - Tailgate w/Plow Cutting into Room

Fig. 17 is the recovery room at mid-face as the plow was cutting into the recovery room.

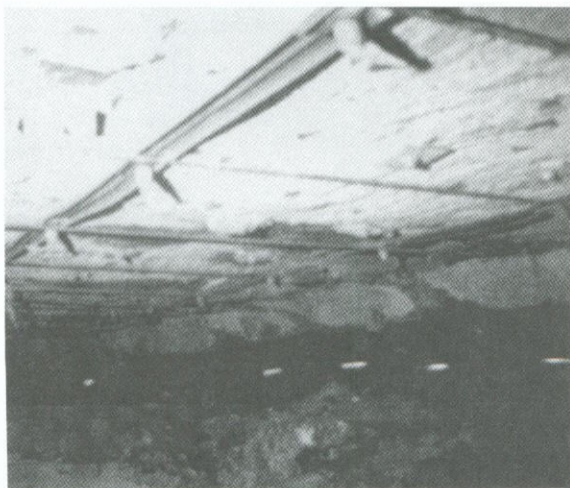


Fig. 17 7K Recovery Room - Mid-Face w/Plow Cutting into Room

Fig. 18 is the recovery room at the headgate side as the plow was cutting into the recovery room.

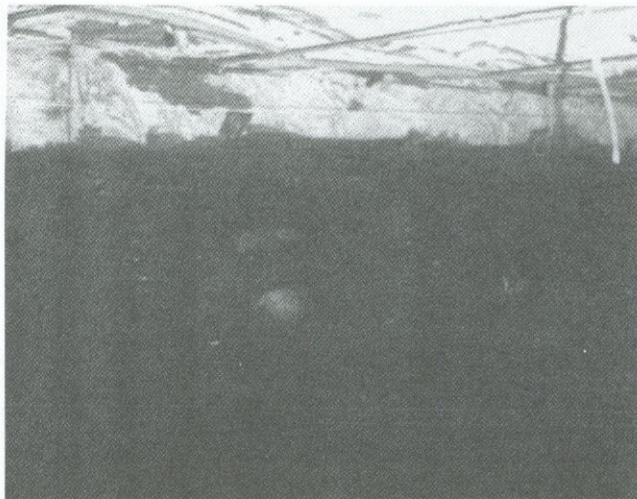


Fig. 18 7K Recovery Room - Headgate w/Plow Cutting into Room

Fig. 19 shows the tailgate condition inby the shield.



Fig. 19 7K Recovery Room - Tailgate Condition Inby Shield

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Fig. 20 is an overall view of the recovery room during the plow cutting through.

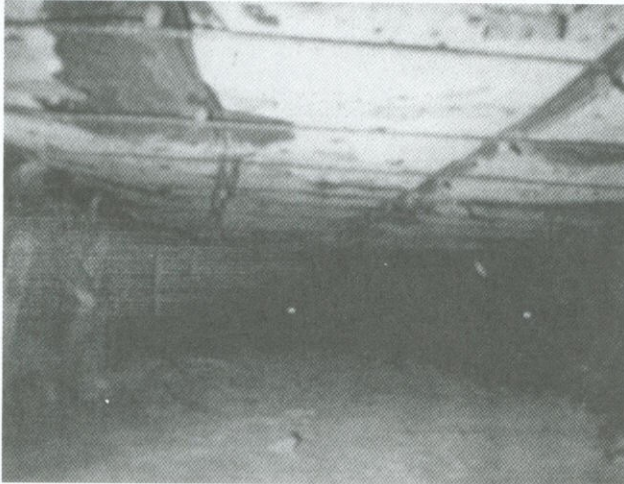


Fig. 20 7K Recovery Room - Overall View w/Plow Cutting into Room

The instrumentation indicated that a number of separations and lateral roof movement occurred below the 7' roof horizon. The cable bolt load was 13 tons. This equates to 57 tons of the transferred load per 4 linear-foot recovery room. The maximum convergence in the recovery room was 2". Visual observation showed that a small roof shear crack occurred at the outby rib side. Overall, the supplemental support system successfully controlled the roof and transferred the load to the shield and outby pillar. The instrumented results were very close to the modeled values.

Case 3: 9 North Panel - Partial In-Panel Entry, In-Panel Entries and Full-Face Recovery Room

Shown in Fig. 21 is the 9 North panel.

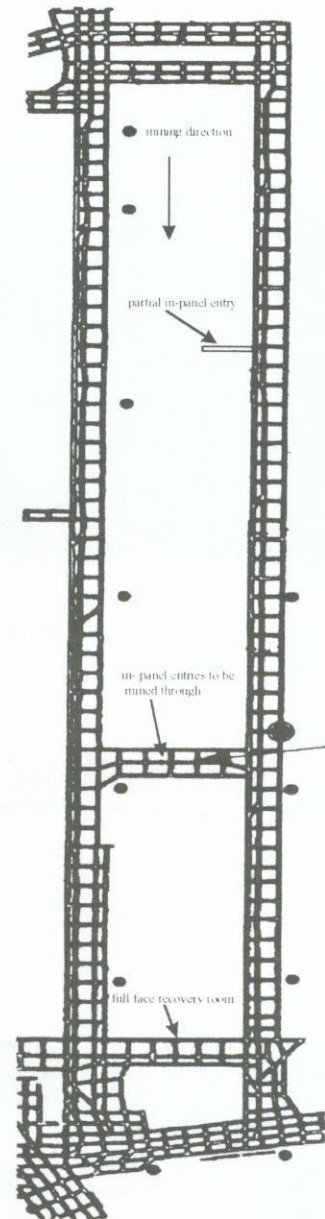


Fig. 21 9 North Panel

During panel development a fault was found in the headgate. The rock in the fault was mined out by a pre-driven entry. Initially, the recovery room was driven across the panel 2000' short of the original longwall recovery site because the thickness of the coal seam decreased to 38" and conditions outby that point were similar. However, as the panel was mined

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and favorable experience was gained from mining at these heights, the decision was made that the face would plow through the recovery room and mine the last 2000'. The face was eventually recovered from the original recovery site. This panel involved three cases: a partial in-panel entry, three in-panel entries and a full-face recovery room.

The roof in the partial in-panel entry was bad because of the fault influence. In addition to the supplemental support plan shown in Fig. 6, two extra rows of cable bolts were used as shown in Fig. 22.

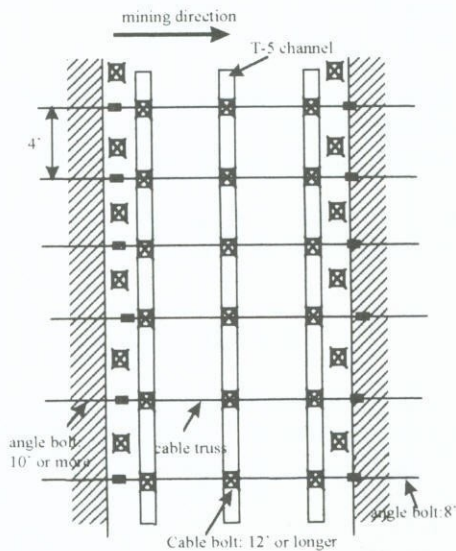


Fig. 22 9 North Panel - Support Plan Partial In-Panel Entry

The in-panel entry and full-face recovery room were supported by using the supplemental support plan shown in Fig. 6. However, difficulties were expected in cutting through the in-panel entries due to the low mining height. It was decided that before the face cut through, 1'-5" of draw rock (see Fig. 23) would be cut to give a higher mining height. Instrumentation was set up in the in-panel entries to monitor the convergence and the cable bolt load.

The partial in-panel entry (fault) was cut through without any problems. The in-panel entries were cut through with some difficulties, but the supplemental support plan provided adequate support allowing the longwall to maneuver through without becoming permanently trapped.

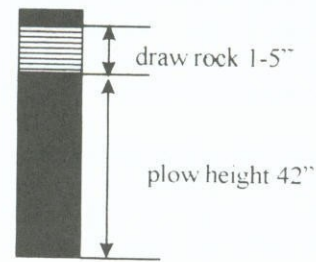


Fig. 23 9 North Panel - Draw Rock Above Coal Seam

As expected, some difficulties were encountered during cut through in the in-panel entries. Geological conditions and equipment failure contributed to the difficulties that arose during the cut through. Before mining into the first entry, the plow height was set immediately under the draw rock. The draw rock accumulation atop the shields limited its setting span to 4". Once the plow mined within 5' of the first entry, equipment failure occurred and mining was delayed. As analyzed by finite element method, this was the most critical time. The remaining pillar lost its residual strength and the majority of the roof load in the recovery room transferred to the shields. The roof convergence in the recovery room and face reached the maximum value, and the time-dependent roof deflection increased the total deflection significantly. The measured maximum convergence was 8" in the recovery room which was about 5" more than usual. Although not measured, the convergence in the face was predicted to be approximately 10" (finite element model predicted 5.2" plus time-dependent 5"). This reduced the shield height to 34", which is lower than the minimum working height of 38". As a result, No. 13 shield was trapped on solid. A hydraulic puller was then used to advance the shield into the entry. In spite of the excessive convergence, the recovery roof was in very good condition. No roof failure was experienced. The supplemental support plan had successfully supported the roof and transfer load to the shield and outby pillar. Fig. 24 shows the measured load on the cable bolt.

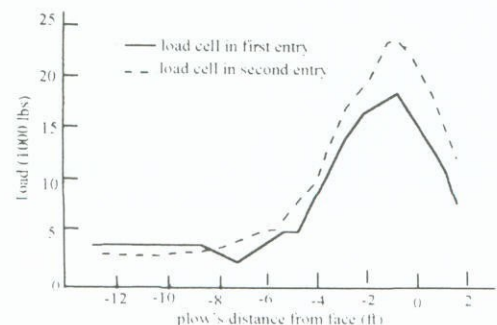


Fig. 24 9 North Panel - In-Panel Entries Load Cell Data

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It shows that the cable bolt load started to increase when the face was 5' inby. This was the most critical time as defined by the finite element analysis. The maximum load was approximately 10 tons.

The second entry was successfully cut through without any equipment delay or roof problem. The measured roof convergence was approximately 3". The load on the cable bolt was 12 tons.

The third entry nearly duplicated the problems encountered in the first entry. When the plow cut through leaving a 27' wide opening, equipment failure occurred on the pan line. As a result, the plow remained idle for nearly 4.5 hours. During this delay, the maximum convergence in the recovery room was measured at approximately 8". Although this presented potential difficulties as the plow began to advance, the shields were able to function properly due to the earlier adjustment to the cutting height.

The full face recovery room was successfully supported and the face was recovered without any problems. The convergence experienced was 1-3".

From this case, the magnitude of the time-dependent roof deflection is determined. The prediction curve shown in Fig. 4 can be modified as shown in Fig. 25 by considering the time-dependent effect. The minimum shield working height is 38". Therefore, if no delay was expected, the minimum cutting height should be 42". Otherwise, it should be 48".

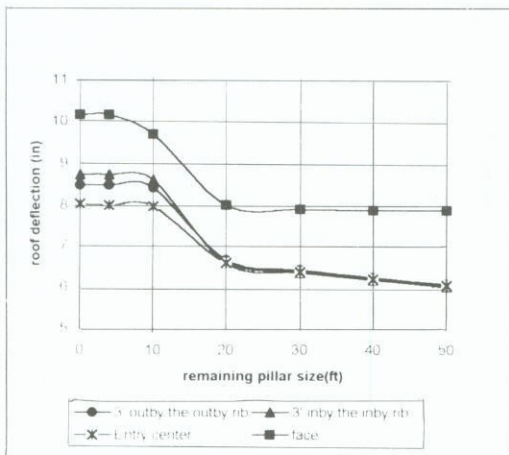


Fig. 25 Prediction Curve - Roof Deflection w/Time Dependent Effect

CONCLUSIONS

Overall the full-face recovery room and in-panel cut through entry have been successfully supported by the engineered supplemental support plan without any

delay during the critical time period. However, in the low mining height area, if any delay occurred during the critical time period, major problems were encountered as predicted. The experiences in the 9N panel provided valuable information for future applications. The major conclusions and suggestions are summarized as:

- The supplemental support plan currently implemented is adequate to support the recovery room and transfer the roof load to the shield and the outby pillar.
- The prediction by the finite element analysis is accurate without the time-dependent effect.
- The prediction curve is modified incorporating the time dependent experiences in 9N panel. This is the base for the future support design.
- Based on the modified prediction curve, if any delay is expected during the critical time period, the plow cutting height should be adjusted to 48" or more.
- Higher capacity shield would help without adjusting the plow cutting height.

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Cutable and Variable Yield Cement Cribbing Successfully Support 84 Mine Longwall Cut Through Entries

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Abstract

The development of increasingly more sophisticated and powerful retreat longwall equipment has led to ever wider and longer panels. To maximize gateroad development efficiency, provide for effective face recovery, and/or reduce ventilation pressures, entries are cut across longwall panels in advance of longwall retreat mining. These entries must be supported to prevent falls of immediate roof in advance of the face. Recently, cut through entries and pre-driven recovery room entries have been supported with various combinations of roof bolts and passive support. The cost and relative success of these support systems has varied considerably. Eighty-Four Mining Company, Fosroc, and Sub-Technical have developed and implemented an efficient, yielding, multiple component, cement crib to support longwall panel cut through entries and pre-driven recovery rooms. The cutable crib consists of an inexpensive flyash cement pillar poured in a bag, within a temporary metal form. The form is moved to a subsequent cutable crib location after the initial hardening of the flyash pillar. The remaining entry height between the mine roof and the top of the flyash cement pillar is filled with a specially designed bag filled with Fosroc's Tekset cement formulation. The diameter of the pillar and thickness of the Tekset bag are designed to accommodate the anticipated front abutment loading and entry convergence.

In the 84 Mine, over 3,700 linear feet of cut through entries were supported with the cutable cribs and combination bolts installed during continuous miner development of the Pittsburgh Coalbed. The flyash cement was pumped from the surface and the Tekset was mixed and pumped underground. The 4 ft diameter cutable crib system provided very effective roof support. No falls of roof occurred throughout the supported entries and minimal longwall mining delays were experienced.

The mining induced deformation behavior of 22 of 431 cutable cribs was measured, confirming the yield induced by abutment pressure in advance of the face was confined to the Tekset yielding bag.

Background

Traditional longwall secondary and supplemental roof support consists of wooden posts and cribs. The deformation characteristics of wood allow the design and construction of roof support compatible with the coal roof, floor, and pillars. However, the remarkable production rates of modern longwall retreat sections require very rapid installation of secondary and supplemental roof support. These same supports must provide ever increasing reliability to protect the massive capital invested in longwall mining equipment. Further, the high strength and low cost wood necessary for longwall roof support is in increasingly short supply. In reaction to these trends a number of products have been developed to replace traditional wood supports (Barczak and Gearhart, 1993).

Fosroc has led the development of pumpable cement products for longwall roof support. In 1993, the Southern Ohio Coal Company employed Fosroc's Tekcrib to support gateroad entries at the Meigs No. 31 Mine. Tekcrib supplemental supports consists of Tekcrib foamed cement pumped into tubular bags. Due to the difficulties to access location, in this case the cement was mixed on the surface and pumped down a borehole (Amick, et al., 1993). In 1995, the Cyprus Coal Company employed Fosroc foamed cement to reinforce a severely deteriorating tailgate at the Cyprus Shoshone mine. An additional 500 tons of support capacity was placed along the tailgate by filling the wooden cribs with Tekseal® pumped down a surface borehole (Woomer and Stewart, 1995). Fosroc has developed and tested a pumpable tailgate support called Tekprop. An 18 in diameter Tekprop provides over 100 tons of support in response

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to less than ½ inch of roof-to-floor convergence (Campoli, et al., 1997).

The 84 Mining Company has pioneered the development of pre-driven recovery rooms during longwall mining of the Pittsburgh Coalbed (Bauer, et al., 1987). Seven previous longwall panel recoveries were completed at the 84 Mine by mining into an open entry that was supported by various types of concrete supports.

Sub-Technical Inc. has significant experience constructing pumpable cement cribbing at many mines, including the previous pre-driven recovery room supports employed at the 84 Mine.

The cumulative experience of Eighty-Four Mining Company, Fosroc, and Sub-Technical was employed in the design, construction, and implementation of an efficient, yielding, multiple component, cement crib to support cut through entries, driven through a longwall panel in the Pittsburgh Coalbed.

Mining Method

The layout of the 84 Mine in the area of the 3 North panel is shown in Figure 1. The 2 North and 3 North sections were developed using Joy 12CM14 "in place" continuous miners with integral satellite bolters. Production equipment consisted of Joy shuttle cars and a Fletcher twin boom bolter.

Both development sections have 3 entries. Entry widths are maintained at 16 feet wide. T-5 steel channels are installed on 4 foot centers in all three entries. The two outside bolts are installed during mining and the two center bolts are installed in cycle prior to advancing the belt. Roof bolts are a 2 piece combination bolt with 4 feet grouted and 4 feet tensioned. The entries are advanced on 130 foot centers with crosscuts on 80 feet and 100 foot centers. The crosscuts are mined on 60 degree angles.

General Conditions

Roof conditions are generally good but do vary in localized areas, especially in areas where stream valleys cross the entries. Mining heights vary but are usually maintained at eight to nine feet in this area. The main bench of the Pittsburgh Coalbed averages 76 inches in this area. The draw slate is approximately 12 inches thick throughout the area and the rider seam will vary from a few inches thick up to three feet thick.

Entry heights in the angled entries between the 2 North and 3 North sections ranged from a low of 88 inches to a maximum height of 144 inches. The average height was 112 inches. Rib conditions were very good with minimal spalling during the complete developmental

stages of the project. Intermittent methane, water problems, and soft bottoms hampered development of these two sections.

Necessity for the Project

The 1 North longwall section quickly advanced out of the 1 North panel with minimal problems. The 2 North panel development lagged behind schedule, causing a severe timing problem for the 2 North longwall panel. In addition, negotiations for an adverse property in the 2 North panel were ongoing and stretching to the critical stage. The timing problem coupled with the adverse property negotiations prompted the decision to cut short the 2 North panel. With this decision being made, a plan was developed to continue to develop the 3 North panel tailgate while the longwall was being set-up and mining in the 2 North panel. This meant that entries must be cut across the panel and a belt and ventilation established from the 3 North side of the panel. This portion of the plan was relatively straight forward. The ability of the longwall equipment to successfully mine through this area required that a detailed plan be developed and implemented.

Planning Parameters

During this time period, other mines were cutting through entries that were developed across the panels or across a portion of the panel. Attempts were made to gather as much information as possible about each of these particular cut through designs. In addition, seven previous longwall panel recoveries were completed at Mine 84 by mining into an open entry that was supported by various types of concrete supports. This history and the current information were utilized to formulate a plan for the longwall to mine through these three entries. Critical areas of the plan were:

1. The project must have minimal impact on the remainder of the mine.
2. It must be completed within a critical window time frame.
3. Risk and costs must be minimized as much as practical.
4. Safety during the initial construction phase and during the longwall cut through must not be compromised.

System Design

In order to complete this project with minimal impact on the mine, the materials would need to be transported through a borehole on the surface to the entries to be filled. The forms must be reusable. They must be easily taken apart transported and rebuilt. Each vendors ability to meet these needs was reviewed and evaluated. In addition, a number of other ideas were

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tried on the surface with varying degrees of success.

The critical window to complete this work was dictated by the need to recover the 2 North beltline and track haulage, crib the bleeder and tailgate entries, and be completed with time to allow proper curing time for the concrete prior to the longwall cutting into this entry.

In order to minimize the risks involved, a number of key decisions were made. These included mining the angled entries from both sides, thus allowing the number of intersections to be minimized, offsetting as many intersections as practical and mining the entries on an angle so that the surrounding coal could be utilized to help support the abutment pressures that would be generated. Entry widths would be maintained at 16 feet. Additional support (roof bolts) would be in the form of 15' cable bolts or 12' combination bolts 1" in diameter. The cuttable cribs would be placed to provide support densities that have proven successful in the past. These support densities varied for entries, crosscuts and intersections. This enabled the mine to reduce the volume of material required which reduced the costs and the time period to complete the project.

Throughout the entire construction phase of the project, the roof and rib conditions remained excellent. Efforts were taken to insure that the concrete forms were stable and would not fall during the filling process. The concrete pumping operation was set up to minimize problems. The forms were built to be easily constructed and disassembled as well as moved from place to place. Communications between the various crew members were maintained by remote headsets and pager phones. This proved to be critical during the pumping operation.

During the time period that the longwall mined through the cuttable cribs, rib conditions in the front abutment zone were poor. Exposure to this area was minimized to the group of people assigned to monitor the cuttable cribs.

The ability to maintain support until mined by the longwall shearer was another critical aspect of the support design. The support had to maintain its structural integrity to the end as well as it having to provide yield to the roof rather than being extremely stiff. The poured concrete would not provide any yield, therefore another component had to be added to the system. A yieldable material was added to the top of each concrete column. This filled the gap to the roof and provided initial support to the immediate roof (figure 2).

Support Densities

During the previous seven open ended longwall recoveries, the type of support and the support densities varied. Using this information, a system of support was

designed for each area in the angled entries. The area was divided into four separate groups, the entries, the crosscuts, three way intersections and four way intersections. A support density was designed for each area. A total of 431 cuttable cribs were constructed in approximately 3,700 linear feet of cut-through entries.

Crosscuts

Each of the seven crosscuts was oriented very close to perpendicular to the approaching longwall face. Past observations of longwall cut-out rooms showed that roof conditions in this particular orientation were very good. In most cases, rooms and recovery entries in this orientation did not have any additional support and stayed up well during longwall recoveries. Therefore, a minimum amount of additional support would be required. The design support density in these crosscuts was 52 psi. The actual support density varied up to 72 psi based upon the strength versus cure time graphs provided by Fosroc.

Entries

These three entries were oriented approximately 60 degrees off of the headgate entry or 30 degrees off of the approaching longwall face. As the longwall face approached, the surrounding coal would be required to support a good portion of the longwall abutment pressures. The coal pillars between the entries would also be required to support the immediate roof. For these reasons, the amount of supplemental support required could be reduced. Support densities for past longwall recoveries utilizing similar types of concrete supports in the entries varied from a high of 756 psi utilizing fibre cribs to a low of 116 psi utilizing donut cribs. Due to the amount of coal adjacent to the area, the design support density was 68 psi for all of the entries. Actual support densities were slightly higher (72 psi) due to the reasons mentioned above.

3 Way Intersections

There were ten (10) three way intersections. Providing proper support to the intersections was viewed as critical to the success of the longwall. As the longwall face approached from a thirty degree angle, a large area was opened up at each intersection. Additional support would be placed in each intersection to insure that the roof stayed up as the longwall approached. Design support density for these areas was 214 psi with actual support densities approaching 256 psi.

4 Way Intersections

There were only two 4 way intersections in this area. These critical intersections were supported with a design of 255 psi. Actual support density was 305 psi. It is noteworthy that as the longwall mined into the first of these four way intersections, a vast area was opened up. A linear distance over 80 feet long was open in front of the shields. The concrete cribs maintained good roof control through-out this entire area. The yieldable

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pillows were put to a test.

Performance

As the longwall approached the inby end of the angled cut through on the 2 North tailgate side of the longwall panel, roof conditions in the tailgate deteriorated. Cutter roof and a small roof fall occurred on the side of the cribs away from the longwall panel coal pillar. The cutter roof continued to advance in front of the longwall face and crossed to the coal pillar side of the cribs between the first and second of the angled entries. Additional wood cribs were added to the tailgate and polyurethane was injected into the lead cut-through entry where the cutter turned the corner and combined with a cutter that was already in this area. This was done to prevent further deterioration of the roof in the lead cut-through entry.

A total of 22 cutable cribs were monitored for deformation of the Tekset bag and the cement pillar, as illustrated in figures 3 and 4. The longwall shearer cut through the initial cutable crib without any problems. This cutable crib and some of the other nearby cutable cribs had taken weight and the pillows were yielding. The core of these cutable cribs remained intact until they were cut by the shearer. Instrumented cutable crib 16 located in the tailgate entry had taken a large amount of weight and the outer portions of this cutable crib had broken off. The Tekset bag on this cutable crib yielded over 5 inches due to the weight in the tailgate, yet it continued to provide support to the roof through its core which remained intact and experienced little to no deformation (fig. 5).

The longwall continued to mine through the entries, mining out the cutable cribs without problems. Some delays in production occurred as the cutable cribs were mined through. These delays were primarily due to cutting slow and attempting to mine as much of the cutable crib as possible (fig 6). A few of the cutable cribs were cut through on the top half and not the bottom half. These rolled onto the panline causing problems at the headgate crusher entrance. One other problem occurred on the surface at the scalping screen causing short delays. The form bags and pillow bags blocked the screens to the point where the reclaim belt had to be turned off and the screens cleaned so that the cleaning plant could operate.

The roof was allowed to yield slightly due to the design of the pillows. The cutable cribs adjacent to the tailgate experienced between 1 and 1.5 inches of Tekset bag deformation and little to no flyash pillar deformation (figs. 7, 8, and 9).

The mid-panel cutable cribs experienced less than 1 inch of Tekset bag deformation and little to no flyash pillar deformation (figs. 10, 11, 12, and 13). The roof

conditions remained excellent throughout the entire time period it took to mine through all three recovery entries. Rib conditions continued to deteriorate as the longwall approached and mined through an area. This did not pose a problem for the longwall equipment or personnel.

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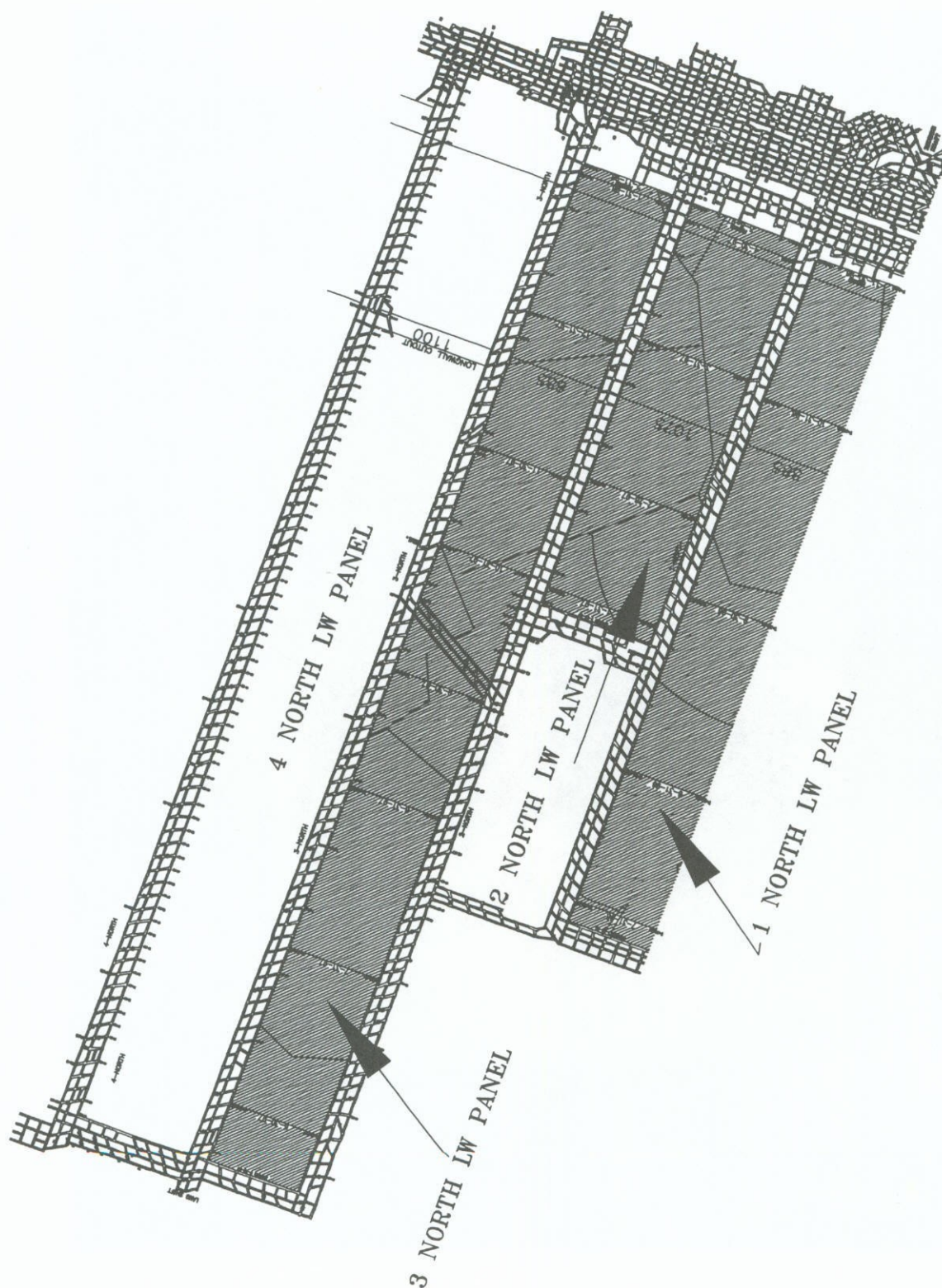


Figure 1. Map of the 84 Mine in the area of the 3 North panel, the three cut through entries are located mid-panel.



Figure 2. Completed 48 in diameter cuttable cribbing, in an approximately 10 ft high entry.

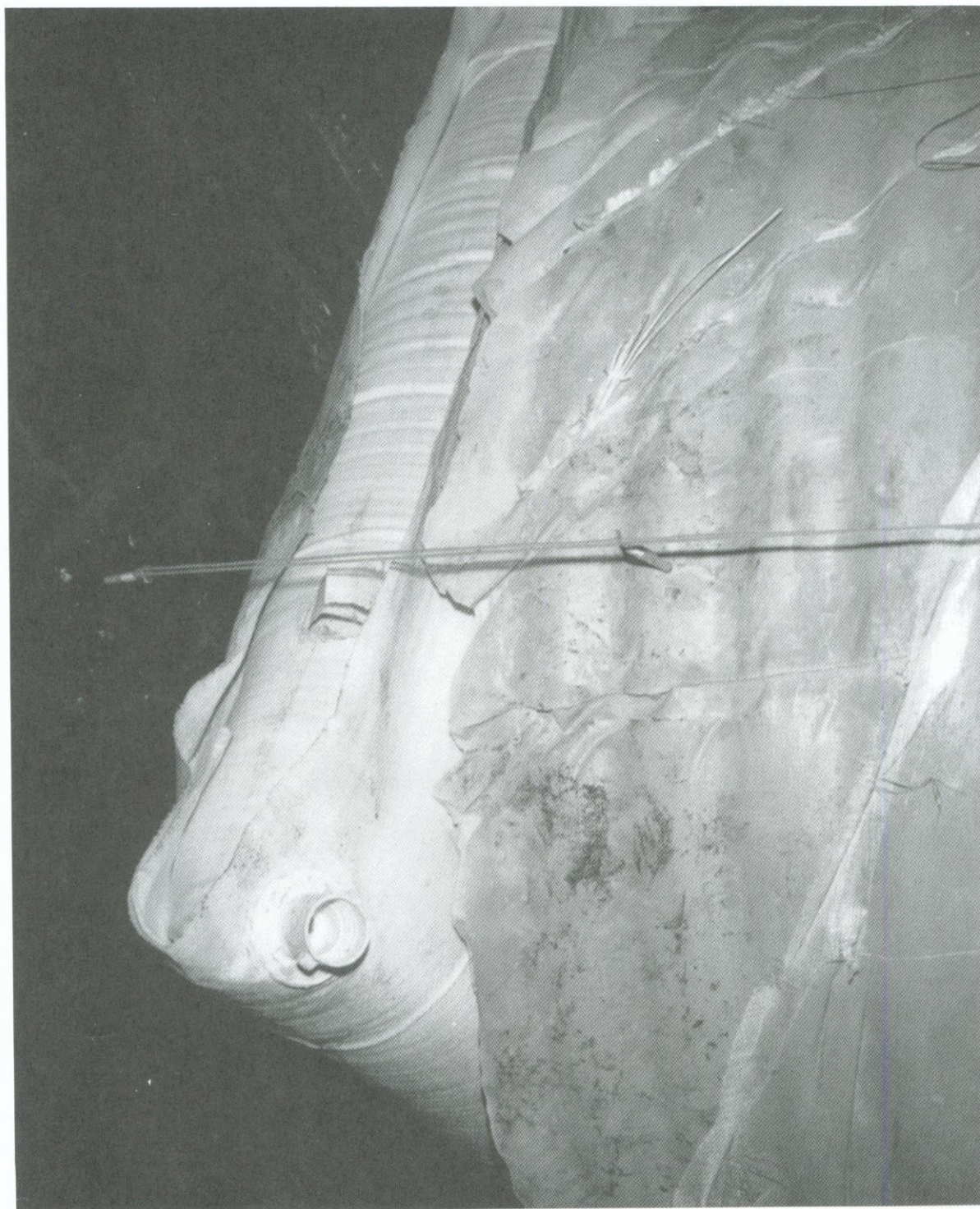


Figure 3. Tekset bag deformation monitoring system.

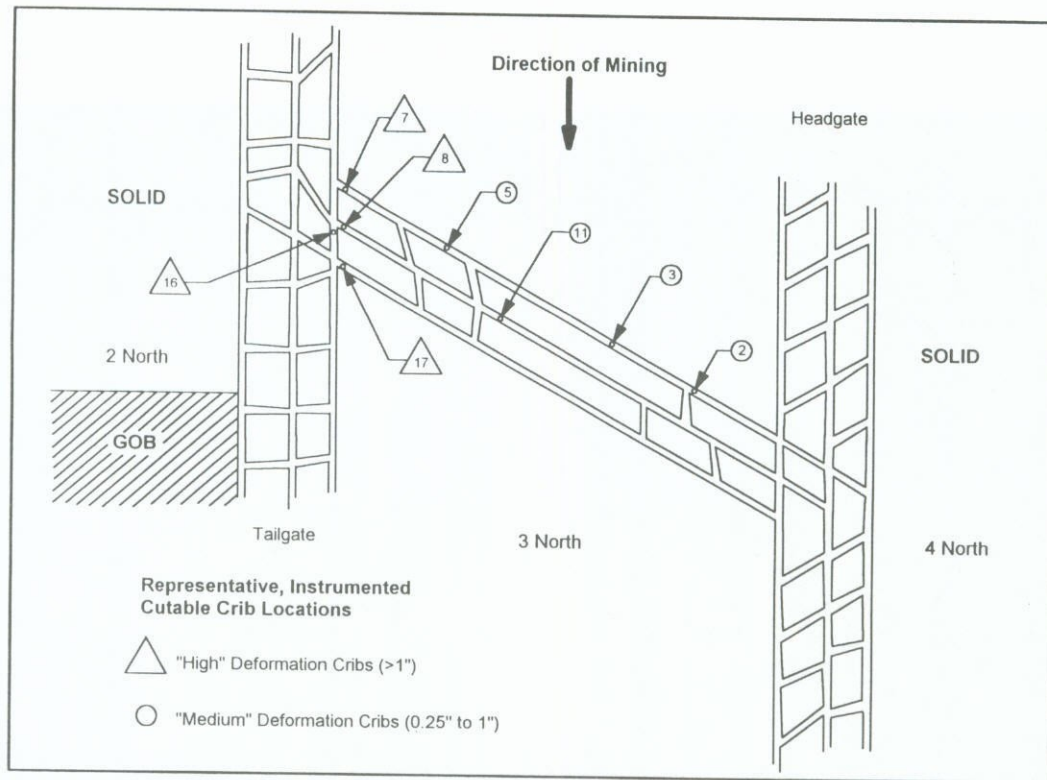


Figure 4. Map of representative instrumented cuttable crib locations.

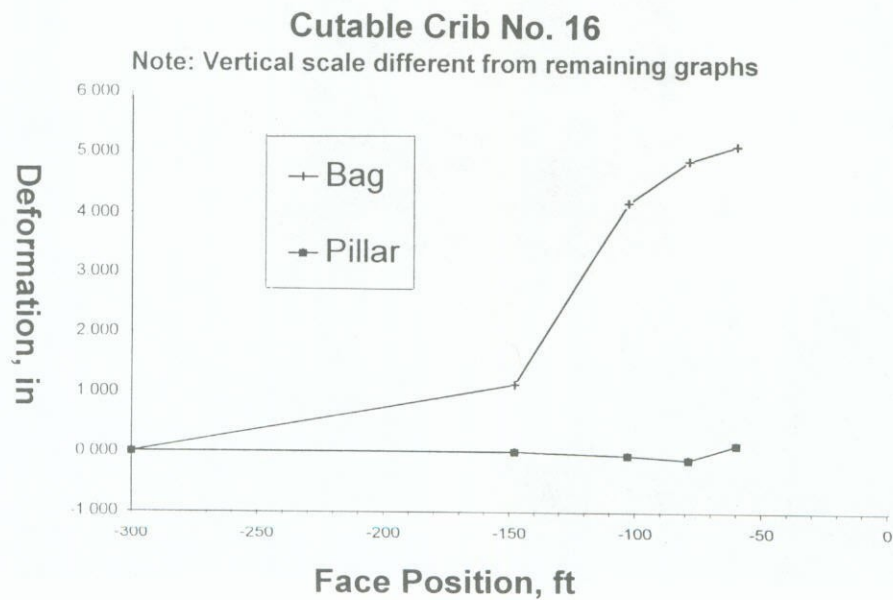


Figure 5. Deformation of instrumented cuttable crib 16, located in tailgate entry in area of roof instability.



Figure 6. Half mined cuttable crib directly in front of the longwall shields.

Cutable Crib No. 7

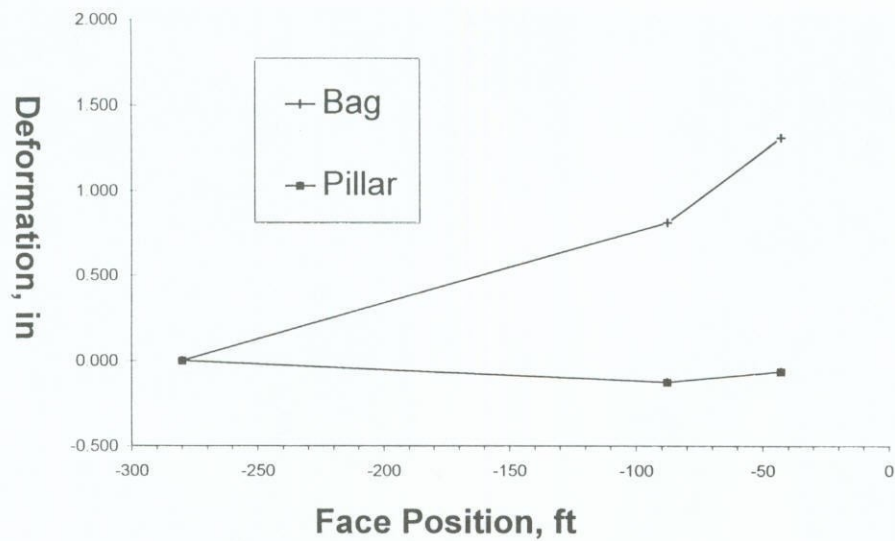


Figure 7. Deformation of instrumented cutable crib 7, located on the tailgate end of the lead cut through entry.

Cutable Crib No. 8

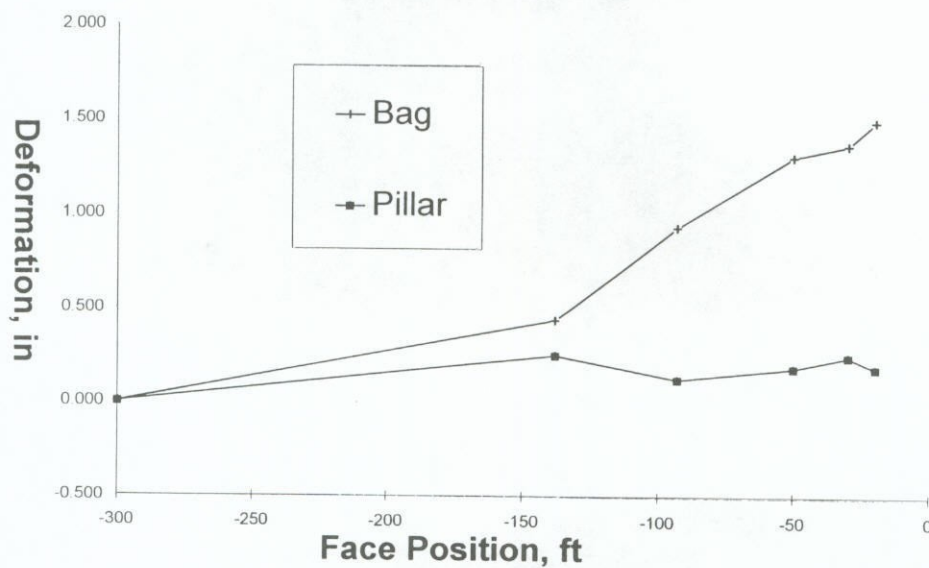


Figure 8. Deformation of instrumented cutable crib 8, located on the tailgate end of the middle cut through entry.

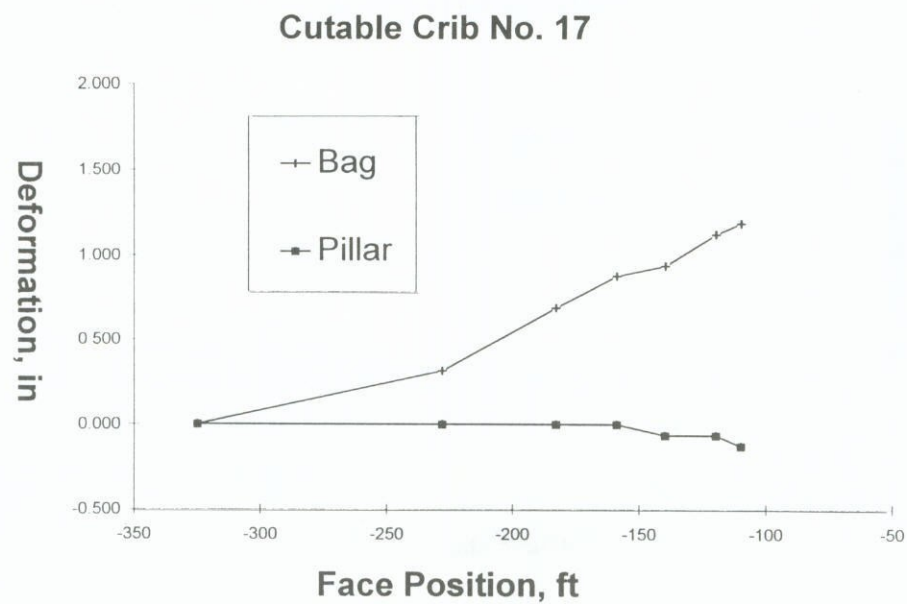


Figure 9. Deformation of instrumented cutable crib 17, located on the tailgate end of the final cut through entry.

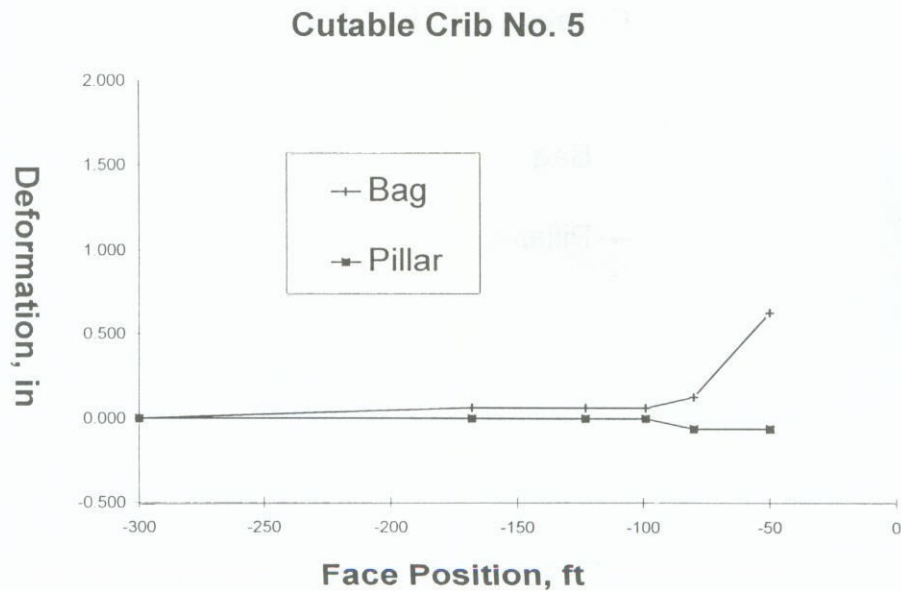


Figure 10. Deformation of instrumented cutable crib 5, located mid-panel in the lead cut through entry.

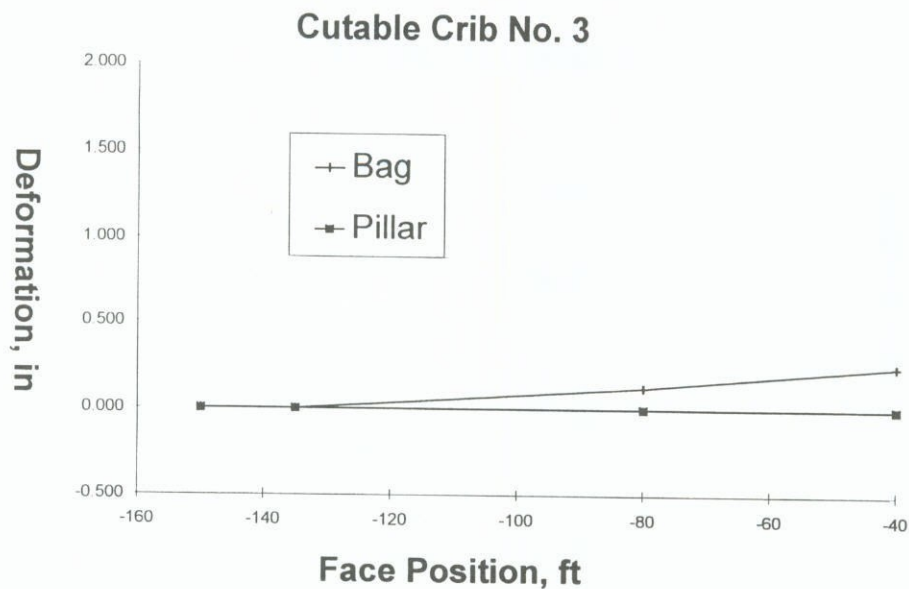


Figure 11. Deformation of instrumented cutable crib 3, located mid-panel in the lead cut through entry.

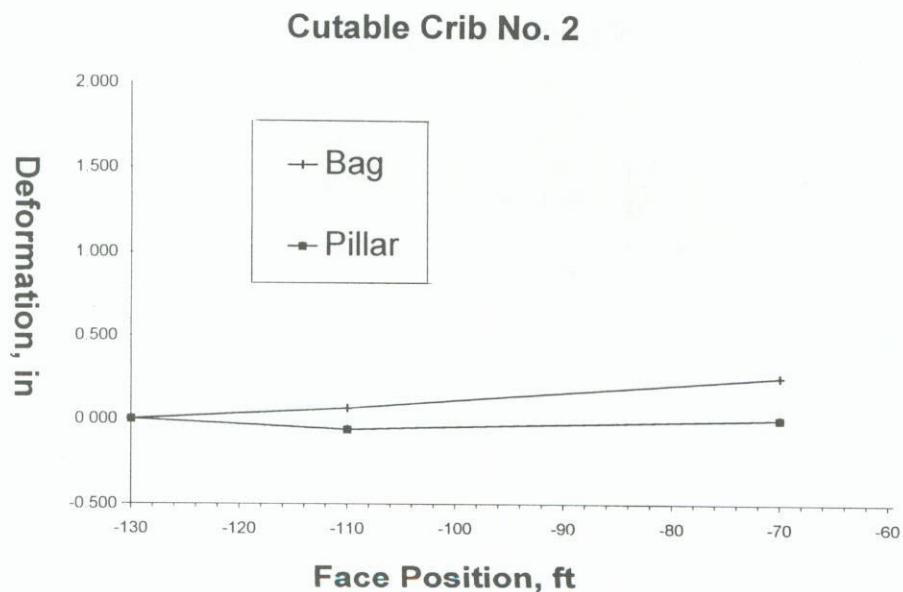


Figure 12. Deformation of instrumented cutable crib 2, located in a crosscut near the headgate in the lead cut through entry.

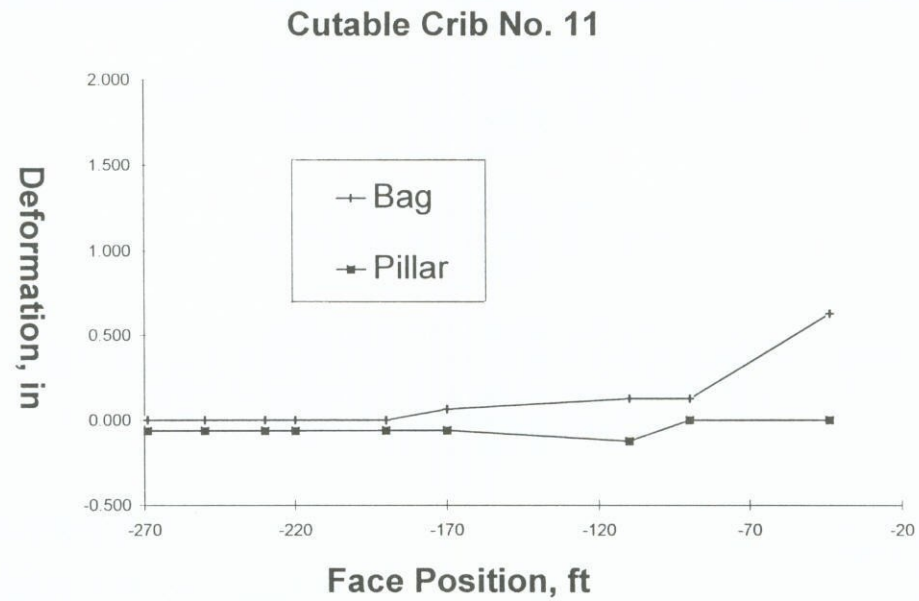


Figure 13. Deformation of instrumented cutable crib 11, located mid-panel in the middle cut through entry.

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International Experience with Longwall Mining into Pre-driven Rooms

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ABSTRACT

Unusual circumstances may require that a longwall retreat into or through a previously driven room. The operation can be completed successfully, but there have been a number of spectacular failures which exposed miners to serious roof fall hazards. To help determine what factors contribute to such failures, the National Institute for Occupational Safety and Health (NIOSH) compiled a comprehensive international database of 130 case histories. The cases include five failures where major rock falls occurred in front of the shields, and six even more serious failures involving major overburden weighting. This suggests two room failure mechanisms. The first is a roof fall type failure caused by loading of the immediate roof at the face as the fender narrows. The second is a weighting type failure caused by the inability of the roof to bridge the recovery room and face area, and affecting rock well above the immediate roof. The data indicate that the roof fall type of failure is less likely when intensive roof reinforcement (bolts, cables, and trusses) is employed together with higher-capacity shields. The overburden weighting failures, in contrast, occurred when the roof was weak and little standing support was used.

Acknowledgments

The authors would like to thank Thomas M. Barczak of the NIOSH, Pittsburgh Research Laboratory, for providing the data required for determining standing support load capacities.

INTRODUCTION

Although not standard industry practice, pre-driven longwall recovery rooms and cross panel entries have been used in a number of mines for various reasons, including:

- to speed up recovery of the longwall upon completion of a panel;
- with "super" longwall panels, to mine through entries that were driven mid-panel to facilitate ventilation and escape;
- to extract old barrier pillars that may include crosscuts or crossing entries, or;
- when a geologic feature, such as a dike or a fault, has been removed prior to longwall mining.

Experience has shown, however, that there are serious ground control risks with the procedure. Several spectacular failures have occurred, where rock falls or severe weighting pressures on the shields required weeks or even months to work through. During these incidents, miners were exposed to extremely hazardous conditions while working in very confined spaces.

To help prevent such failures in the future, NIOSH has compiled a comprehensive database of all known examples of longwalls mined into or through pre-driven rooms. A total of 130 case histories have been collected from 17 mines in the U.S., Australia, and South Africa, covering the period from the mid-1980's through 1997. The data were obtained primarily from the literature, supplemented by personal communications and experience where applicable. For each case history, every effort was made to obtain information on the geology, the dimensions of the pre-driven room, the support installed, and the results. Table 1 shows the complete database. Some further details on the mines and case histories is provided in the paragraphs that follow.

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Table 1.-Data base of parameters used in analyzing the performance of longwall mine throughs of pre-driven rooms.

Country	State	Reference ¹	No. Of Rooms	Soft Floor ²	Depth m	CMRR	Seam Ht, m	Panel Width, m	Room Length, m	Shield Capacity ³	RDI ⁴ MPa-m	Standing Sup. MPa	Slow Mining ⁵	Outcome ⁶
US	PA	1,2,3,4	1	N	150	40	2.4	244	61	454	0.37	5.6	N	1
US	PA	1,2,3,4	1	N	150	40	2.4	183	183	454	0.43	5.6	N	1
US	PA	1,2,3,4	1	N	150	40	2.4	183	183	454	0.37	4.1	N	1
US	PA	1,2,3,4	1	N	150	40	2.4	183	183	454	0.37	2.8	N	1
US	PA	12	3	N	168	40	2.9	270	270	794	0.22	1.5	N	1
US	PA	7	3	N	210	40	2.2	250	250	635	0.53	1.8	N	1
US	PA	7	3	N	210	40	2.2	305	305	635	0.29	4.8	N	1
US	PA		1	N	220	40	2.2	305	305	635	0.88	0.0	N	3
US	PA		1	N	220	40	2.2	305	305	635	0.72	1.8	Y	1
US	MD	5,6,19	16	N	190	40	2.6	229	229	599	0.33	1.2	Y	1
US	MD	5,6,19	1	N	190	40	2.6	229	229	599	0.33	1.2	Y	2
US	MD	5,6,19	6	N	190	40	2.6	229	229	599	0.66	4.6	N	1
US	CO	13,14	1	Y	140	35	2.1	168	168	590	0.62	0.3	Y	3
Aus	NSW	8	1	Y	90	60	3.1	200	200	590	0.64	0.1	N	1
Aus	NSW	8	6	Y	90	60	3.1	200	200	590	0.75	0.0	N	1
Aus	NSW	8	1	Y	50	82	3.4	200	200	590	0.75	0.0	N	1
Aus	NSW	8	4	N	290	50	3	200	200	590	1.83	0.4		1
Aus	NSW		1											
Aus	NSW		1	N	275	45	3	150	150	617	0.00		Y	3
Aus	NSW		3	N	275	45	3	150	150	617	0.93	0.1	N	1
Aus	QLD	15	1	N	190	50	2.4	200	200	726	0.76	0.1	N	3
Aus	NSW		2		400	70		200		907				1
Aus			several					50		363				1
S. Africa			1					200	150					1
S. Africa		9,10,11	1	N				200	200					2
S. Africa		9,10,11	4	N	125	50		200	200	327		3.5	N	1
S. Africa		10,11	1	Y	70	35	3	200	100	327	0.55	0.0	Y	3
US	WV	18	1	N	305	50	1.5	244	244	590	0.55	0.1	N	1
US	WV	18	1	N	305	50	1.5	244	244	590	0.42	0.1	N	1
US	WV	18	6	N	305	50	1.5	244	244	590	0.42	0.0	N	1
US	WV	18	6	N	305	50	1.5	244	244	590	0.42	0.0	N	1
US	AL	17	3	N	366	67	2.1	265	265	726	0.13	0.7	N	1
US	AL	17	3	N	366	67	2.1	265	265	726	0.41	0.3	N	1
US	AL	17	12	N	366	67	2.1	265	53	726	0.41	0.0	N	1
US	AL	16	1	Y	610	47	2.3	107	107	590	0.15	0.2	N	3
US	AL	16	3	Y	610	47	2.3	107	107	590	0.15	0.7	N	1
US	AL	16	1	N	610	57.5	2.5	76	76	590	0.15	0.0	N	2
US	AL	16	1	N	610	57.5	2.5	76	76	590	0.15	0.1	N	2
US	AL	16	16	N	610	57.5	3.0	76	76	590	0.29	0.1	N	1
US	AL	16	2	N	610	68	3.0	122	67	590	0.09	0.0	N	1
US	AL	16	6	N	610	68	3.0	122	67	590	0.32	0.0	N	1
US	AL	16	1	N	610	68	3.0	122	67	590	0.32	0.2	Y	2

¹References. Refers to the numbers of the references at the end of this paper.

²Soft Floor. Y=Soft. N=Normal or not noted as soft by the original source.

³Metric tons.

⁴RDI. Reinforcement Density Index. The product of the support capacity and the support length, divided by the tributary area affected by the support and summed for all support types. In the case of trusses the length of one anchor is used. The index does not apply to standing supports.

⁵Slow Mining. Y=Slow mining. N=Normal or rapid mining or rate unknown.

⁶Outcome. 1=Successful outcome. 2=Failure due to face break or roof fall. 3=Failure due to major overburden weighting

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U.S. CASE HISTORIES

approximately 15% wood.

Alabama Mine "A": This mine has gained considerable experience with mining through pre-driven rooms in recent years (Hendon, 1998). Successful mine-throughs include:

- 18 crosscuts extracted with a 75 m (250 ft) wide longwall face;
- two "probe entries" driven across a 120 m (400 ft) wide face; and,
- a number of 42 m (140 ft) crosscuts inside the same 120 m (400 ft) face.

In each of these cases, the face entered the pre-driven room at an angle, generally about 7 degrees. Relatively little additional bolting was used to reinforce the roof, which was usually competent siltstone. Standing support consisted of, at most, a single row of fiber cribs on 6 m (20 ft) centers.

There was also one notable failure. At the "pull-out crosscut" of the same 120 m (400 ft) face, a "massive roof fall" occurred at mid-face which required 2 weeks to clean up. This was the most heavily supported of any of the mined through entries, with a double row of propsetters installed on 1.5 m (5 ft) centers. However, the other difference was that the wall approached the pull-out crosscut much more slowly to facilitate meshing. It was concluded that "substantial standing support was needed at the pull-out point where the face retreat rate was reduced significantly" (Hendon, 1998).

Alabama Mine "B": A longwall was used to extract a 110 m (350 ft) barrier pillar which was crossed at right angles by a set of four main entries (Hendon, 1998). The first entry was supported by double rows of propsetters, but a "massive squeeze" developed as the last coal was removed from the fender. The shield canopies were forced onto the face conveyor and 1 month was required to get the longwall moving again. The remaining three entries were reinforced with double rows of fiber cribs and propsetters, and were extracted without incident. The roof consisted of 1.5 m (5 ft) of mudstone and coal, overlain by competent siltstone.

Alabama Mine "C": Partial recovery rooms, 45-60 m (150-200 ft) long, have been used for many years at this mine (Stansbury, 1998). These have been located either in the middle or near the gate ends of the panels. Relatively light roof reinforcement, and no standing support, has been sufficient in the partial recovery rooms.

When a recent panel was extended through three pre-existing entries, the decision was made to fill them with a 0.7 MPa (100 psi) cellular concrete. There were no strata control incidents, but other complications made the experience unsatisfactory.

Most recently, cable trusses and concrete pilasters were employed in a full-face recovery room. The coal fender punched into the floor, there was significant shield convergence, and numerous pilasters crushed out, but the face was recovered on schedule. The pilasters were built of solid concrete blocks with

Colorado: On one of the early longwall panels at this mine, a decision was made to recover the longwall from a sub-main entry. The room was supported by a single row of square fibercrete cribs topped with 0.15 m (6 in) wood blocks (Ropchan, 1990). Roof reinforcement included 2.4 m (8 ft) fully grouted rebar on 0.65 m (2 ft) centers with chain link fence. The immediate roof consisted of a weak, highly slickensided shale about 3-4 m (10-12 ft) thick, overlain by a series of weak siltstones, sandstones, and shales. The shale-siltstone floor was also weak.

When the panel fender was between 1-2 m (3-6 ft) wide, the face advance stopped for 6 hours because the pan line was stuck. The roof began to converge rapidly as the fender crushed, and many shields yielded with several becoming iron bound. The pillars in the recovery room punched into the roof with heavy rib spalling and cutter roof failure. All the fiber cribs failed either by splitting or crushing with many showing a hour glass failure configuration. The recovery room was then heavily reinforced with wood cribs, though convergence continued.

The subsequent investigation concluded that the roof had broken at the pillar line with the rock mass moving toward the face as shown in figure 1 (Pulse, 1990). Investigators noted a "tensile fracture" which resulted in the "roof moving toward the gob." With the roof beam apparently pivoting around the pillar rib, the largest roof movements were experienced at the face, and a significant amount of the abutment load had shifted onto the shields. Under such conditions the face could not move the last few feet into the room.

Recovery of the longwall was accomplished where the face had been halted short of the recovery room. The remnant fender was then mined out with a continuous miner after the roof was grouted and heavily supported. Approximately 2 months were required to recover the longwall face.

Maryland: This mine has used 23 recovery rooms, 6 of which were 11 m (36 ft) wide. The only failure was one of the standard rooms where roof falls necessitated 2 weeks of remedial action. The extra support in this case was a row of concrete cribs on the inby side of the recovery room and a row of wooden cribs on the outby side. The damage occurred when the face was 11 m (35 ft) from the room. Mining rates were slow because of wire meshing activity.

The wide recovery rooms were designed so that the face would not have to slow down for meshing (Wynne et al., 1993). The room was developed and supported in two passes. Supplemental support included eight rows of concrete donut cribs, three rows of truss bolts, and two rows of 5 m (16 ft), 25 mm (1 in) diameter roof bolts. All the wide rooms were reportedly mined without serious incident (Wynne, 1998).

Pennsylvania Mine "A": A total of four recovery rooms were successful at this Pittsburgh seam mine (Bauer et al., 1988; Bauer et al., 1989). Three different types of concrete supports

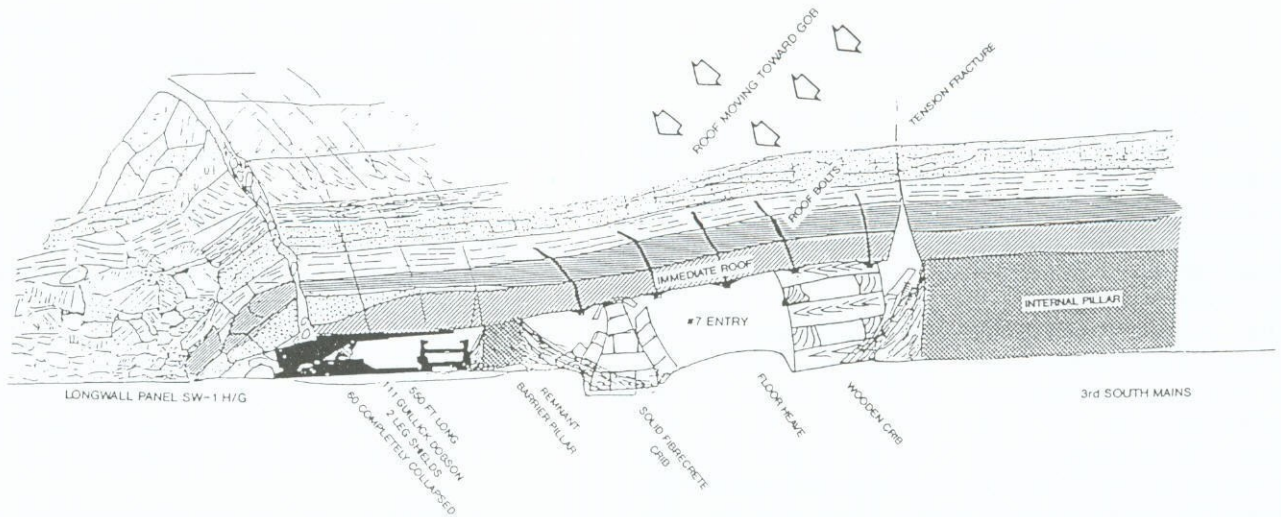


Figure 1, Typical physical behavior of a pre-driven room under weighting type roof failure. From underground observation of the Colorado recovery room failure. Note the fracture observed to develop at the front edge of the abutment pillar. (From Pulse, 1990)

were used. The first room employed 1.2 by 1.8m (4 by 6 ft) flyash concrete piers. The piers were designed to "replace the load-bearing capacity of the coal" by providing a support resistance of 5.5 MPa (800 psi). To reduce costs, fibercrete cribs with 0.3 m (1 ft) wood cap blocks were used on the next room. Again, the cribs were placed to give a support density of 5.5 MPa (800 psi). The concrete cribs were hard on the shearer and stageloader, however, and so the last two panels employed poured concrete cylinders. The concrete was pumped from the surface to fill 1 m (3 ft) diameter cardboard tubes. The top 0.2 to 0.3 m (8 to 12 in) above the concrete cylinders was wedged with wood. The support density was reduced on the fourth panel following good results from the third. No ground control problems were encountered in any of the recovery rooms.

Pennsylvania Mine "B": Three different techniques have been used at this Pittsburgh seam operation. Little information is available on the results from the first three 4.9 m (16 ft) wide recovery rooms. The mining company considered the rooms successful, but did not have full confidence in the technique because of floor heave and the failure of some of the donut cribs and several later panels were recovered short of prepared rooms, using conventional techniques.

More recently, a set of three entries were driven across a longwall panel so that the panel could be lengthened (Chen et al., 1997). The entries were filled with a low strength cement-flyash mixture. There were few ground control problems in mining through the entries, although the wood and steel left in the rooms caused equipment problems.

A full-face pre-driven recovery room was less fortunate. No standing support was used, though the roof was heavily reinforced with 2.4 m (8 ft) mechanically-anchored resin-assisted bolts, 3.7 m (12 ft) cable bolts, mesh, and channels. The mining rate averaged more than 15 m/d (50 ft/day) as the panel approached the room. When the face was 3 m (10 ft) away, the room began to deteriorate and a number of shields were

constantly on yield. The face entered the recovery room at the headgate and tailgate areas, but the roof converged to the floor over much of the entire mid-panel section. It took several weeks to get through the collapsed roof, with remedial efforts including the use of polyurethane grout and the installation of cribs.

The face was then advanced to a second entry that had been mined at the same time as the original recovery room. This room was supported by two rows of donut cribs, a row of 0.76 m (30 in) wooden cribs, and cable bolts. The longwall recovery was successful, although the donut cribs were heavily damaged and a large amount of convergence took place.

Pennsylvania Mine "C": A three-entry system was driven across one panel to allow it to be extended beyond an adjacent, shorter panel. The entries were driven at a 30 degree angle to the panel, each supported by a single row of 1.2 m (4 ft) diameter poured cement cribs on approximately 2.4 m (8 ft) centers (Bookshar et al., 1998). The body of the cribs was made up of a stiff high strength concrete, and the cribs were then topped with a plastic bag approximately 0.3 m (1 ft) thick and filled with a yielding proprietary cement. The mine throughs were successful, with the largest measured deformations less than 80 mm (3 in) in the tailgate.

West Virginia: Eleven faces have been recovered using full-face recovery rooms at this mine (Smyth, 1998). The first two used some standing support, but most have used just roof bolts and cable bolts for reinforcement. The coal is quite thin, leaving little room for convergence. While some shields have been stuck, most face recoveries have been conducted without incident.

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AUSTRALIAN CASE HISTORIES

New South Wales (NSW) Mine "A": Recovery rooms were used on 12 panels in 3 different coalbeds at this mine (Simpson et al., 1991). On the earlier panels in the Fassifern coalbed 1.8 or 2.1 m (6 or 7 ft) resin bolts, "w" straps and mesh were used for support. Standing support, consisting of two rows of timber props was used only on the first panel. The rooms were 4.8 m (16 ft) wide, except the first, which was only 4.2 m (14 ft) wide. A panel was also recovered in the shallower Great Northern coalbed using the same configuration. When the first recovery room was planned for the deeper Young Wallsend coalbed, three rows of 10 m (33 ft) fully grouted cable bolts were added, (at least near the headgate, the published report is unclear on whether cable bolts were used across the entire room) and a row of 1.8 m (6 ft) diameter standing supports known as Big Bags were installed on 5 m (16 ft) centers near the outby rib.

The mine management attributed the success of the recovery rooms to a large extent to the presence of a soft claystone floor which allowed the panel fenders to be slowly punched into the floor, with manageable floor heave, and avoiding fender yield. They reported that the fenders typically did not yield until the last few meters and in some cases did not yield at all. In the deeper Young Wallsend coalbed, this mechanism was not relied upon, the room was heavily supported by both secondary and standing support, even though the shale floor there was also soft. The sandstone or conglomerate immediate roof above the Fassifern, Great Northern and Young Wallsend probably also contributed to the success of the recovery rooms at this mine. The mine preferred to keep the tailgate back, as much as 8 m (26 ft) in the Young Wallsend coalbed, so that if ground control problems occurred, they would be less likely to affect the entire panel.

NSW Mine "B": Four rooms were mined into, the first being a full face recovery room 6.5 m (21 ft) wide and the next three being narrow (3.5 m or 12 ft) full face entries driven for ventilation. The recovery room failed, causing the shields to go on the solid and several months were required to recover them. The room was supported by a standard primary bolting pattern, with spot cable bolting only in areas considered critical, such as gate road and chute intersections. No standing support was used. The motor on the shearer ranging arm broke down when the face was just a few meters from the room and the face remained idle for an extended period. The fender failed when the face was 2 m (6 ft) from the room. High water flows from the gob and gas flows into the tailgate entry were noted in the course of the mine through and interpreted as the effect of the failure and subsequent weighting of an overlying sandstone aquifer and the opening of fractures in failed rock to allow gas flow.

The remaining mine throughs were supported by cable bolts, props and glue injection into the panel fender. Primary support consisted of 2.4 m (8 ft) bolts and "w" straps on a 0.8 m (2.5 ft) spacing, 10 m (33 ft) cable bolts on a density of 3 cables/2 m of entry and 3 rows of 150 mm (6 in) props on 0.8 m (2.5 ft) centers, over the entire length of the entry. These mine throughs were successful, although there was no necessity to remove the shields.

NSW Mine "C": A recovery room was used to successfully recover a single longwall at this mine. No specific information is available on the supports or on the panel geometry, but it is known that significant secondary support in the form of cable bolts and timber props was used.

NSW Mine "D": Recovery rooms were used on two short, 200 m (660 ft) wide panels at this mine. The immediate roof in both cases is the massive, competent Coalcliff Sandstone. Both mine throughs were considered completely successful. No information is available on the type of support used in the rooms in either case. Recovery rooms were not used on subsequent panels because the roof lithology changed from competent sandstone to shale and because later panels were also significantly longer reducing the importance of rapid face moves.

NSW Mine "E": No information is available except that several panels were recovered, the panels were narrow, about 50 m (164 ft) and they were successful.

Queensland: A single 5.2 m (17 ft) wide recovery room was attempted at this mine (Klenowski et al., 1990). Primary roof support consisted of five rows of 2.1 m (7 ft) resin bolts on 1.5 m (5 ft) spacings with "w" straps. The inby and outby bolts were angled over the fender and barrier pillar. Two rows of 8 m (26 ft) cable bolts were also used on 4 m (13 ft) centers. Grouted 1.8 m (6 ft) fiberglass rib bolts were installed in the fender and in the barrier pillar. Standing support was only planned for use on an as required basis. The design was based upon the results of an instrumented 15 m (50 ft) stub entry.

Convergence was noted to begin to accelerate when the fender was 6 m (20 ft) wide. When the panel was 5 m (16 ft) from the room the fender failed and the shields began to continuously yield. Just prior to entering the room a maximum of 0.42 m (17 in) of convergence took place at one place on the face where the shearer was hung up under the canopy of a shield and typical convergence may have been 0.26 m (10 in). Less convergence took place in the recovery room. When the shields entered the room the mining height was greater and the available hydraulic fluid was insufficient to set them (due to fluid losses from continuous yielding of the shields). A delay took place until the hydraulic reservoir was refilled. The convergence continued after the room was entered and eventually it was necessary to set timber props, with a total of 392 props finally being used. Of the 440 shield legs on the face, 104 were found to have failed after the mine through due to malfunction of the gas yield valves. One conclusion arrived at by the mining company was that had the shields not failed, the roof convergence would not have been so large and fewer timber props would have been required. However, it is possible that the high rates of convergence caused the damage to the shields.

After-the-fact, back analysis indicated that a "tensile failure" extended upward to a rider seam located 16 m (50 ft) above the German Creek Seam (figure 2). Convergence at the face occurred much more rapidly at the face than in the recovery room, indicating that "the pivot point of the failing roof beam was over the barrier pillar" (Klenowski et al., 1990).

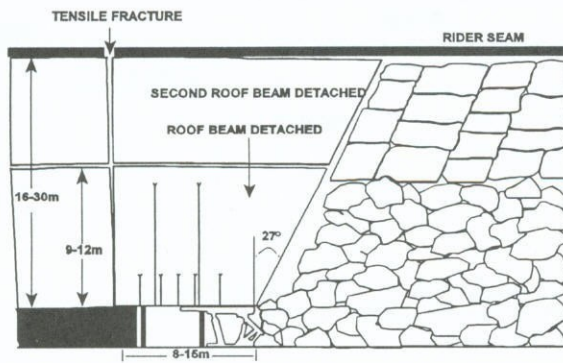


Figure 2.-Conceptual model of a weighting type room failure, based on observations and measurements at the Queensland recovery room failure (after Klenowski et al., 1990).

SOUTH AFRICAN CASE HISTORIES

South Africa Mine "A": Six mine throughs were attempted, five to mine through dikes and an experimental recovery of part of a face (van der Merwe, 1988; 1989). The dike mine throughs used either mat packs (rafts of timbers wired together and laid one on top of the other) or filled entries, except for the third mine through where only timber props were used. For the first and second mine throughs mat packs were used, for the third timber props, for the fourth a back fill material consisting of cement, plaster and sand, and sifted ash as well as timber props, and for the fifth a mix of coal fines and other unspecified materials used to improve the flow characteristics and strength of the mix.

All of the mine throughs except the third were successful. In that case, the room was entered 0.3 m (1 ft) too low and the recovery room eventually collapsed before the dike could be blasted away to allow advance into the room.

The sixth mine through was a recovery room test and only covered a portion of the panel. During the mine through the conveyor belt broke and the face sat idle for 8 hours when the face was 3 m (10 ft) from the room. Apparent water entry from a joint in the fender softened the floor rock and caused the fender to punch into the floor. The floor under the face conveyor heaved and left the face conveyor and shearer too high to allow the shields to be advanced. The recovery room roof also converged until it was impossible to enter, forcing shield recovery in place 3 m (10 ft) from the recovery room, a "lengthy process".

After the event, the conclusion was that "in longwalling, a deviation from standard practice should only be considered if the potential benefits outweigh the potential negative consequences of failure, longwalling is not a very tolerant mining method."

South Africa Mine "B": A room was mined across a longwall face to remove a dike. The roof rock included a 30 m (100 ft) sandstone approximately 30 m (100 ft) above the coalbed. In order to reduce the risks of the mine through the

panel width was reduced some 50 m (160 ft) before reaching the room, from the normal 200 to 150 m (660 to 490 ft). No information is available on the support types used, except that mine management felt that if the technique was used in the future, standing support would be used. The reasons given were to give better resistance to abutment loading and to control the dike, pieces of which tended to fall out while entering the room.

FAILURE MECHANISMS

In the vast majority of cases, longwalls have been successfully mined into pre-driven rooms. Of the 130 cases, only 11 were apparently complete failures. However, the costs associated with these failures, and the hazards they created, were very substantial.

The failures can be divided into two categories. The first includes five cases where the problems were due to roof falls occurring in front of the shields. The second group consists of cases involving severe shield weightings accompanied by major convergence.

Roof falls occurred in two situations as the longwalls approached the pre-driven rooms. At the face, failures developed in the unsupported roof span between the shield tips and the remnant fender (the portion of the longwall panel between the shields and the room) because of an increased span resulting from extensive coal yielding in the fender. In this type of failure, there was insufficient secondary support in position to prevent the roof fall. Roof failures in the recovery room developed because of the high stress and deformation environment and the large span from the shield tips to the abutment pillar once the fender was mined out. Often a substantial portion of the room was involved with this type of failure.

Weighting type failures resulting in the most severe ground conditions were due to main roof action caused by the failure of a weak roof. These types of failures were accompanied by significant room and face convergence where shields were loaded to yield, often becoming iron bound. The development of a new cave break at the edge of the abutment pillar resulted in the loss of the immediate roof cantilever as a supporting element (figures 1 and 2). In the six cases where the weighting failure occurred, accelerated rates of convergence leading to failure initiated when the fenders were 3 m (10 ft) or less in width. Up until then, the remnant panel was providing substantial support to the main roof.

Where insufficient standing support is present in the room, the shields may be called upon to control most of the weight of the new caving block that extends across the recovery room, doubling the roof span they previously supported. As shown in figure 2, the failure may extend up higher into the main roof because of a wider pressure arch causing a further increase in the shield load. Further, the shields are not in good position to handle this new load distribution. As the detached block moves, the main roof is affected. If the main roof is not strong enough to bridge over the detached block it will subside, progressively adding to the loads on the face area. With this type of failure,

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intrinsic support will not resist the failure because the support is internal to the detached block. A strong main roof can help the overburden to bridge across the room and face resulting in more favorable face and room conditions.

CHARACTERISTICS OF PRE-DRIVEN ROOMS

In this study, every effort was made to obtain information on a variety of descriptive characteristics of all the case histories. The results are summarized in Table 1. The goal was to identify those characteristics which correlated with the failed case histories. Statistical techniques, including Pearson correlation and logistic regression, aided the analysis.

Immediate Roof: Descriptions of the roof geology were usually contained in the literature, and were quantified using the Coal Mine Roof Rating (CMRR). In almost every instance, the published description was supplemented by CMRR data collected by the fourth author in past visits to the mines.

A very strong correlation between the CMRR and weighting failure was found. All of the six weighting failures occurred where the roof was relatively weak (CMRR < 50). The correlation is less evident for roof falls (figure 3).

Main Roof: Insufficient data were available on main roof

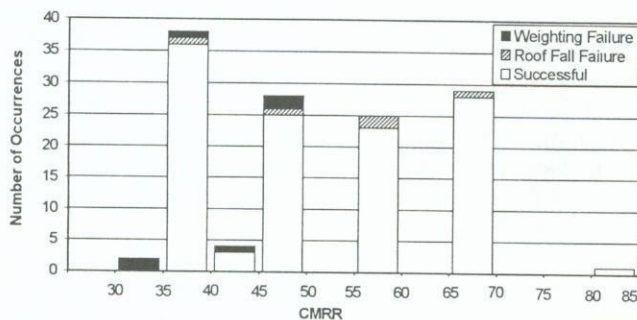


Figure 3.-Histogram of CMRR versus mine through occurrences. All weighting type failures took place with CMRR < 50, while roof fall failures took place over a wide CMRR range

geology for analysis. The authors believe, however, that the main roof geology should be very important to weighting failures. It seems likely that the CMRR is substituting for a characterization of the main roof in many instances.

Floor: Soft floor was reported in 14 of the mine throughs, but these included three of the weighting failures. Under some conditions where the thin, heavily loaded fender is likely to punch into the floor, the potential for failure could be increased, although there were also successful cases where soft floor was credited with delaying fender yield and contributing to the success of the recovery rooms.

Overburden Depth: The case histories cover a very wide range of cover depths. Roof falls were somewhat more likely to occur at greater depth, but not weighting failures (figure 4).

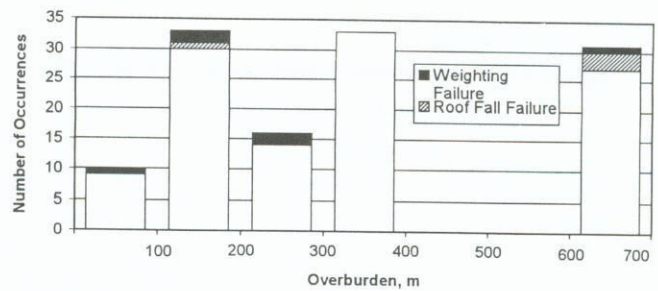


Figure 4. Histogram of overburden versus mine through occurrence. Weighting failures were not strongly correlated to room depth

Seam Thickness: No correlation was found between seam thickness and either type of failure. It is worth noting that while there is less potential for roof weighting when the seam is thin, the tolerance for convergence is also usually much less.

Mining Rate: A slow mining rate seems to be a strong predictor of both types of failures. Twenty mine throughs were associated with reports of slower than normal mining, either because they were meshing, or because a breakdown occurred near the recovery room. Four of the weighting incidents, and two of the roof falls, were in this group. However, it may be that slow mining rates are more likely to be reported when a failure occurred. The rate may actually have been slow in many of the reported successes.

Room Width: There seems to be a little correlation between room width and difficulties. The very widest recovery rooms, the 11 m (36 ft) rooms at the Maryland mine, were trouble free. However, it seems reasonable that overall stability will improve as the room width decreases, all other factors being equal.

Room Length: It might be expected that overburden weightings would occur more often in longer rooms. No such correlation is apparent in these data, however (figure 5). Two of the severe weightings (one in South Africa and the other in Alabama) occurred on faces that were less than 110 m (350 ft) wide.

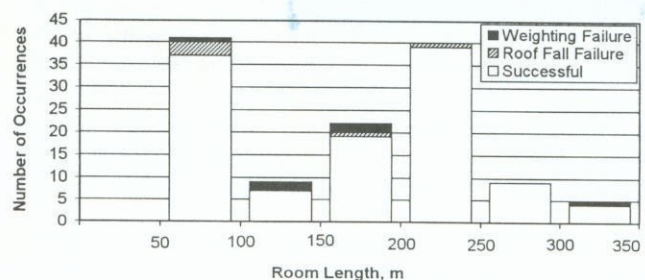


Figure 5.- Histogram of room length versus mine through occurrence

Shield Capacity: Some correlation was observed between shield capacity and roof falls, but heavier shields apparently have not helped prevent weighting failures (figure 6).

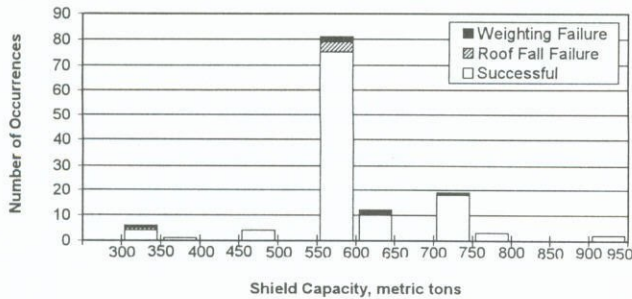


Figure 6.-Histogram of shield capacity versus mine through occurrence

Roof Reinforcement: Roof reinforcement includes all intrinsic supports elements, such as roof bolts, cable bolts, and trusses. The quantitative rating is obtained by determining the load capacity of each element per unit area of roof supported by the element and multiplied by the length of the element. This Reinforcement Density Index (RDI) has the units of MPa·m. Where several types of support were used, the ratings for individual supports are summed.

Heavy roof reinforcement was apparently successful in reducing the incidence of roof fall type failures. Roof reinforcement was apparently not effective in preventing weightings (figure 7).

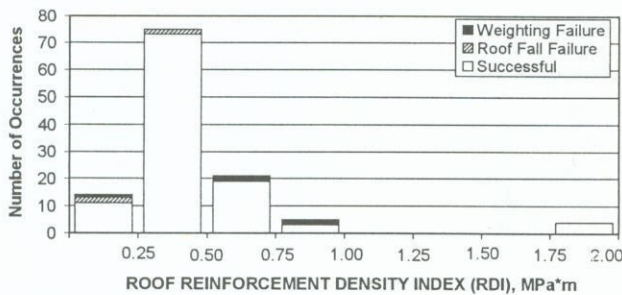


Figure 7, Histogram of Roof Reinforcement Density Index (RDI) versus mine through occurrence

Standing Support: A characteristic of every one of the weighting failures is the lack of standing support (figure 8). In two mines, after a severe weighting failure developed in a room without standing support, adjacent rooms were mined successfully with standing support. These two cases indicate that standing support can be the difference between success and failure. The importance of standing support also lends credence to the proposed mechanism of weighting failure.

Face-Room Angle: Because of the limited data on cases where the room was at an angle to the face, no detailed analysis was conducted on this parameter. However, the mine throughs in the documented cases were successful. As noted earlier, some mines have entered the pre-driven rooms at small angles with the tailgate lagging 6-10 m (20-30 ft) behind the headgate to limit the extent of ground control problems. The evidence that a narrow fender still provides significant support suggests that this approach may be valid.

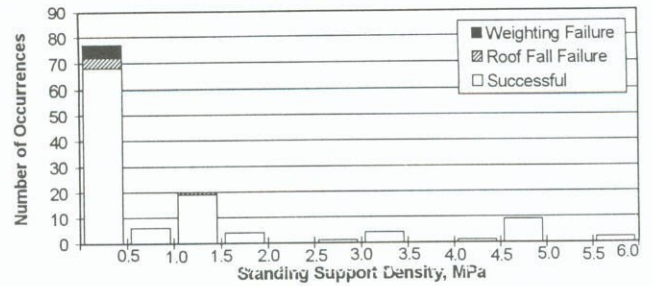


Figure 8.-Histogram of standing support versus mine through occurrence. No occurrences of weighting failures were observed when the standing support density was greater than 0.5 MPa

MULTIVARIATE ANALYSES

Logistic regression was employed to obtain insight into possible design guidelines. Three analyses were performed, analyzing the roof falls, the weightings, and the combined set of failures. Mining rate and weak floor were not included in the analysis because the data were judged incomplete.

Looking at roof falls alone, the two most important variables were shield capacity and the total intrinsic support (RDI or reinforcement density index). It appears that as the shield capacity increases, the required standing support decreases, and vice versa (figure 9). However, the relationship explains only half of the roof failures.

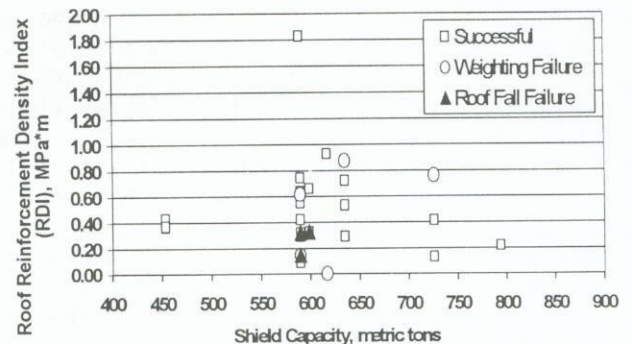


Figure 9.-Shield capacity versus Roof Reinforcement Density Index, for successful mine throughs and cases of roof fall and weighting type failures

For weighting failures, the CMRR and standing support are the two important variables (figure 10). A highly significant relationship indicates that when the CMRR was greater than 50, little standing support was necessary. For CMRR=40, the successful cases used a standing support density of at least 1.0 MPa (145 psi). For CMRR values in the range of 45 to 50, standing support densities as low as 0.5 MPa (73 psi) appeared sufficient to prevent or control weighting failures. However, the cost of standing support is small compared to the cost of a weighting room failure, and the observations made from figure 10 should not be taken as a recommendation to eliminate standing support.

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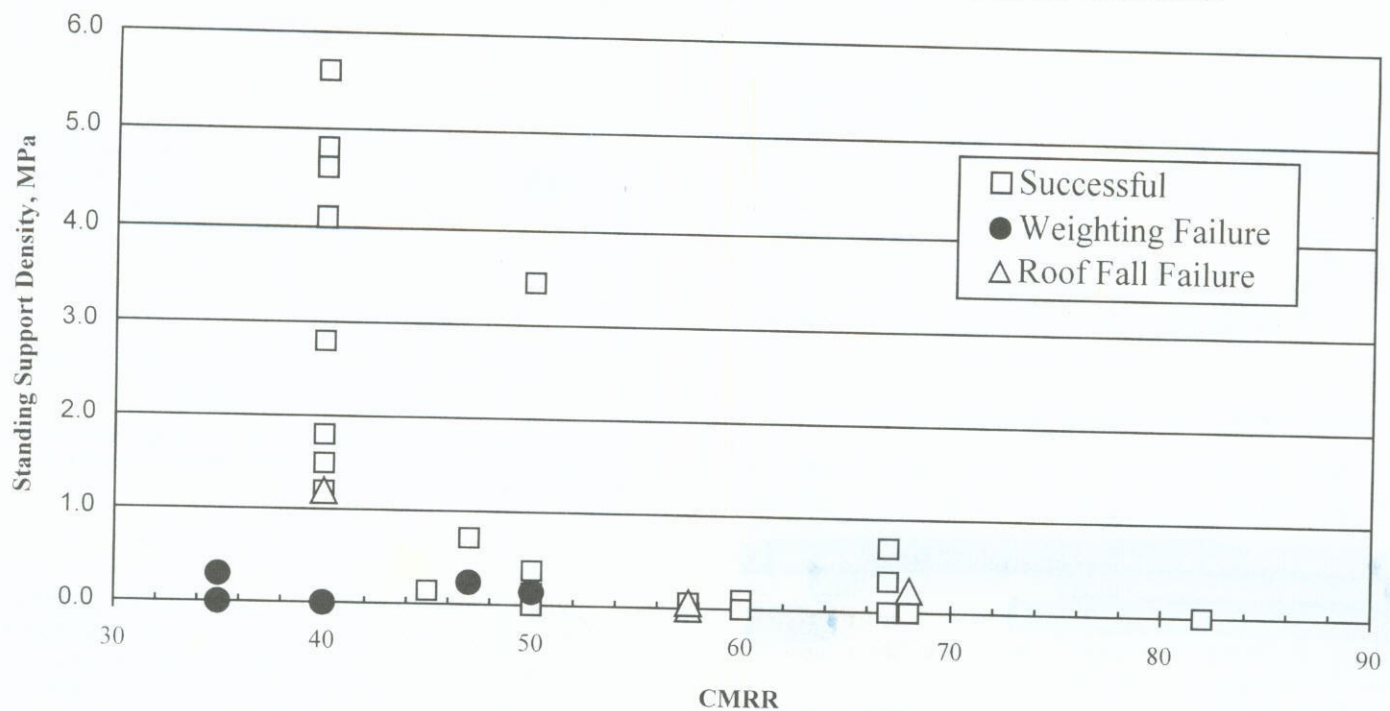


Figure 10.-CMRR versus Standing Support Density, for successful mine throughs and cases of weighting and roof fall type failures. All weighting failures took place with CMRR<50 and standing support density <0.5 MPa.

CONCLUSIONS

As the vast majority of cases attest (92%), longwall mining into pre-driven rooms can be safely accomplished with adequate support for the conditions. However, severe ground control problems from weighting can be encountered where there is weak roof and little or no standing support. Although it appears of little value in preventing weighting failures, intrinsic roof support is of significant value in preventing roof falls. In the cases of weighting failures, the failure involves the main roof and the establishment of a new caving break ahead of the face along the abutment pillar. The costs associated with these failures and the hazards created are substantial. However, even with a weak roof, (CMRR=40), a standing support density of 1.0 MPa (145 psi) appeared sufficient to prevent or control failure and allow the longwall to enter or pass through the rooms. With stronger roof (higher CMRR values) the standing support density required appeared to decrease. This is an observation and should not be considered a recommendation to eliminate standing support solely on the basis of measured CMRR values. Other factors that contribute to this type of failure include soft floors that can result in pillar punching and slow mining rates.

With a stronger roof, where the weighting type failure does not occur, much less standing support or intrinsic roof reinforcement alone have been successful. However, even under these conditions roof falls occur either in the unsupported area between the shields and the fender or in the room itself. In the cases where the falls occur in the rooms, they can be controlled by increased roof reinforcement or by the addition of standing support.

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Analysis of Geologic and Geotechnical Conditions and Their Effects on Longwall Mining to Optimize Mine Planning at Shoal Creek Mine

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ABSTRACT

Mining cost reduction has been a major goal since longwall mining began at the Shoal Creek Mine. Several geological anomalies are encountered at the mine, including unexpected undulations of the mine floor, rapid changes in coal seam thickness, and a massive sandstone channel in the main roof. The geologic anomalies combined with other geologic factors, e.g., faults, joints in the mine roof and cleats in the coal seam, significantly affect daily mining operations and cause ground control problems such as roof and face rock (middleman) failures. This results in a higher mining cost by slowing the face advance and increasing equipment downtime rates. During the last two years, studies have been made on the mine geology to minimize the adverse effects of the geologic factors and to determine the optimum mining plan. Higher production can be achieved with an improved mine plan by optimizing longwall panel orientation, and by predicting severe roof weighting, scheduling planned idle periods on the longwall face, etc. It is believed that maximum utilization of geologic information and geotechnical data will help mine management to accurately evaluate roof behavior to improve operations planning. This paper gives a detailed discussion on effects of the geologic factors on longwall performance at the mine and summarizes the findings on characteristics of mine geology associated with the Shoal Creek reserve in the Warrior Basin.

INTRODUCTION

The Shoal Creek Mine located near Birmingham, AL is a large coal mine with two active longwall panels A-4 and B-4. As shown in Figure 1, the current two 228.6 m (750 ft) wide longwall panels are oriented perpendicular to each other. The Blue Creek seam is the primary source of coal production and ranges from 0.91 to 3.05 m (3 ft to 10 ft) in thickness. The seam is located at a depth of 275 to 440 m (900 to 1,450 ft) below the surface with an approximate 29° southwest dip. The Marylee seam and the parting (middleman) are mined in most of the mine area where the total mining height is less than 3 m (10 ft).

A comprehensive study has been made on the mine geology and its effect on longwall performance during the past two years. The main objective of the study was to understand and characterize the reserve to minimize the effect of adverse geological conditions on longwall performance. The study includes a correlation analysis between mining conditions and longwall performance using the geologic and geotechnical data collected from the mine. The study concludes that longwall performance is closely related to mining conditions including roof conditions, floor undulation, and parting thickness. Modifications to the mine plan can minimize the effects of these geological factors, to reduce the downtime rate and to increase productivity. This paper gives a detailed discussion on the effects of the geological factors on longwall performance at Shoal Creek Mine and summarizes the findings on characteristics of mine geology associated with the Shoal Creek reserve in the Warrior Basin.

MAIN CHARACTERISTICS OF MINE GEOLOGY AND MINING CONDITIONS

The original mine plan was developed from corehole and electric logs with a typical spacing of 305 m to 457 m (1,000 ft to 1,500 ft). Once actual mining began, it was discovered that traditional modeling techniques using data points with the above spacing did not accurately model the reserve due to the rapid changes in coal thickness and seam elevation. To better characterize the reserve for mine planning purposes, extensive mine geology data has been collected from both the underground and the surface. The data collected includes coal thickness, mine floor elevation, interburden between the Marylee Seam and the New Castle Seam above the Marylee, orientations of coal cleats and joints in the mine roof and on the surface, surface elevation, surface traces of linears, etc. Other roof information such as thickness of sandstone and RQD of roof strata, was provided by additional surface drilling and an underground exploration drilling program. The underground drilling program included core holes on approximately 305 m (1,000 ft) centers, drilled 3.05 m (10 ft) in the floor and 18.3 m (60 ft) in the roof, once the longwall gateroads were developed. The data was used in correlation

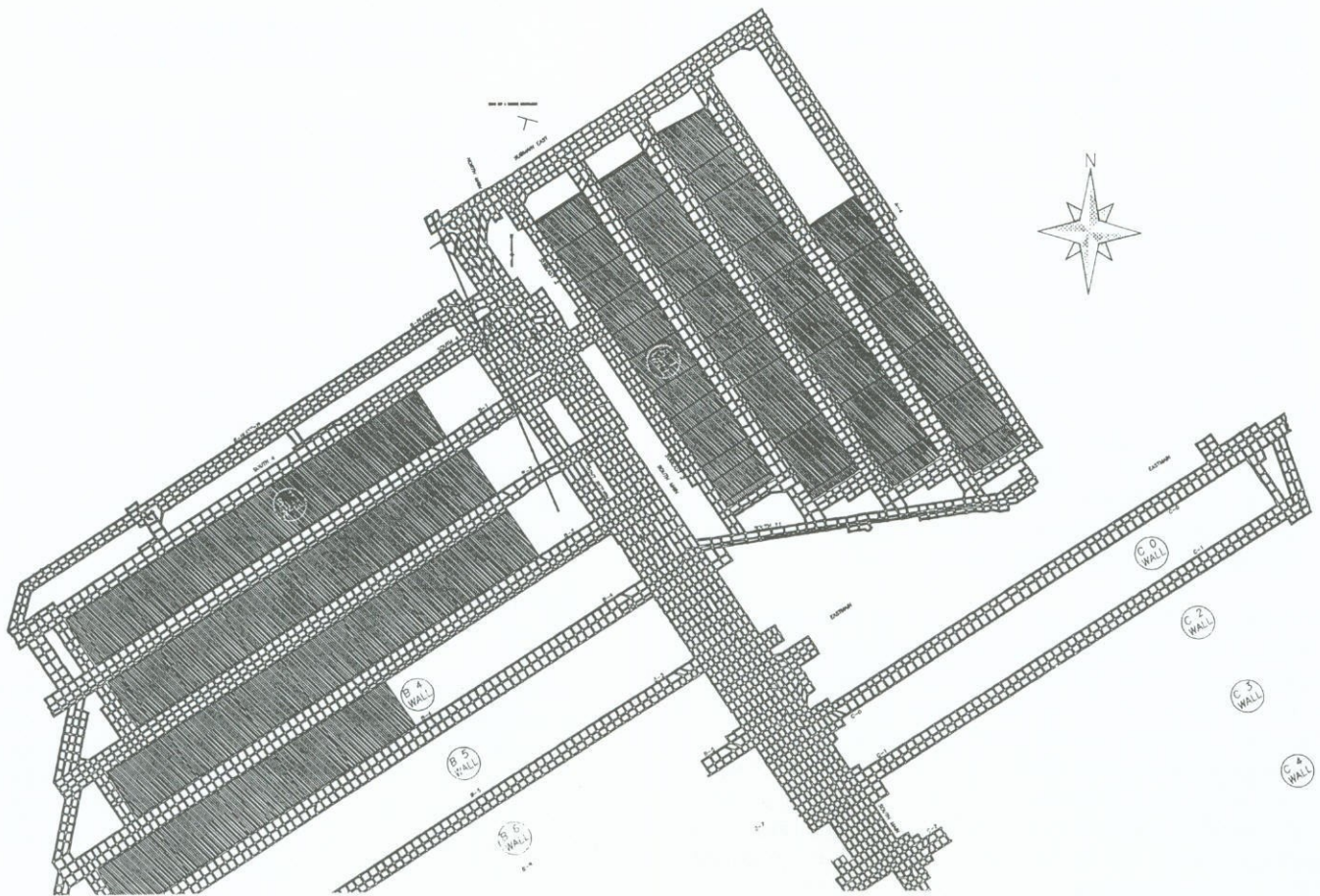


Figure 1 Layout of Longwall Panels at Shoal Creek Mine

analyses of geology, mining conditions and longwall performance.

Figure 2 shows a typical profile of the seam along the C-1 gateroad. It can be seen that the elevation of the mine floor, i.e., bottom of the Blue Creek seam, dramatically changes from area to area in the mine. The maximum floor gradient in the current mine area is more than 18%.

The analysis shows that the mine undulations are strongly associated with a rapid changes in coal thickness, especially the Blue Creek seam. Note that the Blue Creek thickness is usually 1.2 m (4 ft) or less in the reserve area, but can rapidly increase up to 3.1 m (10 ft), resulting in a variation of total mining height from 1.7 to 4.3 m (5.5 to 14 ft). The other findings from the analysis are given as follows.

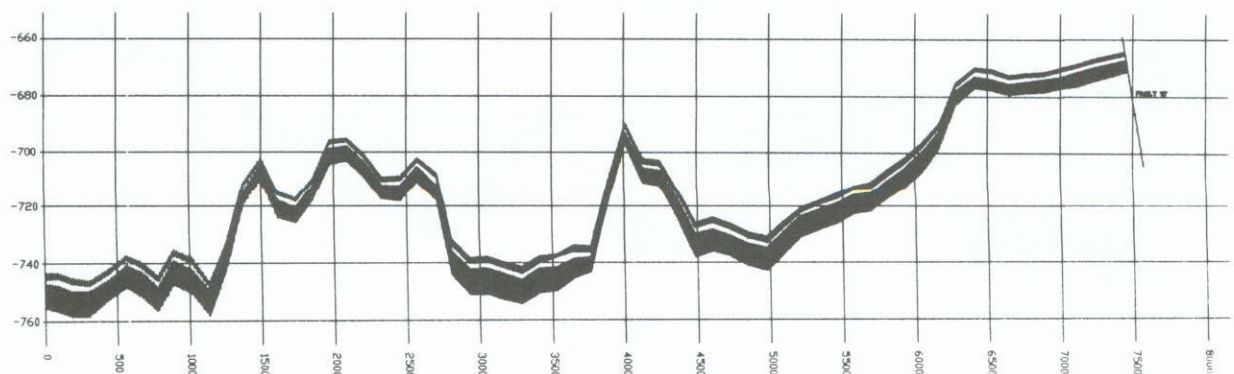


Figure 2 Variations of Floor Elevation and Coal Thickness Along C-1 Gateroad

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- The Blue Creek floor elevation drops approximately 2.9 m (9.5 ft) for each increase of one foot in the Blue Creek seam.
- The interburden between the New Castle seam and the Marylee seam increases approximately 2.6 m (8.5 ft) for each increase of one foot in the Blue Creek seam.
- The thinner coal areas generally underlie surface areas where the ridgetops are at a lower relative elevation and contain a higher concentration of eroded drainage paths and hollows.
- Surface areas containing many traces of northwest oriented linears correspond to channels of thicker Blue Creek coal and lower seam elevations.
- The thickness of the parting between the Marylee and Blue Creek seams is more consistent, ranging from 0.46 to 0.91 m (1.5 to 3 ft) in the mine area. The parting thickness gradually increases to the southwest in the reserve area.
- Up to 13.7 m (45 ft) of sandstone is found in the main roof. The thicker sandstone is closely related to the thick Blue Creek channel, lower seam elevation and thick interburden between the New Castle and Marylee seams.

The findings indicate that there exists a strong correlation between areas of thick Blue Creek coal, seam elevation, and roof strata and a lesser correlation between seam thickness and surface topographical features such as surface elevation and linears, etc. It implies that the mine rolls encountered underground were formed by early gullies or stream channels that existed during coal deposition. The great compression of the areas of thick Blue Creek peat during coalification resulted in linear traces on the present ground surface. Thus, the probability of finding thicker Blue Creek in a roll in the seam is higher between two northwest linears on the surface. The findings help to define the thick Blue Creek coal seam areas where drill holes are widely spaced and to predict the mining conditions, including rolls in seam elevation and the occurrence of sandstone channels. Figure 3 shows a comparison between the actual coal profile and the projected profile in the C-1 headgate before longwall gateroad development based on the correlation results. It can be seen that the projected coal thickness and the mine floor elevation generally follow the trend of the actual profile. It is believed that continuous collection of geologic data from the mine and updating these correlations will allow better projections in the future.

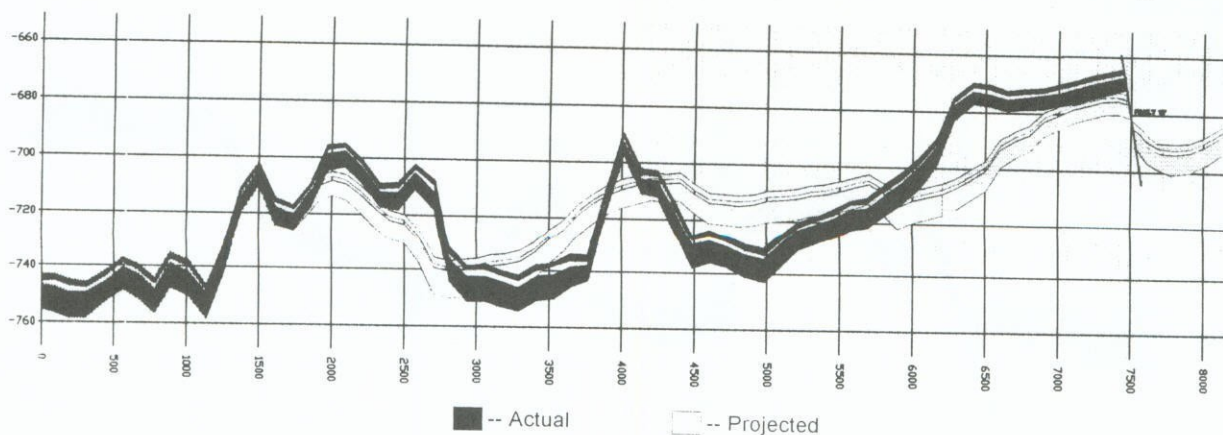


Figure 3 Comparison of Actual and Projected Coal Section Profiles

LONGWALL PERFORMANCE WITH VARIOUS MINING CONDITIONS

Roof falls and face (middleman) failures at the longwall face are two of the main concerns at the mine. The roof falls and face failures usually occur when a difficult mining condition exists, such as severe undulation of the mine floor, thick middleman parting, or a massive sandstone channel in the main roof. The failures cause large rocks to fall onto the panline, which frequently cause blockages of the face conveyor, causing mechanical failures of the longwall equipment and production delays. Production reports show that the face advance rate is reduced and face downtime is increased by at least 30% when difficult mining conditions are encountered. As discussed in detail below, it can be seen that several geological factors have a critical effect on longwall performance. They are severe undulations in the mine floor, thick sandstone in the main roof, the coal cleat orientation and parting thickness.

Effect of Various Geological Conditions on Longwall Shield Pressures

A continuous full-face real-time monitoring system (LoSCoMS) has been installed since 1995 at both longwall faces to collect the longwall shield pressure data (Park et al., 1997 and Deb et al., 1997 and 1998). The data are collected and sent to a surface computer through dedicated telephone lines, modems and serial ports. The data are analyzed and used to evaluate the roof behavior and its effect on longwall performance.

Effects of Sandstone Channel. Analysis of longwall shield pressure data indicates that the mine roof behaves very differently from area to area due to different roof conditions, especially sandstone thickness in the main roof. The Figures 4(a) and 4(b) give a comparison of shield pressures resulting from mining under the two main roof conditions encountered in the mine:

Case A: This is a typical roof condition where the sandstone channel exists. The roof consists of two thick sandstone layers, separated by a thin fractured shaly sandstone layer with a total thickness of 5.5 to 12.2 m (18 to 40 ft). The upper sandstone

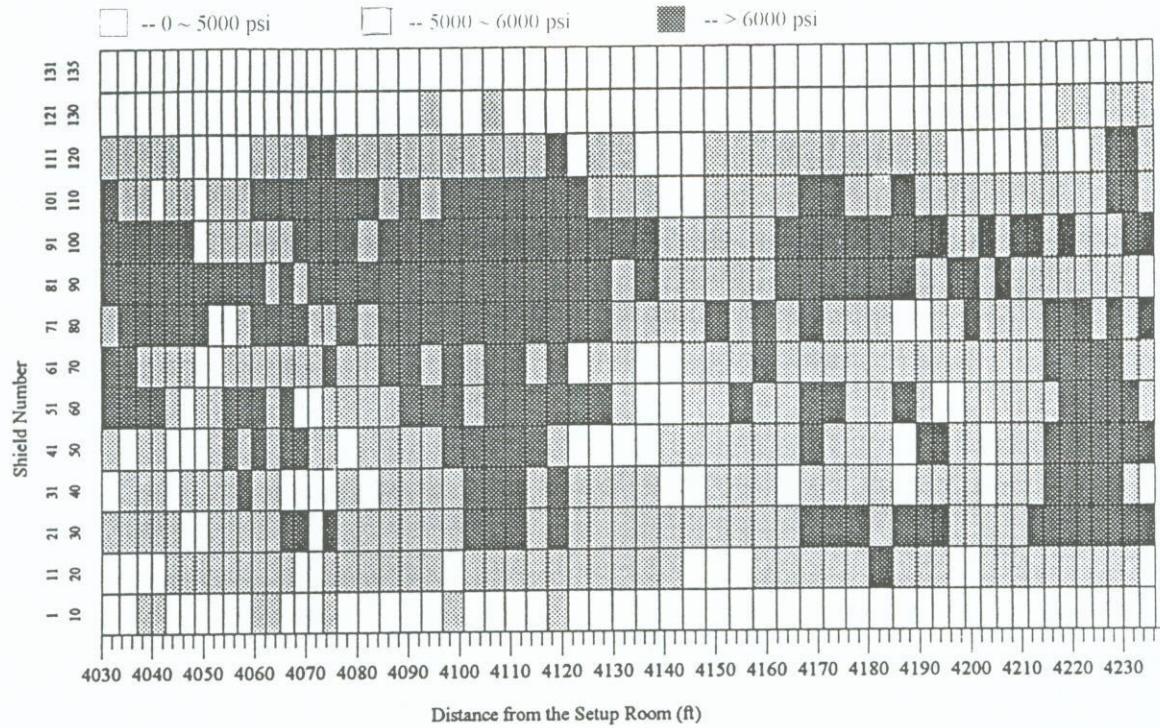


Figure 4(a) Final Shield Pressure Distribution for Case A Roof Condition

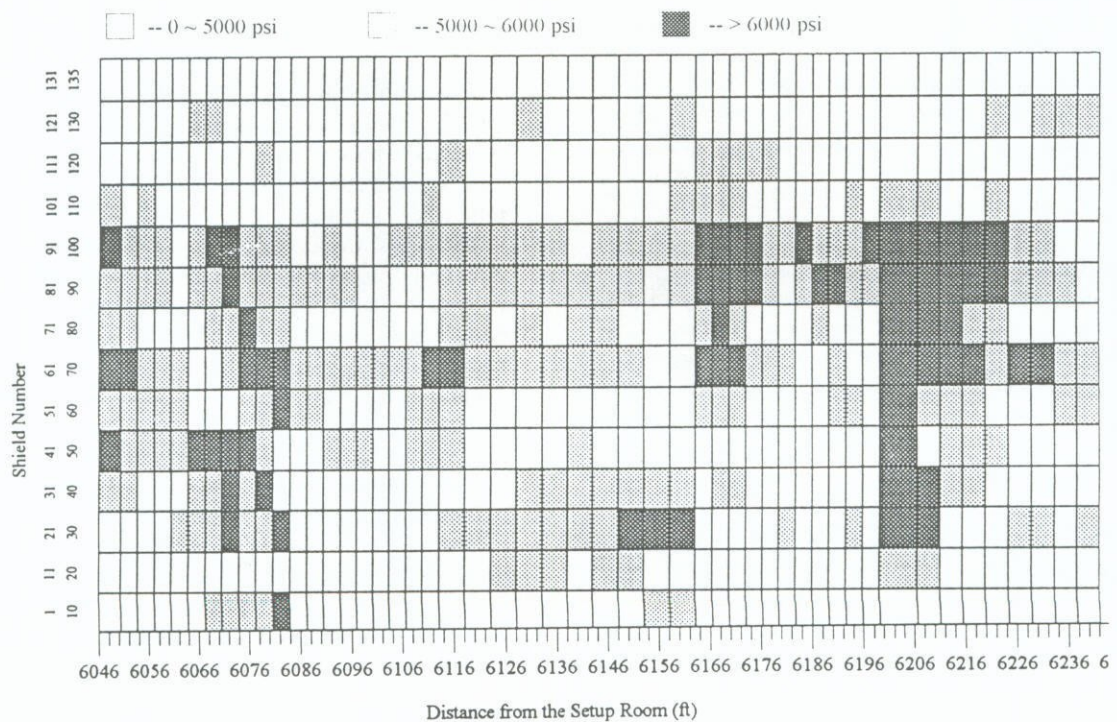


Figure 4(b) Final Shield Pressure Distribution for Case B Roof Condition

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layer contains more fractures and weak planes and has a lower RQD. Shield pressures are very high when the longwall face mines through an area with this roof condition. More than 60 percent of the shields across the face usually yield during a periodic roof weighting event. The yielding is sometimes very severe with a rapid leg closure rate.

Case B: The immediate roof consists of sandy shale approximately 15 ft thick. The main roof is competent sandstone but less than 4.6 m (15 ft) thick. In this case, the periodic roof weighting is more gentle, and the shield pressures are usually lower, compared with Case A.

A large database has been established to analyze the variation of shield pressure with the different roof conditions. The roofs in the mine are classified into different categories based on different roof behavior and their effects on the shields (Deb et al., 1998). Thus, once the geological information is collected, the roof behavior and their effects on the longwall can be predicted using the LoSCoMS program, especially during periods of severe roof weighting.

Effects of Mine Floor Undulation Mine floor undulation has a significant effect on shield pressure distribution. When the longwall passes through a region where there is a severe change in floor elevation, from upward to horizontal as well as horizontal to downward, a heavy loading usually occurs on the shields often causing severe shield yielding. The yielding lasts for more consecutive mining cycles than usual. Several large roof falls reported at the mine were associated with a condition of severe floor undulation. As discussed earlier, the mining conditions including coal thickness and roof strata dramatically change with changes in floor elevation. It is believed that more cracks and fractures may exist in the overburden strata in the area with severe floor undulations. An excessive high shield pressure can be caused by the fractures that are near parallel to the faceline. This is specially true when a massive sandstone exists in the main roof.

Effects of Mine Roof Joints The orientations of the dominant joints (more than 90%) found in the roof varies from N85°E to N100°E, i.e., an approximate 30°-60° angle with both facelines of the A and B panels. The joints have a near vertical dip and are continuous across a large area. The density (or interval) of the joints largely varies from location to location. Widely spaced joints in the mine roof are found in areas with thicker coal or a massive sandstone channel in the roof.

The existence of joints in the roof affects roof behavior, mostly the main-roof cantilever beam developed as the face advances. The joints are considered as a weak plane with a near-zero tensile strength. The study shows that the failure mode of the roof dramatically changes with variations in joint spacing. The main effects of the joints on the longwall are discussed as follows:

- Shield pressures are usually higher in the half of the face that the joints strike towards in the advancing direction. For example, in a panel oriented NW-SW and advancing NW

with joints oriented in a near East-West direction, shield loading would generally be higher on the west side of the face. At Shoal Creek, higher loading occurred in the tailgate half in the A panels but in the headgate half of the B panels.

- Where joints are closely spaced, the roof will not extend a long distance behind the shields before breaking. The broken roof falls into gob and provides support to the overburden strata. Thus, the weight of the overburden strata on the shields is reduced.
- Where joints are more widely spaced, e.g., wider than 100 ft, the main roof will overhang a considerable length. As a result, the periodic roof weighting occurs less frequently but is more severe.
- The location of shields experiencing higher pressure is often associated with the locations where joints intersect the coal face. A review of the shield pressure data for a series of successive passes of the longwall shows points of high shield pressure moving across the face in conjunction with the joint intersections with the face. Face failures are more likely to occur at these same points. The spacing of points of high shield pressure will usually be similar to the spacing of the joints.

The study concludes that the angle between the joint and faceline and the joint spacing are two major factors affecting the shield pressure distribution and face stability. If the longwall panel is designed to have a near parallel angle between the faceline and the joint orientation, a tremendous loading will occur across the entire face significantly increasing the potential for a roof fall when the joints are widely spaced.

Face Failures and Orientation of the Coal Cleats

The direction of the cleats in the Blue Creek is approximately N60°E ~ N65°E, i.e. a near parallel angle with the faceline of the A panels but a perpendicular angle with the faceline of the B panels. Face failures frequently occur during periodic weighting time when both seams, the Marylee and the Blue Creek, are mined. This was more prevalent in the A panels where the cleat direction is parallel to the faceline. A comparison of middleman failures for the A and B panels was conducted. Figure 5 shows a comparison of results, showing the differences of the probability of face failures. Note that the probability of a face failure is developed to describe the frequency of face failures in each mining cycle. For example, a probability of 10% means that a face failure may occur 10 times in 100 cycles in front of one shield or may have a single face failure across a 10-shield area over 100 shields. It can be seen that the probability of face failures in the B panels is much less than in the A panels at all times, with 2.8% and 8.0%, respectively. It must be pointed out that the only difference in conditions in the comparison areas was the angle of the cleats in the Blue Creek seam relative to the faceline.

The face failures significantly increase the amount of face downtime and reduce the rate of face advance. The following table shows the compared results from the A-3 and B-3 longwall panels:

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Table 1 Comparison of Longwall Performance between the A-3 and B-3 Panels

Items	Panel A-3	Panel B-3
Face Advanced Rate (ft/day)	30.55	38.75
Face Downtime Rate (%)	33.70	20.19
Time Lost at Face (min/wk)	1,832.00	1,082.00
Time Lost Due to Rock (min/wk)	471.00	120.00
Mech. Failures of Equip. (min/wk)	767.40	439.00

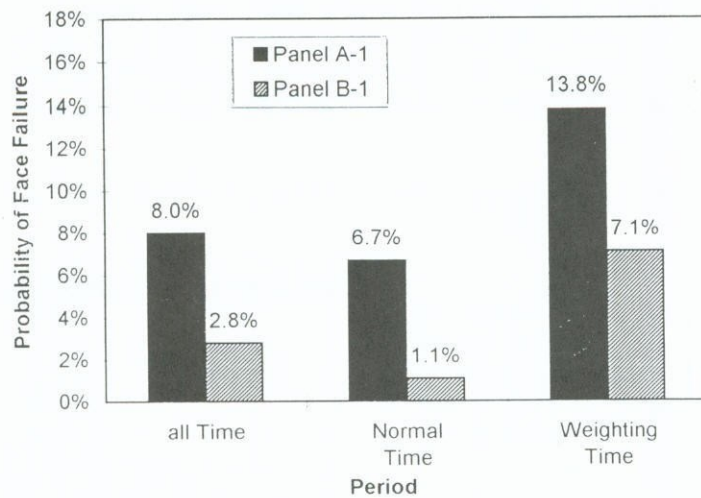


Figure 5 Comparison of Probability of Face Failure in Panels A-1 and B-1

The above table lists the downtime associated with large rock. It can be seen that time lost due to roof falls and face failures at panel A-3 was nearly four times higher than that at panel B-3. Figure 6 shows the face components causing the most downtime in Panel B-3. Three of the four most common failures are chain flights, ranging arms and conveyor chain. Large rock falling into and in front of the panline from face failures contributes to the number of failures of these components. Downtime due to mechanical failures of the face equipment was higher in the A panels where face failures were more frequent versus the B panels.

The observation also showed that the face failures usually occur two to four mining cycles before the peak loading imposed on

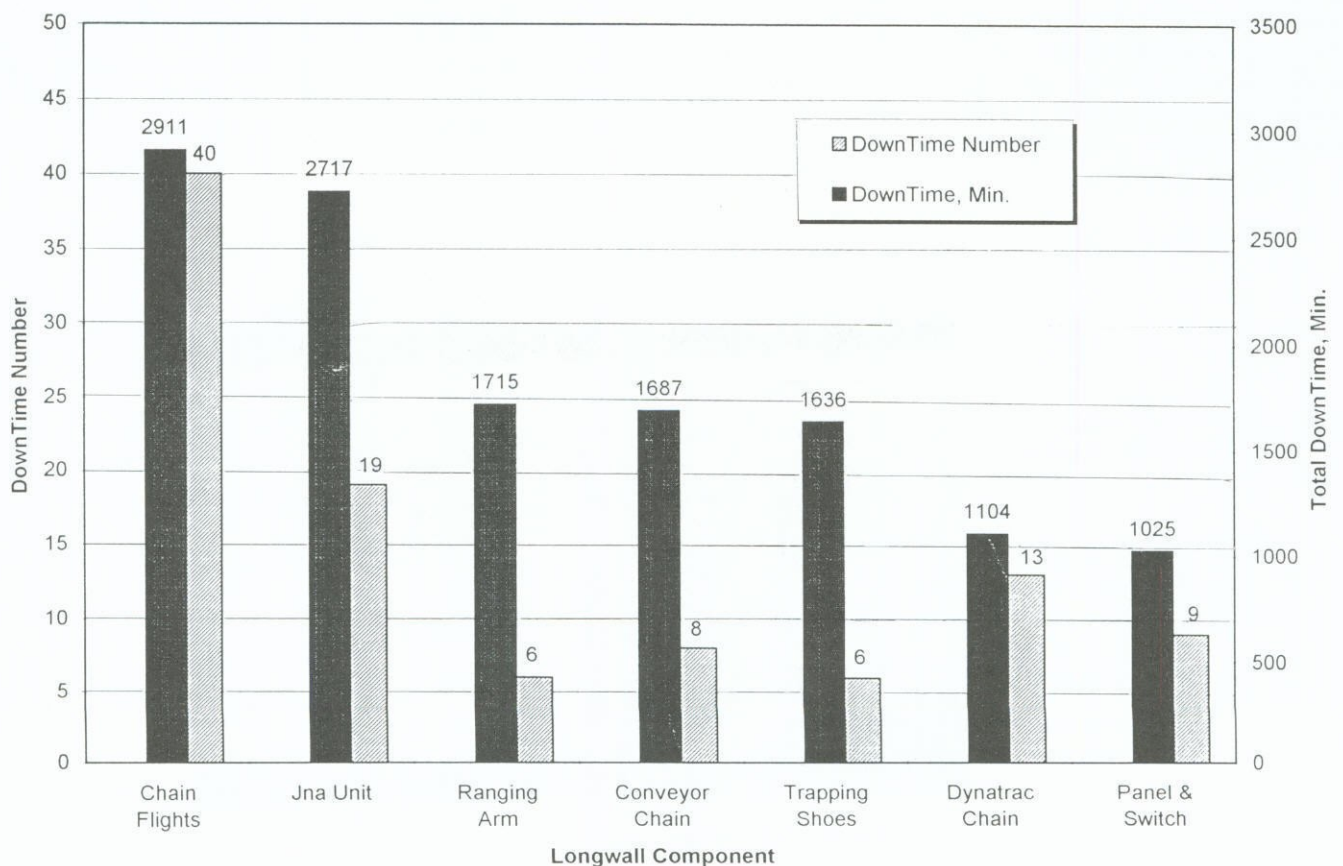


Figure 6 Total Time and Number of the Most Common Failures on Longwall Panel B-3

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the shield during the periodic weighting time. As displayed in Figure 7, in the initial period of the development of a periodic roof weighting, the development of a crack in the main roof causes the roof to rotate at the face rib (usually the coal seam has a higher stiffness than the shield supports). Although the shields are beginning to take weight, an extremely high stress is applied on the face rib forcing the face to fail. As the longwall continues advancing, a roof fall is more likely if there is a severe face failure. This indicates that the Blue Creek stability plays an important role on the face stability including the parting and roof. When the faceline is perpendicular to the direction of cleats in the Blue Creek as in the B panels, the strength of the seam and its capability to support the middleman is maximized, reducing the potential for face failures. This greatly increases the capability of the Blue Creek to support the middleman. However, when cleat direction is parallel to the faceline as in the A panels, the Blue Creek is much more easily crushed when the face is exposed, causing the loss of its supporting capability to the middleman and increasing the span of the unsupported area between the canopy tip and the coal.

Mining the Middleman Parting Versus Mining Under the Parting

The magnitude of the effect of the parting thickness on longwall performance when mining total seam was investigated. Figure 8 shows the longwall face advance rates at different parting thicknesses. An increase of one foot in thickness of parting resulted in an increase of 14% in the downtime rate or a reduction of 25% in face advance rate. The parting often falls out of the face without being cut, especially during periodic weighting time. Larger blocks of rock from the thicker parting can jam the face conveyor causing more mechanical problems.

Mining under the parting where the Blue Creek seam has sufficient thickness eliminates most of the face failure problems. Much better longwall performance has been achieved at the mine when only the Blue Creek seam is mined. Table 2 shows a comparison of mining the total seam and the Blue Creek only in the same panel. Significant differences in longwall performance can be seen, between mining under the parting and mining total seam.

Table 2 A Comparison between Mining Total Seam and Under the Parting

Items	Blue Creek Only	Total Seam
Face Advanced Rate (ft/day)	38.93	22.75
Face Downtime Rate (%)	21.67	45.73
Time Lost at Face (min/wk)	1,102.00	2,631.00
Time Lost Due to Rock (min/wk)	207.00	751.50
Mech. Failures of Equip. (min/wk)	387.10	1151.00

However, when mining underneath the parting, roof stability becomes a great concern, especially in the unsupported area between the canopy tip and the faceline. The immediate roof consisting of the parting, Marylee seam and overlying shale has a much higher potential for a roof fall. During a severe roof weighting event, the weak Blue Creek seam can fail, resulting in a large unsupported area in front of the canopy tip line. Although the potential for roof failures is higher and the Marylee coal is not recovered when mining the Blue Creek only, the increased productivity and reduced maintenance costs make this method of mining the reserve the most economical where the Blue Creek is thick enough for the face equipment.

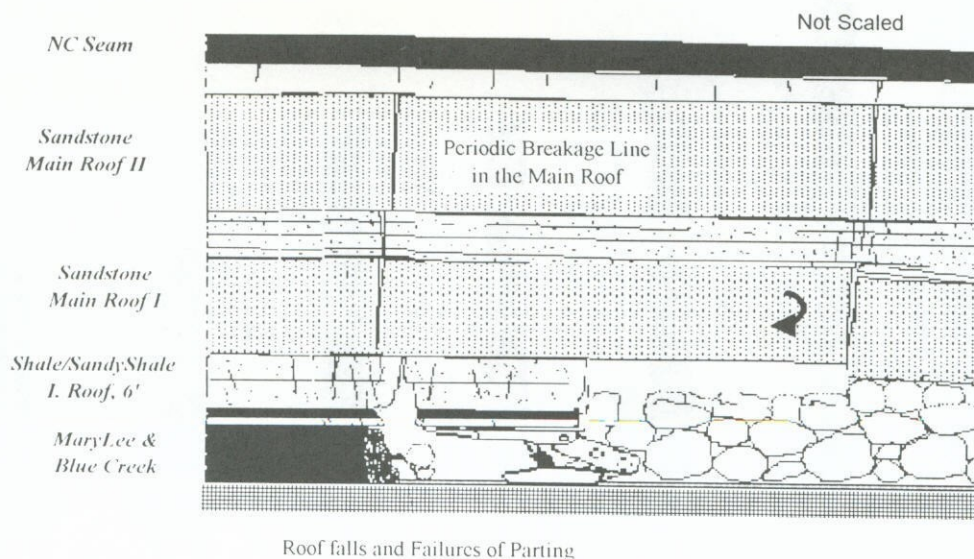


Figure 7 Main Roof Breakage and Immediate Roof Falls During Periodic Roof Weighting Time

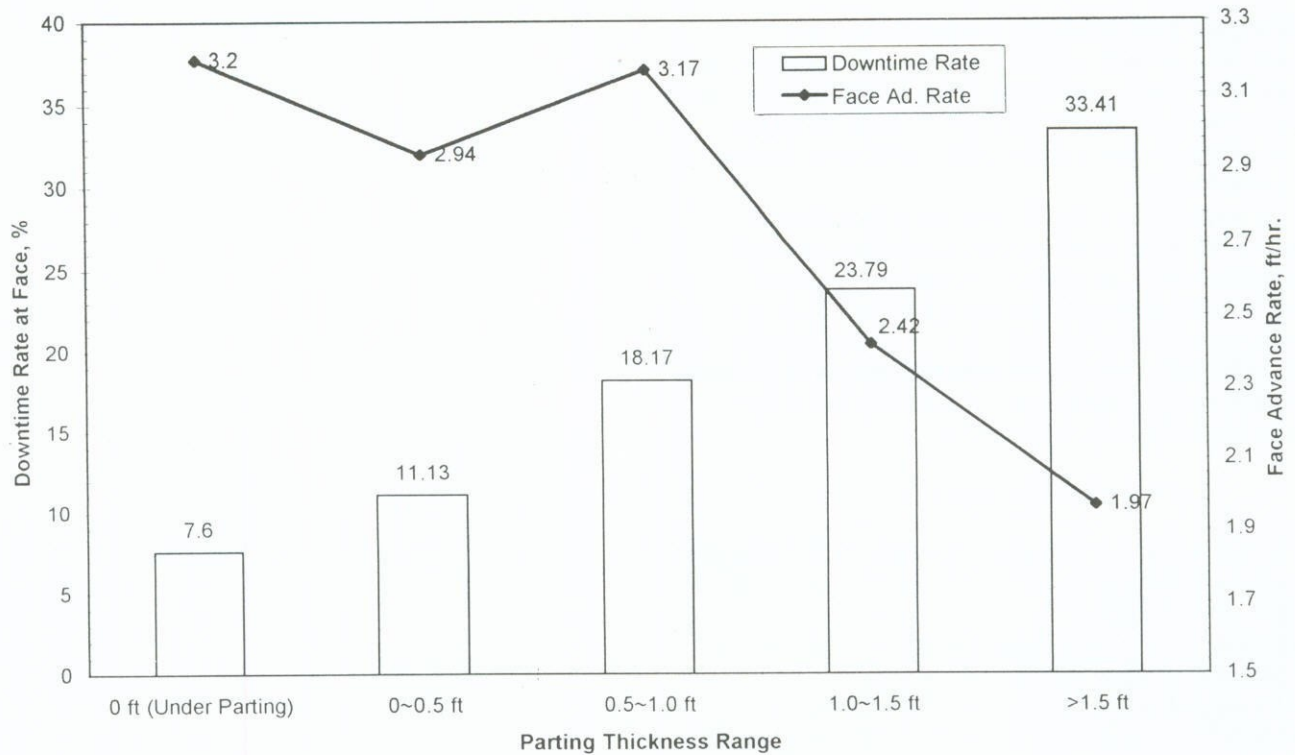


Figure 8 The Effect of Parting Thickness on Longwall Performance at Panels B-2 and B-3

OPTIMIZATION OF MINING PLANNING

As discussed above, **minimizing** effects of the geological factors on longwall mining is a key to improving longwall performance and reducing mining costs at Shoal Creek Mine. Higher production can be achieved with better panel orientation, predicting severe roof weighting, scheduling planned idle periods on the longwall face, etc.

- 1) Panel Orientation. A near parallel angle between the longwall faceline and the cleats or mine roof joints significantly increase the potential for ground control problems. Current mine planning focuses on avoiding a design where the longwall faceline is near parallel to the cleat orientation and the roof joints. It is believed that the optimum design is to orient the panels in a northeast orientation, similar to the orientation of the B panels. This results in the longwall faceline being nearly perpendicular to the cleat orientation and provides more than a 30° angle between the joints and the faceline and gate entries.
- 2) Mining Under The Parting. When thick Blue Creek seam is encountered, only the Blue Creek seam is mined to eliminate most of the face failure problems and reduce the downtime of the equipment. The cost savings also include reduced conveyor and coal preparation costs. Roof falls are minimized by closely monitoring shield performance with the **LoSCoMS system and scheduling idle face time when a lower shield pressures are predicted by the software as discussed below.** This is especially important when the longwall face is passing areas with thick sandstone channel in the main roof.

3) Monitoring Shield performance and Scheduling Idle Time.

Severe roof or/face stability problems have been experienced in the mine during several idle periods when a severe roof weighting event occurred. The effects of severe roof weighting can be minimized by advance knowledge of increased roof weighting and mining through the area without stopping until the high pressures subside. Early preparation is a key for continuous operations to prevent severe yielding and face and roof failures. Thus, monitoring shield performance and predicting the location of the severe roof weighting event allows mine management to determine the optimum face location for a planned idle period such as maintenance, holidays, weekend, etc.

CONCLUSIONS

1. There exists a strong correlation among the thick Blue Creek seam, mine floor elevation, roof conditions and the surface features. Geological information indicates that the mine rolls encountered in the mine were formed by gullies or streams that existed during coal deposition. The great compression of the areas of thick Blue Creek peat during coalification resulted in linear traces on the present ground surface.
2. The mine floor undulation, thick sandstone channel, the coal cleat orientation and parting thickness are main geological factors affecting the longwall performance and mining cost. The angle between the longwall faceline and the orientation of the cleats in the Blue Creek has a very significant impact on longwall performance. An optimum design is to orient the panels in a northeast direction, similar to the orientation of the B panels.



3. Mining under the parting reduces face failures and mechanical downtime and increases longwall advance rates. Further cost savings are realized by reduced conveyor wear and increased preparation plant yield. These advantages outweigh the increased potential for roof failures and loss of the Marylee coal.
4. Early preparation is a key for preventing face and roof failures caused by excessive roof loading during periodic weighting events, especially where massive sandstone is present in the main roof. Monitoring shield pressure and the predicted location of the severe roof weighting provided by **the LoSCoMS system allows mine management to determine** the optimum face location for a planned idle period such as maintenance, holidays, weekend, etc.

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Comparison of Ground Conditions and Ground Control Practices in the United States and Australia

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ABSTRACT

Australia and the United States both have large, modern underground coal mining industries. Many companies have invested in both countries, and there is growing awareness that technological developments in one country can rapidly affect the other. Yet the ability of technology to pass between the countries depends on comparability of ground conditions. Many Australian observers believe that their ground conditions are significantly poorer than typical U.S. conditions, but some U.S. professionals are not convinced.

The author recently visited 10 Australian longwall mines and two room-and-pillar mines. Each mine visit included determination of the Coal Mine Roof Rating, roof support installed, pillar design, and an in-depth discussion of ground control experience. The Australian conditions observed are compared with an extensive data base from U.S. mines. The paper also offers observations on geotechnical data collection, monitoring, and the roles of the mine inspectorate and research community.

Acknowledgments

I would like to express my sincere appreciation to Mark Colwell and Russell Frith for arranging all the longwall mine visits, and to Ian Anderson, Steve Mathews, and John Shepherd for arranging other portions of the trip.

INTRODUCTION

Early in 1998 the author visited Australia to gather information on the Australian state-of-the-art in mine safety technology. The focus was on ground control and disaster prevention and response. A secondary goal of the trip was to assist an Australian Coal Association Research Program (ACARP) project that is evaluating the applicability of the

Analysis of Longwall Pillar Stability (ALPS) and the Coal Mine Roof Rating (CMRR) to Australian mines (Colwell and Frith, 1997).

A total of 12 mines were studied in the course of the trip. One week was devoted to five longwall mines in the newest Australian coalfield, central Queensland (figure 1).

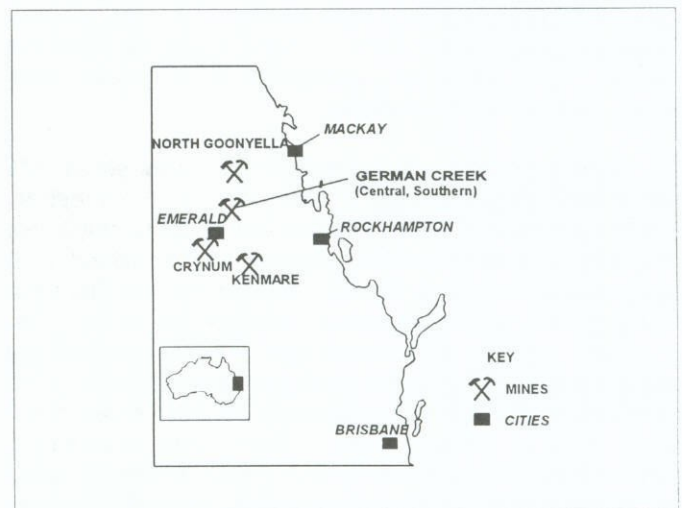


Figure 1. Coal mines visited in Queensland.

Two of the mines were only on their second longwall panel. The second week of the trip included visits to Mining Engineering departments at three universities and meetings with other ground control specialists. Two room-and-pillar mines were also studied, one a deep-cover, full-extraction operation, and the other a shallow, partial-extraction operation beneath massive conglomerate roof. During the final week of the trip, five longwall mines were studied in New South Wales (NSW). Two were in the Lake Macquarie/Newcastle district, and one each in the Hunter Valley, Western (Lithgow), and Southern (Wollongong) coalfields (figure 2).

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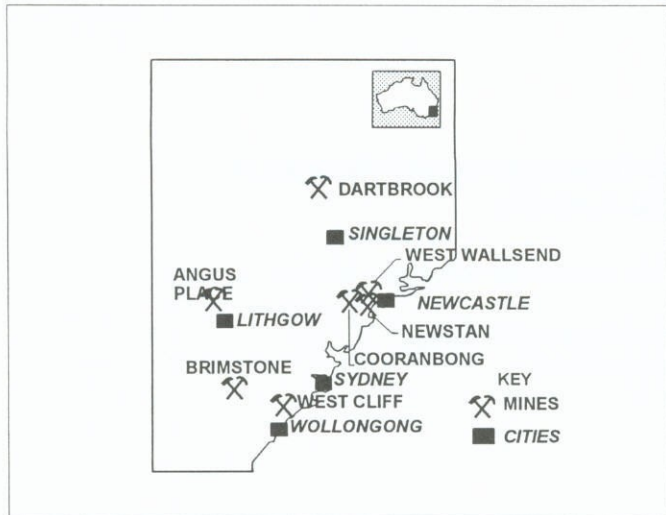


Figure 2. Coal mines visited in New South Wales.

BACKGROUND

Australia is the world's leading exporter of coal. Although Australia's total production of about 250 million raw tonnes of bituminous coal is considerably less than China, Russia, or India, Australia's mining technology is extremely modern. It is fair to say that the Australian coal industry has more in common with the U.S. coal industry than any other in the world. Indeed, in recent years the same coal producing companies have made large investments in both countries.

Underground mining currently accounts for about 28% of Australian bituminous coal production. Almost all underground production (and 96% of all coal production of black coal) comes from the two states of Queensland (Qld) and New South Wales (NSW). The last decade has seen rapid growth and technological change in the Australian industry. Underground production in NSW expanded by nearly 20% (to 55 million tonnes) between 1986 and 1996, and it nearly tripled during the same period in Queensland (from 6 to 16 million tonnes). The decade also saw a pronounced shift to longwall technology. Longwall mines produced about 13 million tons in 1986, and that figure has grown to 48 million tons a decade later. Longwalls now account for nearly 80% of underground production in NSW and almost 100% in Queensland (figure 3).

On the whole, Australia maintains a mine safety record that is similar to the U.S. The Australian lost time injury frequency rate (LTIFR) in 1994 was 16 per 200,000 hours worked, compared to an LTIFR of 11 for the U.S. (Munton, 1995). There have been two major mine disasters in Australia during the last five years. Eleven miners perished in the Moura mine explosion in 1994, and four miners were killed at Gretly Colliery by an inundation in 1996. There were a total of 12 other fatalities during the period, 4 of which were attributed to roof or rib falls (Doyle, 1997).

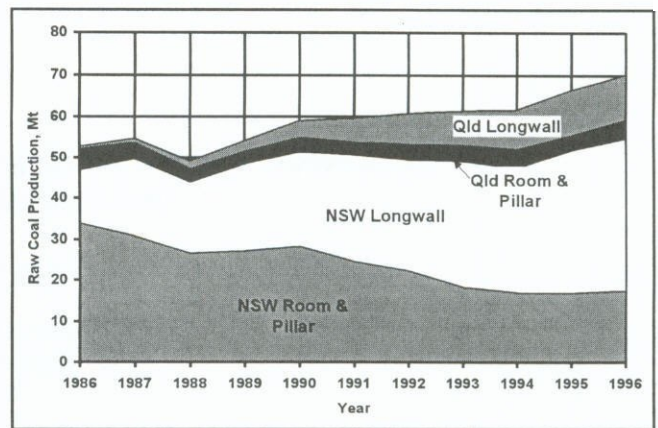


Figure 3. Trends in Australian underground coal production.

Today, the Australian coal mining industry seems to be at a crossroads. The optimism based on several decades of sustained growth seems to have given way to uncertainty. Just a year ago the trade magazines were filled with plans for major new projects, but now layoffs and mine rationalizations seem to be the order of the day. Labor strife has also increased, with major confrontations at Arco's Gordonstone mine and the Rio Tinto operations. Low coal prices, competition from Indonesia and South Africa, and concern about the economic slowdown in Asia (the primary market for Australian coal) underlie the uncertainty.

The Australian coal industry is now struggling to increase productivity, while continuing to improve its safety record. Ground control is a serious hazard throughout the industry, along with spontaneous combustion and control of mine gasses. The Australian mining community is meeting the challenge with a vibrant, cooperative research effort that is on the cutting edge of many mine safety technologies. The remainder of this paper will describe the state-of-the-art in Australian ground control, and compare it with the U.S.

ROOF GEOLOGY

The Coal Mine Roof Rating (CMRR; Molinda and Mark, 1994) was determined at all 12 underground operations that were visited. The range of CMRR values is shown in figure 4a. The two strongest roofs were a massive conglomerate (CMRR=90) and an interbedded shale and sandstone (CMRR=60). The weakest top (CMRR<45) included four mines with thick coal roofs and a sandstone laminated with igneous tuff layers. The intermediate roofs included one thick coal roof, two Bulli seam "laminates" (interbedded shales and sandstones), a sandy shale, and a stackrock sandstone. On the whole, the roof in Queensland seemed somewhat more competent than that in NSW.

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AUSTRALIAN HISTOGRAM OF CMRR

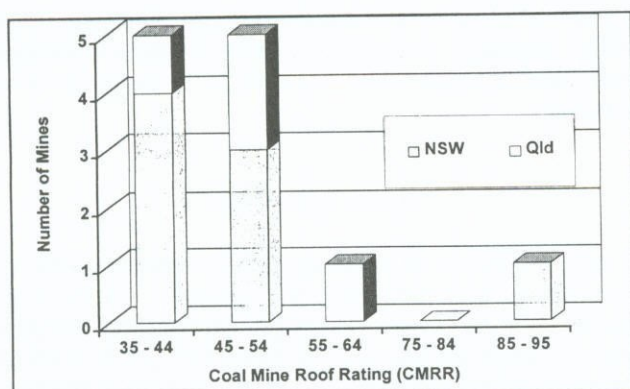


Figure 4a. Range of Coal Mine Roof Rating (CMRR) values observed in Australia.

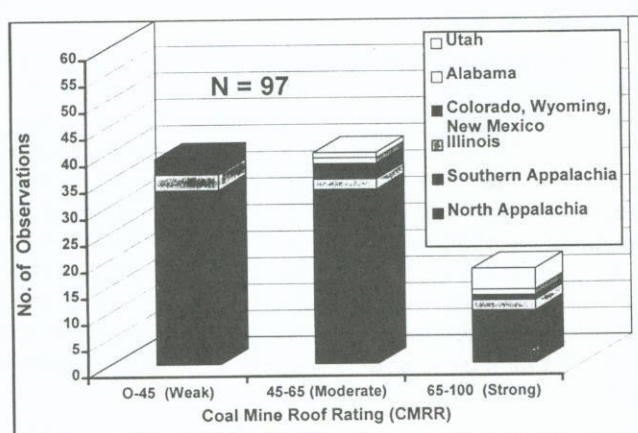


Figure 4b. Range of Coal Mine Roof Rating (CMRR) values observed in the U.S.

Figure 4b shows the range of CMRRs found in U.S. coal mines. On average, it appears that the roof in NSW may be roughly equivalent to that found in the Northern Appalachian coalfields, while Queensland roof may be closer to that in the Central Appalachians. Overall, the range of roof quality was similar to that found in the U.S.

Large-scale geologic features had an important effect on longwall performance at a number of mines. Igneous dikes seemed to be the most common feature, followed by faults. Massive roof, as discussed below, was another important issue.

The depth of cover varies significantly from coalfield to coalfield. Most of the Queensland mines gain access to seam from the highwalls of surface mines, and the cover is relatively light. Mines in Southern NSW have the deepest cover because the coalfields are relatively mature and the terrain is mountainous. Cover is moderate in the Northern and Western NSW coalfields.

The presence of horizontal stress was not very noticeable at most of the mines visited. Active mines in the Queensland coalfields particularly seem to have low stress levels, though some deeper operations there have had problems in the past. Horizontal stress appears to be most important in the Western and Southern NSW coalfields. In the Western coalfields, where the coal seams are only a few meters above the basement rock, "structural zones" have been associated with some of the most difficult ground conditions in the country.

It seemed that the Australian mines were well supplied with geotechnical information from core logs and laboratory testing. For example, Dartbrook mine has a coverage of 18 holes per square km, or one hole every 235 m. About half these holes were fully cored, and many samples were tested for uniaxial, triaxial, and bedding plane shear strength (Hayward, 1998). Many mines estimate rock strength from geophysical logs, often using relationships that have been derived from their own testing. Measurement of horizontal stress also seems routine and nearly universal.

The geologic characteristics of the visited mines are summarized in Table 1.

Table 1
Geologic Characteristics of 12 Representative Australian Coal Mines

Coalfield	CMRR	Depth (m)	Horizontal Stress
Queensland	40-60	<250	5-15 MPa
NSW - Northern	40-90 ¹	100-300	5-10 MPa
NSW - Western	35	100-350	20 MPa
NSW - Southern	45-55	300-500	20-35 MPa

¹CMRR=90 with massive conglomerate roof.

LONGWALL LAYOUT AND MINING METHODS

All of the longwall mines employed two-entry gates. The crosscut spacing was almost universally 100 m. Pillar widths varied between 28 and 40 m, and entry widths were 4.5 to 5.3 m. One striking feature was the thickness of the coal seams. In the mines visited, the pillar heights ranged from 2.5 to 4 m.

Most of the gates were driven by a single machine, typically an ABM20 with mounted roof drills. Often only a single shuttle car was used. The typical procedure was to mine 1-1.5 m, and then install a row of bolts. The machine will drive crosscut-to-crosscut before moving to the other heading. There is currently quite a bit of interest in American-style "place-changing," using a separate roof bolting machine. At least four of the mines that I visited have tried place changing, with satisfactory roof stability at

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only two of them. In the U.S., it has been found that "extended cuts" are more likely to be successful when the CMRR exceeds 40-50 (Mark, 1998), and the Australian experience seems to fit the same pattern. Also, the large Australian mining machines (like the ABM20) are usually slow to tram, and two-entry gates with long crosscuts are far from optimal for rapid place-change cycle times.

ALPS Stability Factors (SF) have been determined at a number of mines that are involved in the ACARP project. The preliminary data show similar trends to those found in U.S. studies. Figure 5 shows that when the case histories are plotted against the U.S. design equation (Mark et al., 1994), there are only four misclassifications. Three of the misclassifications fall near the line, and the fourth may be successful because of an extremely high density of primary support. The very worst tailgate conditions in the data base were encountered at a mine where the ALPS SF was 0.8, well below the suggested value for a CMRR of 35. In contrast, at other operations where the ALPS SF was well above the suggested value, it was possible to drive a vehicle all the way up to the tailgate corner of the longwall.

A variety of techniques are used to design pillars in Australia. The University of New South Wales (UNSW) recently developed a pillar strength formula from back-analysis of room-and-pillar case histories (School of Mines, 1994; Hebblewhite and Galvin, 1996) which has been widely applied. Numerous mines have employed stress

measurements to evaluate pillar behavior, and sophisticated computer simulations are available (Gale, 1998). However, the relative lack of variation in longwall pillar size indicates that there may be room for more innovation.

ROOF SUPPORT

One immediate and obvious difference between the two industries is their tolerance for roof falls. In the U.S., approximately 2000 falls of supported roof occur each year, which is approximately one for every 120,000 tons mined underground on advance. In Australia, none of the large mines I visited had experienced more than a handful of roof falls. Indeed, I was told on several occasions that their unwritten goal was "no outby roof falls." The intolerance of roof falls even extends to inby roof failures during mining.

The universal primary support in Australia is what would be called a torque-tension bolt in the U.S. These are fully-grouted with a two stage resin (a fast set for the anchor at the back of the hole, and a slow set around the tensioned portion of the bolt). Australian roof bolts tend to be of larger diameter and made from higher strength steel, with a typical yield load of about 20-25 tonnes. The bolt length was 1.8-2.4 m almost everywhere. At the mines I visited, all the bolts were installed with drill rigs mounted on the continuous miner, or with hand-held drills. It seems that it might be difficult to obtain consistent quality installations with the hand-held units.

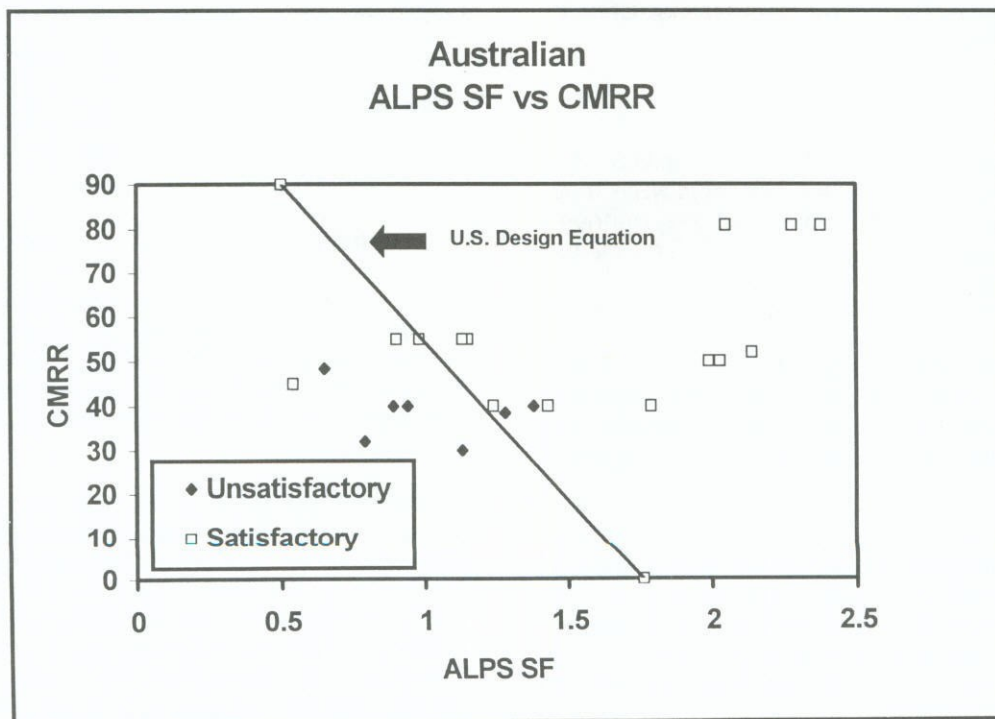


Figure 5. Preliminary data showing Analysis of Longwall Pillar Stability (ALPS) stability factors from Australian mines.

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There has been some controversy in the Australian ground control community over the importance of the installed tension. There are devices available that allow torque-tension bolts to be installed with more than 10 tons of load, twice the usual amount.

The bolt density varied considerably in the mines studied. Six mines used four bolts on 1.2-1.5 m centers, a pattern that would be considered typical in the U.S. Another four mines used more intensive patterns, with six bolts on 1-1.2 m centers. Figure 6 compares Australian with U.S. bolt densities.

The trend towards reduced roof bolt densities seems to be relatively recent. At one mine I visited near Newcastle, the bolt density has been decreased by 56% (from 6 per meter to 4 per 1.5 meter) over the last three panels, apparently without negative effect.

Rib bolting is almost universal in Australia, due to the high seams. Two or four rib bolts are usually installed per row of roof bolts, and additional rib bolts may be installed later. Several mines expressed disappointment in fiberglass bolts, which often fail at low load or even during installation. Some plastic bolts are currently being trialed.

In the U.S., secondary tailgate support is usually installed nearly simultaneously with the headgate pass. In Australia, most mines try to leave the tailgate open for access and ventilation. About half the longwalls visited used no regular tailgate secondary support, and most of

the rest installed it only within 100 m of the tailgate corner. Burrell cans seemed to be the most common standing support, followed by Link-N-Lock cribs. No tailgate I observed was supported by wood cribs (or "chocks," to use the Australian terminology). Cable bolts and flexi-bolts were also used for supplemental support.

In general, it seems that Australian longwalls may have traditionally relied upon primary roof bolts for tailgate support, which may partially explain their higher bolt density. The disadvantage is that intensive primary support patterns slow down gate road drivage rates. The current trend towards reducing the number of primary bolts speeds up drivage, but requires that mines make more use of cable bolts and standing supports in tailgates.

One notable contrast with U.S. longwalls is that more than 60% of Australian faces are using four-leg supports. Many of these are relatively new supports, with capacities of up to 900 tonnes. In the U.S., only 2 of 65 longwalls use four-legged supports.

Retreat Mining: Pillar recovery operations have historically accounted for the lion's share of ground control fatalities in Australian coal mines. In 1991, seven miners were killed on NSW pillar lines. Following a major research, education, and regulatory effort undertaken by the entire mining community (School of Mines, University of New South Wales, 1994; Coal Mining Inspectorate and Engineering Branch, 1992; Shepherd, 1997), no fatalities have occurred during the last five years.

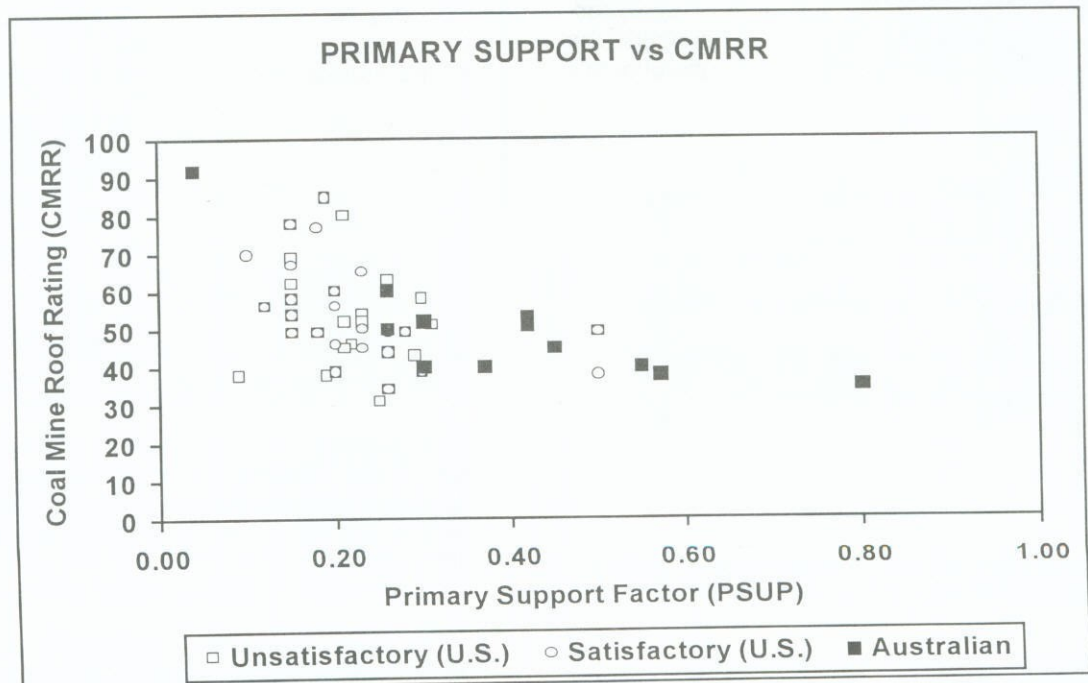


Figure 6. Primary bolt densities observed in U.S. and Australian coal mines. PSUP = (Bolt length * bolt diameter * number of bolts/row)/(Entry width * Spacing between rows of bolts) (see Mark et al., 1994).

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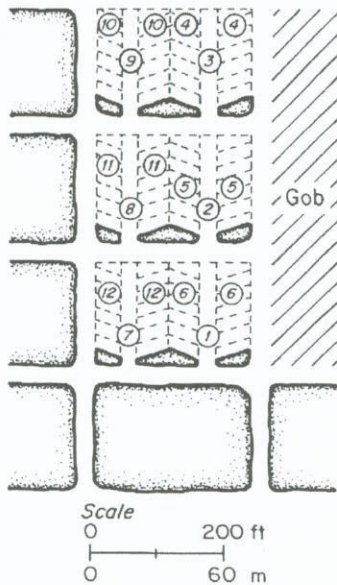


Figure 7. Modified Wonga-willi pillar extraction sequence used at Brimstone Colliery.

One reason for the improvement is better support. Breaker line supports (BLS) were introduced in the mid-80's, and are now standard. One typical application is at Brimstone Colliery, under 400 m of cover. Here pillars are developed on 40 m centers. On retreat, the pillars are split, and then lifted using a Christmas-tree sequence (figure 7). Three BLS are walked down the split as it is recovered. Large "stooks" (pushouts) are left to support the intersections, but they are engineered to crush out at this depth of cover.

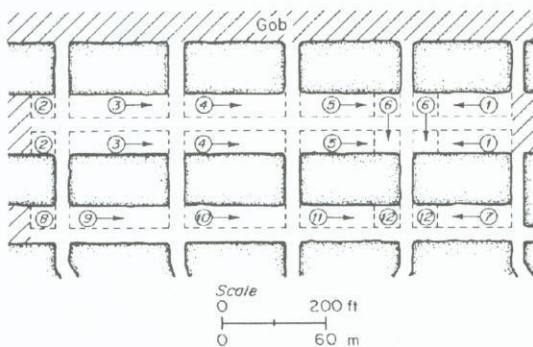


Figure 8. Partial pillar extraction sequence used beneath massive conglomerate roof at Cooranbong Colliery.

At Cooranbong Colliery, the roof is a massive conglomerate under only 100 m of cover beneath a residential area. They also develop large pillars (45 m

centers), but then only Christmas tree down the entries, leaving 20x40 m pillars for permanent overburden support (figure 8). Again, three BLS are used in the lift. The major change here has been to support the roof with a standard bolt pattern, together with a scaling program. Previously, much of the mine was only spot-bolted.

Roof Monitoring: Australian coal mines make far more extensive use of roof monitoring than U.S. mines. Indeed, it seemed that several individual mines probably had more geotechnical monitoring installed than the entire U.S. industry. For example, one of the Queensland mines I visited installs a two-point tell-tale in every intersection. The stations are initially read daily by a shift boss, and plotted up in the mine office. If a station is observed to be still moving rapidly when the deformation reaches 40 mm, then cables or trusses will be installed. Reportedly, approximately 25% of all intersections at this mine require supplemental support. The same mine currently has two pillar stress measurement sites installed. On the other hand, about half of the mines visited only use geotechnical monitoring for "special projects" like longwall recovery rooms or driving through faults.

Table 2 lists roof monitoring "triggers" used by different mines. It is interesting that three of these triggers are based on the rate of roof sag, while the other two use a total sag criterion. Moreover, the trigger rates vary by an order of magnitude. Clearly, there is room for research to relate critical roof movements to geology and other factors. In the U.S., a typical trigger rate is about 5 mm/wk (Mark et. al., 1994).

Table 2 Roof Monitoring Action Points for Five Australian Mines			
Coalfield	CMRR	Type of Criterion	Magnitude
Queensland	40	Total movement	40 mm
Queensland	50	Rate of movement	5 mm/wk
NSW - Northern	40	Rate of movement	10 mm/wk
NSW - Northern	40	Rate of movement	1 mm/wk
NSW - Western	35	Total movement	25-50 mm ¹

¹Additional actions at 100 mm and 150 mm of total movement.

A number of different devices were used for roof monitoring, including an electrical system that uses a rheostats and was read by a tiny, hand-held LCD. Sonic probes are losing favor in some circles because of their price (\$12,000 US), but they are still widely used for research and to determine trigger levels for tell-tales.

A most impressive monitoring system is being developed by Strata Control Technologies and Angus Place Collieries. It is an entirely permissible system that can be easily installed by the bolting crew. Each new station "daisy chains" to the previous one, and automatically identifies and initializes itself. The entire

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system can be tied into a computerized mine-wide monitoring system. The goal is to eliminate the need for specialists to install, read, and plot the data.

Microseismic Monitoring: Microseismics are being used for a number of issues in Australia. In the Lake Macquarie area, massive conglomerate main roof is a major concern for at least three longwall mines. At Newstan Colliery, extreme periodic weighting two years ago caused face falls that cost more than 6 months of production. Longwall panels at Newstan have since been limited to less than 150 m in width. The problems with periodic weighting at Newstan have already been mentioned. At the nearby Moonie Colliery, wind velocities of 40 m/sec were measured during a windblast event. Both mines are now using a South African microseismic monitoring system to warn workers of impending airblasts.

A 1998 paper by Kelly, et. al., describes microseismic studies at Appin and Gordonstone Collieries aimed at predicting water and gas inflows resulting from overburden response to longwall mining.

MINES INSPECTORATE AND SAFETY MANAGEMENT

According to Ian Anderson, Senior Mine Inspector in NSW, the basic qualification for an Inspector is actual experience as a Mine Manager. A degree in Mining Engineering is preferred, as is experience in mine rescue. The high level of training is necessary because Australian mining law gives the Inspectorate much more discretion than in the U.S.

There are few statutory safety requirements in either Queensland or NSW. Instead, the Coal Mines Regulation Act requires each Mine Manager to develop a set of rules that are appropriate for his operation. For example, Manager's Rules on roof bolting would include criteria as to when and where different bolt patterns are required. The Manager's Rules must be approved by the District Coal Mines Inspector. In addition, the plans for each new panel are submitted for approval.

In recent years, the trend has been to upgrade the Manager's Rules into Safety Management Plans (SMPs). SMPs were one of the recommendations that came out of the Moura disaster. The SMPs are Quality Assurance-type documents that are typically prepared for Spontaneous Combustion, Gas, Ventilation, Strata Control, Evacuation, Outburst, and Water (Reece, 1998). The SMPs define the range of hazards associated with each issue, and the necessary controls and procedures for management of the hazards. SMPs are very detailed. For example, the Strata Hazard Management Plan for the North Goonyella mine is 44 pages long. A typical format is:

- Hazard Identification and Control Procedure
- Monitoring Scheme
- Roles and Responsibilities

- Action Response Planning
- Training Requirements
- Corrective Action
- Review Procedures

The SMPs provide auditable procedures that allow stakeholders (workforce, mine management, shareholders, and Inspectorate) to scrutinize whether the objectives are being met. Of particular interest is that the SMP clearly defines the roles of every individual at the mine, when monitoring is to be used, and what the action triggers will be (Byrnes and Doyle, 1997).

MINING RESEARCH

Australia has an extremely vibrant, successful, and confident mining research community. In part, this may derive from the rapid growth of the industry over the past 30 years. The structure of mining research funding also facilitates the success of mining research.

The primary source of research funding since 1978 has been a \$0.05/tonne levy on all black coal production. Under a Memorandum of Understanding with the Australian Commonwealth, the levy is administered by the Australian Coal Association through ACARP. ACARP identifies high-priority problems, and solicits research proposals in those areas. The proposals may come from government agencies, consulting groups, Universities, or mining companies. Cooperative efforts, with cost sharing from the private sector, are looked upon most favorably. Cost-sharing leverages the funding for a typical research project by a factor of 2.5 (Graham, 1998).

The ACARP approach has a number of significant advantages. The research is customer driven, not researcher driven. It helps researchers stay focused on real problems, and it helps keep consultants and company staff abreast of the latest technology. Currently, 68 mine sites are involved in research projects, and 114 company staff are involved in ACARP, either by participating in the project selection process, or as Project Monitors. It has been highly successful in breaking down barriers between government researchers, academics, and mining staff. The breadth of knowledge about mining research at all levels in the industry is truly admirable.

Probably the most significant advantage of the ACARP system is technology transfer. Because the ultimate customers are intimately involved with the entire research process, technology transfer begins almost with the inception of the project. Technology transfer is also built into every project. The result is a large number of successful products, and a dramatically reduced time lag between research and implementation.

Approximately 40% of ACARP's research budget is spent on underground mining (the rest is split between surface mining, preparation, and utilization). Strata control

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is the biggest single research area, with an annual funding in excess of \$500,000. Current research areas include rib support techniques, roof monitoring, windblasts, and replacing primary support with secondary support.

One criticism of ACARP could be that the miner's union and the regulatory agencies do not have direct input into the project selection process. However, it seems that safety is still the primary research driver. Perhaps the reliance on self-policing through "manager's rules" has carried over into the industry-sponsored research program.

CONCLUSIONS

The first, obvious conclusion is that there are no unique "Australian" ground conditions, any more than there are unique "U.S." conditions. A wide variety of conditions are found in both countries. The mines under deep cover and high horizontal stress in the Southern NSW coalfield do seem to suffer more severe problems than are typical in the U.S., but U.S. counterparts come to mind for most of the other Australian mines.

The typical Australian mine uses a higher density of primary support than is common in the U.S. Three factors may explain this:

- Australian mines are less tolerant of roof falls;
- Australian mines have traditionally relied less on supplemental support in the tailgate; and,
- in Southern NSW, lower pillar stability factors may require more entry support.

It seems that there is currently some trend in Australia towards reducing primary support in favor of secondary support.

Another trend in Australia is towards experimentation with place-changing systems. However, the combination of low CMRR roof, two-entry mine layouts, inappropriate equipment, and a low tolerance for roof falls may limit the applicability of place changing. In many instances, the miner-bolter development system used in a number of Pittsburgh seam longwall mines may be a better U.S. benchmark.

There is also quite a bit that we in the U.S. might learn from Australia. Roof monitoring technology is far more advanced in Australia, for example. The Australian approach to safety management and integrating mining research into the industry also have a lot to commend themselves.

Finally, there are many areas in which we can learn from each other. Both countries have recently made valuable contributions in roof support technology, pillar design, and numerical modeling that are readily transferable across the Pacific. More interactions like this Conference could greatly benefit both countries.

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Application of Microseismic Monitoring to Longwall Geomechanics and Safety in Australia

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ABSTRACT

In this paper we report on the use of microseismic monitoring techniques to investigate the fracturing process associated with longwall mining. The study was carried out at three mines in Australia and concentrated on some major concerns of the mines, such as failure patterns and growth rate associated with mining progress, their implications to water ingress and gas emissions, caving processes in the goaf and precursors of roof falls. The results have shown that a zone of shear fracturing has occurred further ahead of the longwall face than predicted using conventional longwall geomechanics theory. Failure patterns in the roof and floor vary from mine to mine and this appears to be dominated by geological structures. The reaction of faults, the formation of fractures which allow gas emissions and precursors roof falls have also observed. Results from this work have opened up new understanding of longwall caving processes and proved that microseismic monitoring is a very useful technique for studies of longwall geomechanics and hazard controls.

These types of problems are common in many countries and restrict the production potential of modern longwall faces. As most of the problems are directly or indirectly resulted in fracturing of rock mass, there needs to be an improvement in the understanding of longwall fracturing processes.

There have been many studies into the geomechanical behaviour of longwall faces using different approaches and underlying assumptions. Application of microseismic monitoring technique to the longwall problems is a relatively new practice. This technique has been successfully used in metalliferous mines for many years for the determination of unstable regions. Styles et al., (1992) applied microseismic monitoring to longwall coal mines in Britain and located rock failures in space and time which were associated with mining activities. Their results demonstrated that microseismic monitoring may become a potential tool for mapping longwall caving processes. We have further developed this technique and applied it to monitor the details of micro-fracturing processes associated with longwall mining in Australia.

INTRODUCTION

Underground extraction of coal in Australia is becoming increasingly reliant upon the longwall method. There are many complex and inter-related issues in longwall mining. Despite the rapid growth in Australian's longwall production, many longwall mines have experienced major geotechnical problems. Unexpected geological intrusions and fault zones have resulted in loss of production over extended periods in some mines. Cyclic loading under massive strata and the associated problems with face and pillar stabilities and windblasts have occurred. Roof falls near the working face and high gas emissions have resulted in noticeable production delays. Water ingress from overlying aquifers or watercourses has also been problems. Subsidence of environmentally sensitive areas, residential areas, railways, roads, water bodies and dams are some of the other problems which have restricted the longwall operations.

AN EXPERIMENT IN WEAK STRATA AT GORDONSTONE MINE

Gordonstone Mine was the largest underground coal mine in Bowen Basin of Central Queensland. In the area of study the 3 m thick German Creek Seam was being mined by the retreat method at a depth of about 240 m. The typical longwall panel was 250 m wide and about 1700 m long. The immediate roof and floor are weak, with UCS values ranging 5-15 MPa (Kelly et al., 1994). Typical roof strata consist of interbedded sandstone and siltstone. Above the seam is 170 m of Permian formations with a further 70 m of Tertiary sediments containing aquifers.

In 1994 a microseismic monitoring system was installed at a longwall panel 103 (LW103) in order to establish the height of longwall caving and the potential for fractures to breach the Tertiary boundary (Hatherly et al., 1995). In order to accurately map the height of fracturing, three boreholes in a triangular

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arrangement and spaced at about 350 m were drilled from the ground surface down to the floor of the German Creek Seam. In each of the boreholes nine triaxial geophones were grouted at different depths. Fracturing which occurred in the roof between the coal seam and ground surface and within or near the network was detected using this configuration.

The field monitoring started on August 8, when the longwall face was 300 m away from the microseismic network, and terminated on November 15 when the face approached the end of the panel. During this period the longwall face advanced about 700 m and more than 530,000 m³ coal was extracted. Approximately 1,900 microseismic events were detected. The seismicity is clearly correlated with the progress of longwall production. Figure 1 shows the locations of 629 events recorded from Sep 16 to Oct 29.

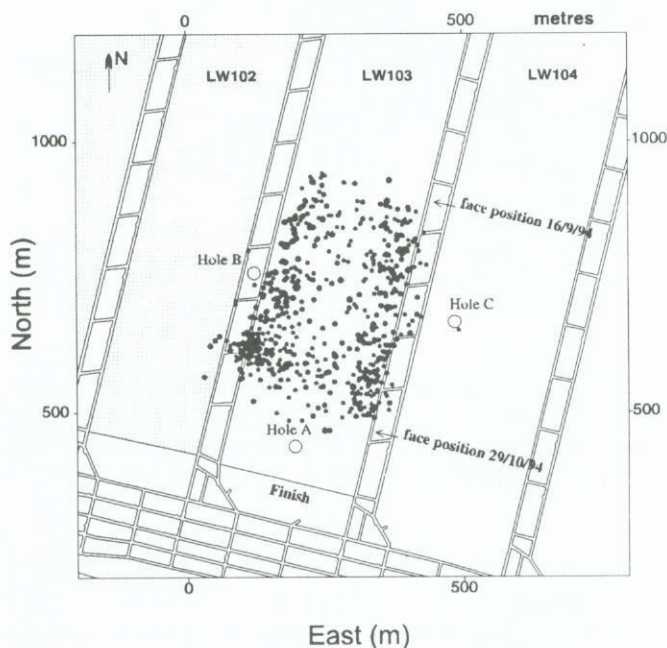


Figure 1 Locations of the three geophone boreholes, A, B and C and seismicity from Sep 16 to Oct 29, 1994 in plan view of longwall 103 (LW103) at Gordonstone Mine. LW102 had been mined and LW104 was in development. The finish line is where LW103 mining was terminated.

Except for a few events, the majority of the events occurred within the longwall panel being mined. There is a symmetrical concentration of events to the sides of the panel (the tailgate and maingate) with a noticeable decrease in the centre. Figure 2 shows the events projected on vertical cross section looking northeast along the longwall panel. As with the plan locations in Figure 1, the rock failure defined by the zones of seismicity is symmetrical. The failure above the tailgate and maingate are confined by envelopes of the events at angles of about 16° from the vertical. Most of the events are located in a region extending from 20 m in the floor to 125 m in the roof. The maximum height of fracturing in the roof is still 45 m below the Tertiary boundary. Hence, no breaching into the Tertiary aquifers due to mining induced fracturing is expected.

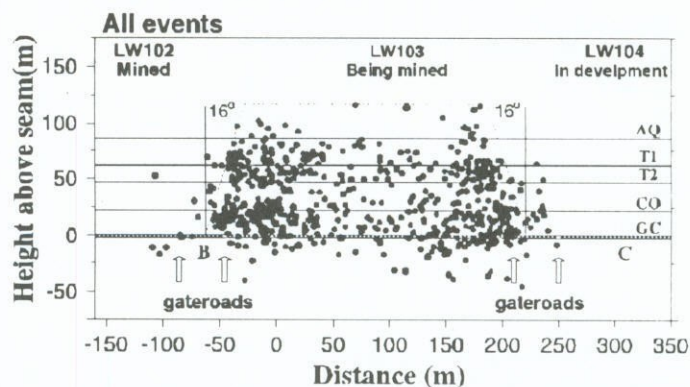


Figure 2: Microseismic events on vertical section looking northeast along LW103. GC denotes the German Creek Seam. AO, T1, T2 and CO are thin coal seams. B and C represent the locations of boreholes B and C. A failure zone in the roof is defined by the dashed lines.

Figures 3 and 4 shows the total seismicity normalised to a fixed longwall face in plan view and on vertical section side on to the longwall panel, respectively. It is apparent that an arcuate fracturing zone is 0-70 m ahead of the face at the seam level. It passes obliquely at about 50 degrees from horizontal back over the face. Source mechanisms determined using a moment tensor solution for selected events of the zone show dominant shear failures in the form of reverse faulting. The angle of the fault plane was determined to be about 50° from the horizontal, consistent with the dip angle of the fracture zone ahead of the face. Surprisingly, very few events are located in the goaf area where tensile fracturing is expected.

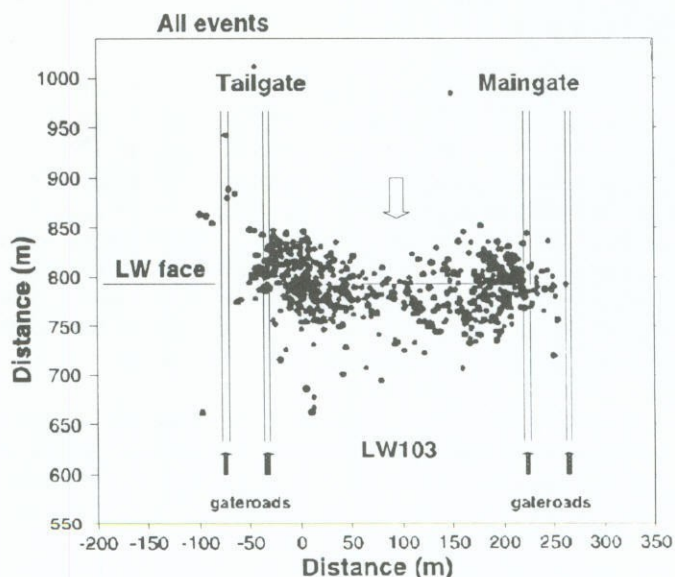


Figure 3: Plan view of event locations relative to a fixed longwall face position. The longwall retreat direction is marked by the arrow.

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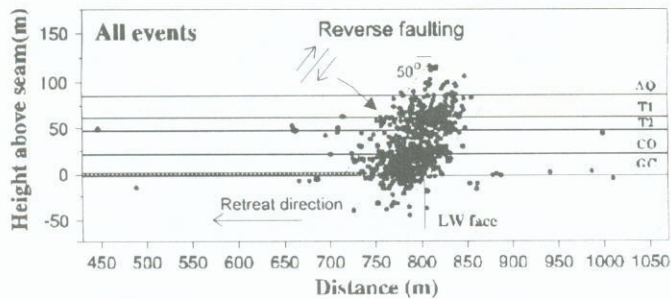


Figure 4: Vertical section of event locations relative to a fixed longwall face position, looking northwest. The fracture zone ahead of the face is defined by the dashed lines. Failure by reverse faulting occurred.

AN EXPERIMENT IN STRATA OF INTERMEDIATE STRENGTH AT APPIN COLLIERY

Appin Colliery is located in the Sydney Basin. The Bulli Seam of 2.3 m thick was extracted using the retreat method at a depth of about 450 m. Typical roof and floor strata consist of interbedded strong sandstone and siltstone with their UCS values of 50-80 MPa. At Appin the longwall production was delayed occasionally due to excessive gas emissions. The gassy coal seams, Wongawilli Seam, 9 m thick and 35 m below the Bulli and the Tongarra Seam, 2 m thick and 64 m below are considered to be the potential sources of gas emissions.

A microseismic monitoring was made of longwall panel 28 (LW28) to determine the nature of rock failures in strata of intermediate strength and to detect whether fractures which allow gas emissions are developed between the Bulli and Tongarra seams.

The microseismic monitoring commenced in July 1996 before the longwall started and completed on November 14 when the face was 100 m from the end of the panel. 17 triaxial geophones was deployed with nine geophones grouted in a vertical borehole at different depths from the ground surface down to 450 m near the Bulli Seam. The remaining eight geophones were installed in 10 m deep boreholes along two perpendicular survey lines within the panel. During the monitoring period more than 750,000 tonnes of coal were extracted and more than 3,000 events were recorded.

Events recorded from September 3 to November 14 were processed. The events are characterised into two groups in terms of their frequency contents. The first group of events were characterised by lower frequencies with dominant frequencies ranging from 10 - 50 Hz. There are 1,175 events in this group. The second group of 313 events has dominant frequencies of about 60 -150 Hz.

The locations of the events in plan view are shown in Figure 5. A separation of the low and high frequency events is immediately apparent. The majority of the low frequency events are located near the tailgate side of the panel and show a cyclic failure at an interval of about 100 m. Some low frequency

events were located in the goaf area and gateroad pillars of previously mined panels. While the high frequency events are located near the maingate side in a joint-shear zone adjacent to the borehole.

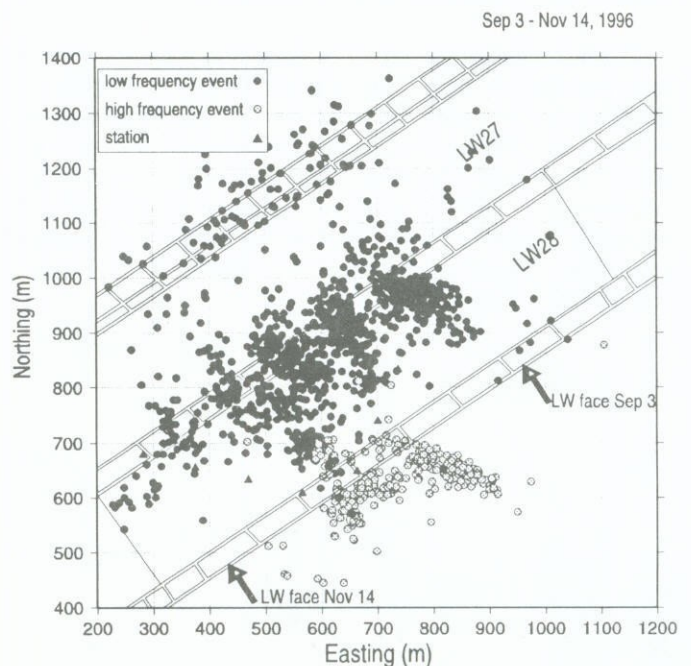


Figure 5: Locations of microseismic events from Sep 3 to Nov 14, 1996 in plan view of LW28 at Appin Colliery. LW27 had been mined. Black circles are low frequency events and grey circles are high frequency events.

The projection of the events on vertical cross section looking northeast along the longwall panel is shown in Figure 6. The low frequency events were well developed near the tailgate side in the roof and floor. Evidently, extensive fracturing has cut through the Wongawilli Seam and some has extended down to the Tongarra Seam. The fractures may form paths for gas emissions.

The high frequency events are mainly located in the immediate roof near the maingate side, with no seismic indication of significant rock failures between the Bulli and Wongawilli seams there. Therefore, gas emissions from the Wongawilli and Tongarra seams along the induced fractures may only occur near the tailgate side. A more efficient gas drainage arrangement may be possible based on the seismic results.

The low frequency events near the tailgate are controlled by the face position and they moved forward with the face advance. Figures 7 and 8 shows the total seismicity normalised to a fixed longwall face in plan view and on vertical section side on to the longwall panel, respectively. Fracturing further ahead of the face seen at Gordonstone (Figures 3 and 4) has also been observed at Appin. In this instance there is evidence of

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stimulated failures associated with the previously mined LW27 panel. An arcuate fracturing zone of 0-70 m ahead of the face is apparent. This is in agreement with the result from Gordonstone except that it is not symmetric within the longwall panel. On the vertical cross section an arc line confining the front of extensive fracturing zone is apparent. Again few events were located behind the face.

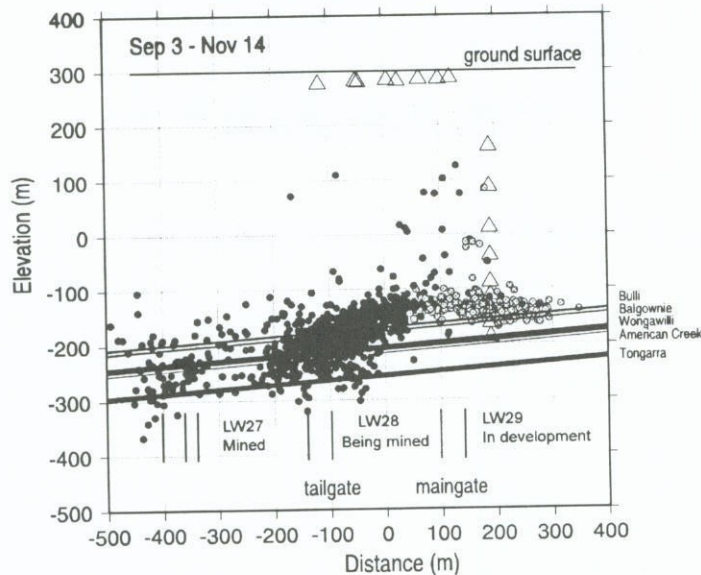


Figure 6: Microseismic events on vertical cross section looking northeast along LW28. The triangles are geophone stations.

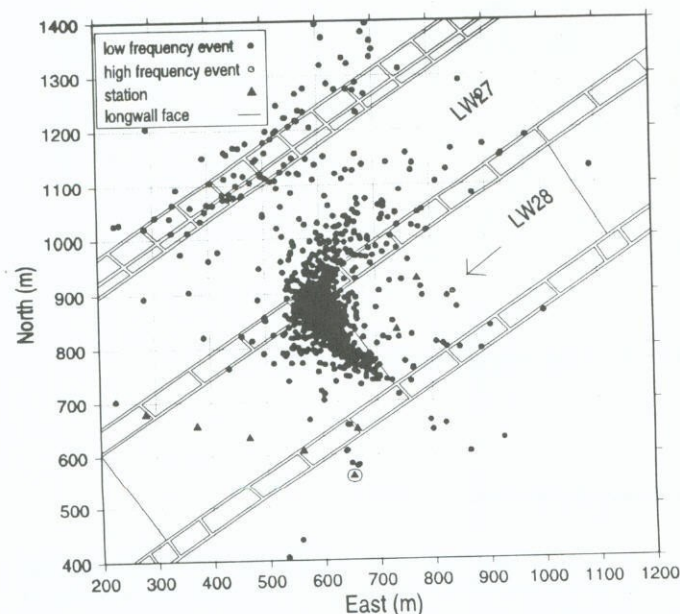


Figure 7: Plan view of event locations relative to a fixed longwall face position. The longwall retreat direction is marked by the arrow.

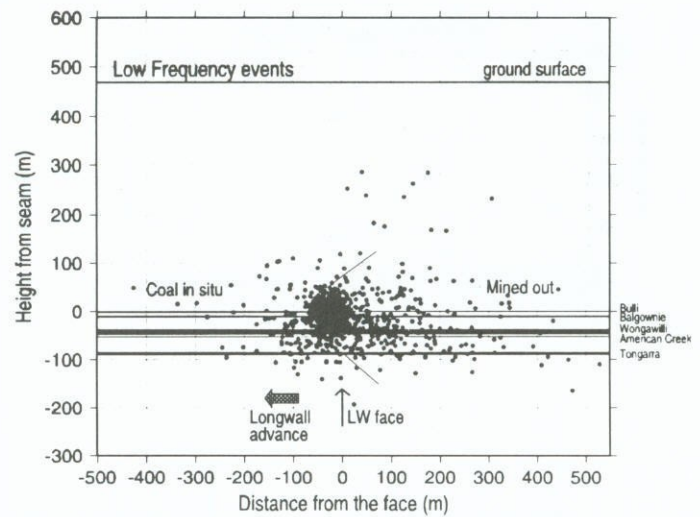


Figure 8: Vertical section of low frequency event locations relative to a fixed longwall face position, looking northwest. The fracture zone ahead of the face is defined by the parabola.

At a nearby mine in the Bulli basin area, high-precision strain monitoring system has been operating for more than four years to provide long term monitoring of the strain variations induced by longwall mining operations beneath the Cataract Reservoir (Gladwin et al., 1998). The system provides direct and real time measurement of the deformations induced by the rock caving during long wall extraction sequences. It therefore complements the information provided by the elastic failures mapped by the microseismics.

The system measures the horizontal components of both the deformation and the tilt fields in the region, including deformations due to slow creep processes or to plasticity in the rocks for comparison with mine plan model expectations. Figure 9 located in the third panel of a longwall sequence, shows the expected cross panel compression at a level of about 80 microstrain following local cross panel tension as the previous panel was removed. As subsequent panels were mined the maximum crosspanel strain decreased till subsidence was complete.

The total mining cycle was accompanied by significant rotation of the principal strain axes with time. The effect is shown in Figure 10 where the changes of principal strains relative to the undisturbed state against the progression of the mining sequence are plotted on a map view of the longwall panel. The mining in this case proceeded from east to west in panels 501 to 508. Though the potential for direct measurement of large scale induced strain response as mining occurs in combination with the microseismics measurements here reported has not yet been demonstrated, it is likely that together the two techniques will capture all key features of the dynamic loading associated with longwall mining.

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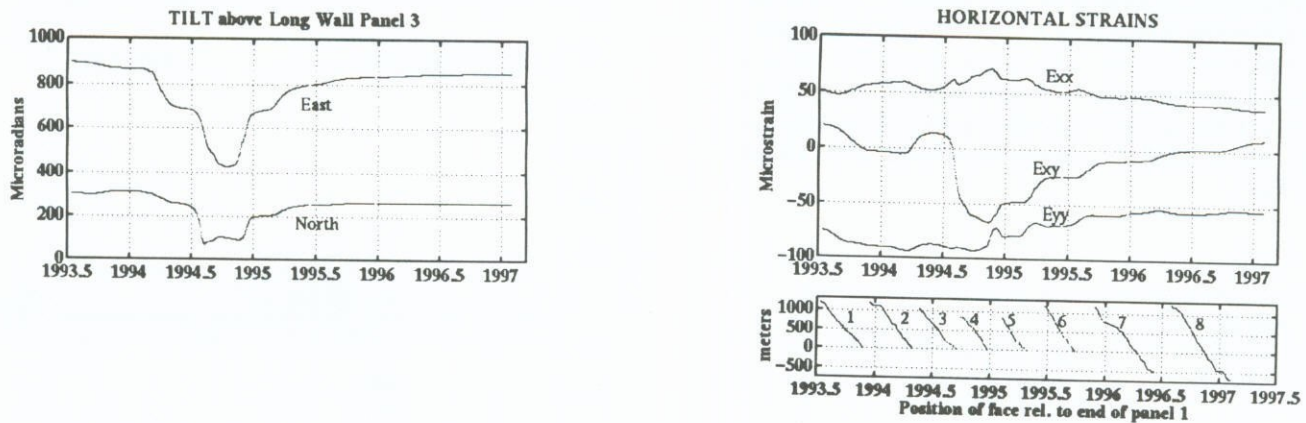


Figure 9: Mining induced strains and tilts above longwall panel 3 during the extraction of the first eight panels.

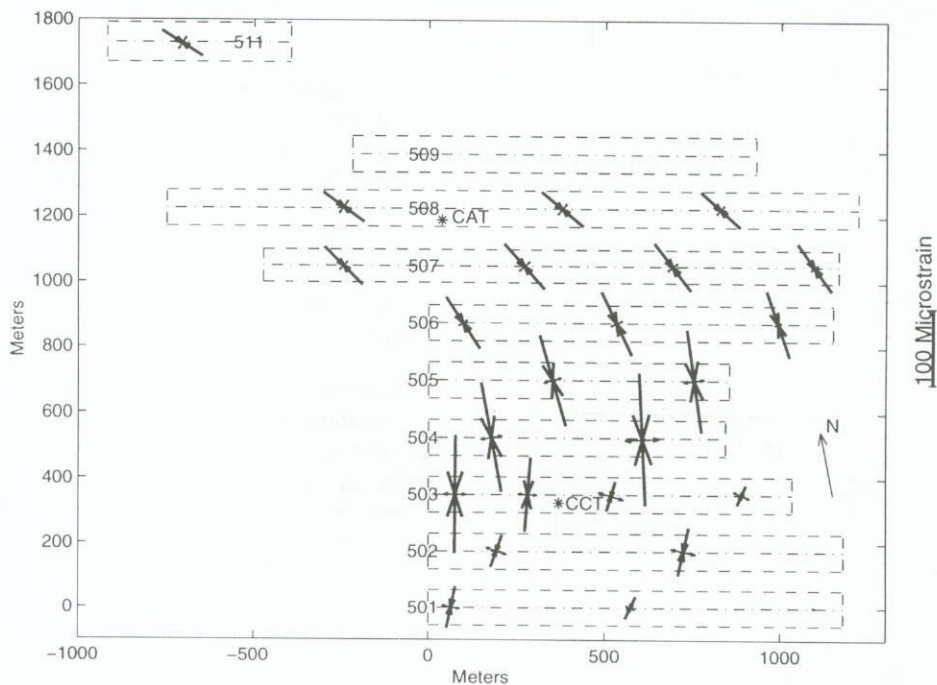


Figure 10: Principal strains observed at CCT site 350m above the longwall panels for various positions of the extraction sequence.

AN EXPERIMENT AT NORTH GOONYELLA MINE

North Goonyella mine is located to the north of Bowen Basin. The Goonyella Middle Seam of 4.4 m thick was mined by retreat method about 170 m below the ground surface. The immediate roof and floor strata consist of weak interbedded siltstone and sandstone. Two faults cut across from longwall panel 2s (LW2-S) to panel 4s (LW4-S) and the mine suffered significant loss of production due to roof falls when they mined

through the fault area in LW2-S.

In order to improve understanding of roof fracturing and caving processes of the fault zone, a microseismic monitoring was carried out at LW3-S. Three boreholes were drilled, and four triaxial geophones were installed in each. The borehole locations were arranged to be similar to those of Gordonstone. In order to capture the details of fault fracturing, two boreholes were located in the fault zone and the tailgate and maingate

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pillars. The monitoring commissioned in November 1996 and finished in January 1998. More than 3,000 seismic events were recorded. During this period LW3-S was suffered significant delays of production due to successive roof falls near the working face.

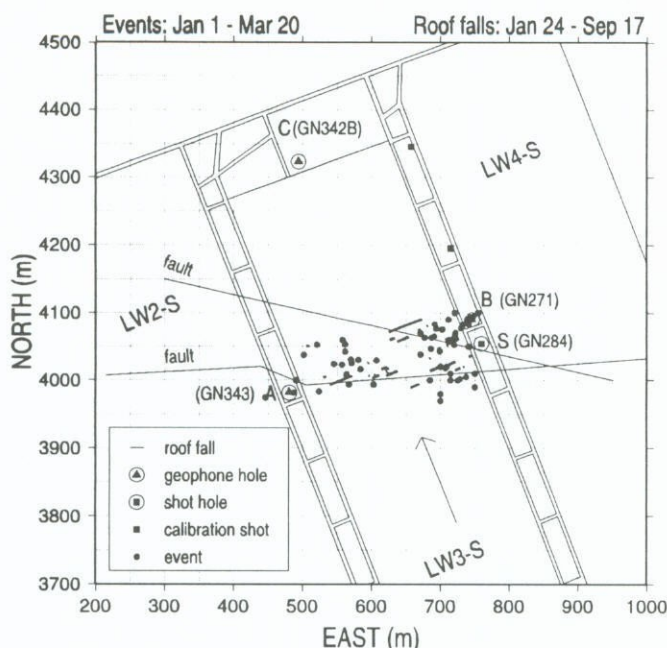


Figure 11: Locations of seismic events and roof falls in plan view of LW3-S. LW2-S had been mined and LW4-S was in development. Locations of three geophone boreholes are denoted by A, B and C.

The seismic signals recorded at LW3-S are very weak, indicating that rocks in the fault zone are highly fractured and this results in strong attenuation to seismic energy. Events recorded before July were processed. The locations of the 54 largest events which occurred from January to March were obtained. Most of the rest events have very poor signal to noise ratio and cannot be reliably located. Figure 11 shows the locations of the 54 events and the roof falls which occurred from January 23 to September 17. All the seismic events are located in the roof fall region that is roughly confined by the two faults. As the seismicity generally occurred before the occurrence of the falls, it might be possible to use microseismic monitoring to predict roof falls at the mine.

DISCUSSIONS

Fracturing processes associated with longwall mining were observed at Gordonstone, Appin and North Goonyella Collieries using microseismic monitoring. The seismicities were directly resulted in longwall mining activities. Shear failures have been observed more than 50 m ahead of the face at Gordonstone and Appin and they are dominated by reverse faulting mechanisms. This is further ahead of the face than

predicted using conventional numerical modelling to the same panel, that indicated failure should occur about 10-15 m ahead of the face (Kelly et al.⁵). The maximum height of fracturing at Gordonstone is still 50 m below the Tertiary boundary and thus water ingress from overlying aquifers is not expected at the mining site.

A cyclic failure with an interval of approximately 100 m along the tailgate side of the face has been observed at Appin. Apparently, fracturing was concentrated near the cross cuts of the pillars. Extensive fracturing occurred in the floor of Bulli Seam and near the tailgate side. It could allow gas emissions from the underlying Wongawilli and Tongarra seams to the working face. This result can be very useful for optimising a gas drainage plan to reduce high gas emissions. A joint-shear structure near the maingate side has been mapped by microseismicity. It appears to be feasible to detect unstable minor faults or joints using microseismic monitoring.

At North Goonyella the larger seismic events are all located in a fault zone where extensive roof falls occurred. The seismicity implies new fracturing was induced in the zone and that the region was in an unstable condition. The occurrence of the events before the roof falls suggests that microseismicity may be used to give warning to the occurrence and locations of roof falls.

Differences of failure patterns have been observed at the three mines. For example, the symmetric failure within a longwall panel observed at Gordonstone was not observed at Appin or North Goonyella. Rock breakage in the floor is more extensive at Appin than at Gordonstone. These differences appear to be attributable to different geological conditions.

That very few events were located in the goaf area behind the face was a surprise to us. The seismic energy released from rock failures ahead and behind the face appears to be very different. Unlike failure ahead of the face which is dominated by shear fracturing, tensile fracturing, detachment of pre-existing fractures or separation of bed planes should be dominant failures behind the face. Events with these source mechanisms normally are much weaker than shear events. As a matter of fact, approximately 70% of data collected from the three mines have not been processed due to the low amplitude and poor signal to noise ratio. We are developing new methods in order to allow the weak events to be located.

Microseismic monitoring is a very efficient technique for detecting brittle failures of rock mass. Measurement of strata deformation induced by longwall mining will complement information provided by microseismic monitoring.

CONCLUSIONS

This paper illustrates the application for microseismic monitoring at three coal mines. The technique allows dynamic mapping of the fracture processes associated with longwall mining and is providing invaluable information for improving

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mine design, productivity and safety. The success of our microseismic monitoring experiments at the mines lies partly in utilising arrays of triaxial geophones within vertical boreholes. This has allowed very precise mapping of microseismic events. Results from this work have opened up new understanding of longwall caving processes and proved that microseismic monitoring is a very useful technique for the studies of longwall geomechanics and production safety. It is our intention in future to combine microseismic observations of stress induced failure mechanism with direct observation of deformation patterns through strain and tilt monitoring. This will provide a total picture of the dynamic creep and failure patterns associated with longwall mining.

ACKNOWLEDGMENTS

Robert Dixon (CSIRO) designed and constructed the microseismic monitoring system. Greg Poole (BHP) supervised the geophone installations. Justin Ross (CSIRO) provided processing assistance. ACARP, CSIRO Exploration & Mining BHP Australia Coal, Gordonstone Mine, Appin Colliery and North Goonyella Mine have provided funding and assistance for this work.

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Control of Hard-to-Collapse Massive Roofs in Longwall Faces Using a Hydraulic Fracturing Technique

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ABSTRACT

Periodic weighting and windblast which are caused by the dynamic effects of overburden in a longwall panel significantly affect longwall mining.

This study describes past experience with wind blast at Miike Colliery, proposes a new loosening system using hydraulic fracturing and moistening, and also discusses strength deterioration in hard-to-collapse roof rocks due to water in order to solve the problems caused by the effects of dynamic roof rocks behavior in a longwall face.

INTRODUCTION

In certain underground coal mines where the roof consists of strong and massive rocks or hard-to-collapse rocks, the roof rocks do not always cave in regularly as longwall extraction proceeds, but roof overhanging does occur, leading to extensive areas where the roof is unsupported in the gob. Here, the dynamic effects of roof rock behavior present difficulties in a highly mechanized longwall face. The major problems are periodic weighting and windblast.

Periodic weighting which is caused by the main roof overhanging the gob area sometimes leads to instability in the longwall face and the failure of powered supports.

Windblast is caused by the collapse of the hanging roof in the gob, which compresses the air beneath it and forces it out of the

gob through the gaps between the powered supports. Windblast sometimes occurs suddenly and without warning. The force of the wind can and sometimes does cause fatalities and injuries in the face area and damages plant and equipment. It can also increase the possibility of explosions by expelling methane from the gob and mixing it with raised coal dust.

Conventional techniques using longhole blasting have been used, in order to prevent these dynamic effects in faces with hard-to-collapse roofs. However, rock loosening using this method is not effective and drilling is rather dangerous.

This study briefly describes windblast at Miike Colliery. In order to control hard-to-collapse roofs in longwall panels, a new hydraulic fracturing technique for rock loosening is proposed and the characteristics of the strength deterioration of hard and strong sandstone roofs due to water are also analyzed by means of laboratory tests.

EXPERIENCE AT MIIKE COLLIERY

Miike Colliery, the largest and undersea coal mine in Japan, closed down in March, 1997, because of adverse market conditions. During its operation, dynamic effects such as coal burst and windblast occurred in certain longwall panels.

Geology

The formations in the Miike coalfields are from the Pre-tertiary, Tertiary and Quaternary ages. Within the Tertiary

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formations, there are a total of nine coal seams, of which three are workable. These are known, respectively in descending order, as the Second Upper Seam, Upper Seam and Main Seam. The principal dip of the seams is between 2° and 5° to the southwest – they go deeper as they go further off-shore. There are numerous faults in this mining area. Most of the faults are normal type ones with a tendency to strike east. There is about a 50 m thick strata between the Main and Upper Seams, and 140 m between the Upper and Second Upper Seams. A general schematic section of Miike Colliery is shown in Figure 1.

Figure 2 shows the lithology of the roof of the Second Upper Seam in the West-80 area as shown in Figure 3. The vertical and horizontal changes in lithology are clearly seen in the roof. A distinct massive sandstone unit over 10 m thick is present above the Second Upper Seam.

Mining Activities

Miike Colliery produced 5.3 Mt in fiscal 1980 and 2.3 Mt in 1991. This drastic fall in production was primarily due to the widening price gap between domestically produced and imported coal and the national plan for domestic coal production. Finally, Miike Colliery, the biggest coal mine in Japan, closed down in March, 1997. Before shutting down, it had been making extensive efforts to survive including the introduction of a new longwall mining system in order to augment coal face productivity to the maximum. The retreat longwall method with re-use of gateroads was used traditionally mainly because of a higher level of recovery. However, from the perspective of productivity, this method was not suitable, requiring intensive work for face-side packing and gateroad maintenance. Therefore, a new mining system, or an interpanel-pillar system using higher

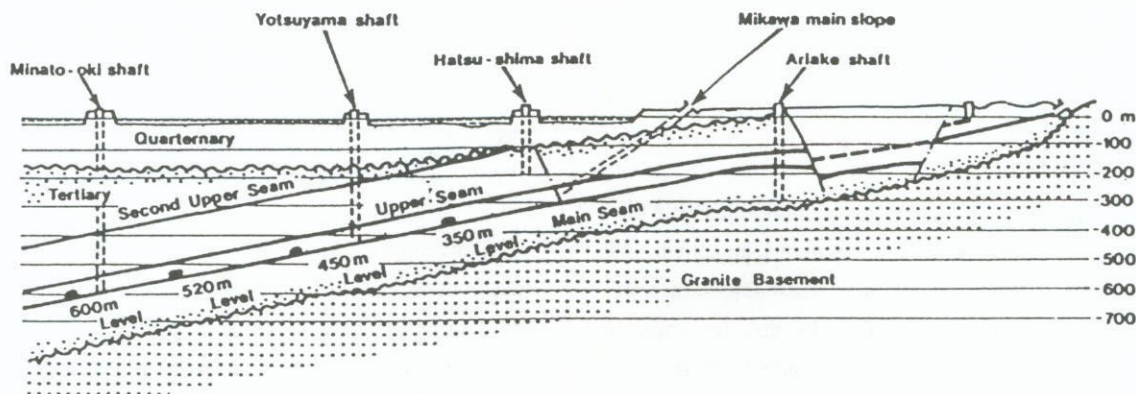


Figure 1 Schematic cross-section of Miike Colliery, showing simplified geology.

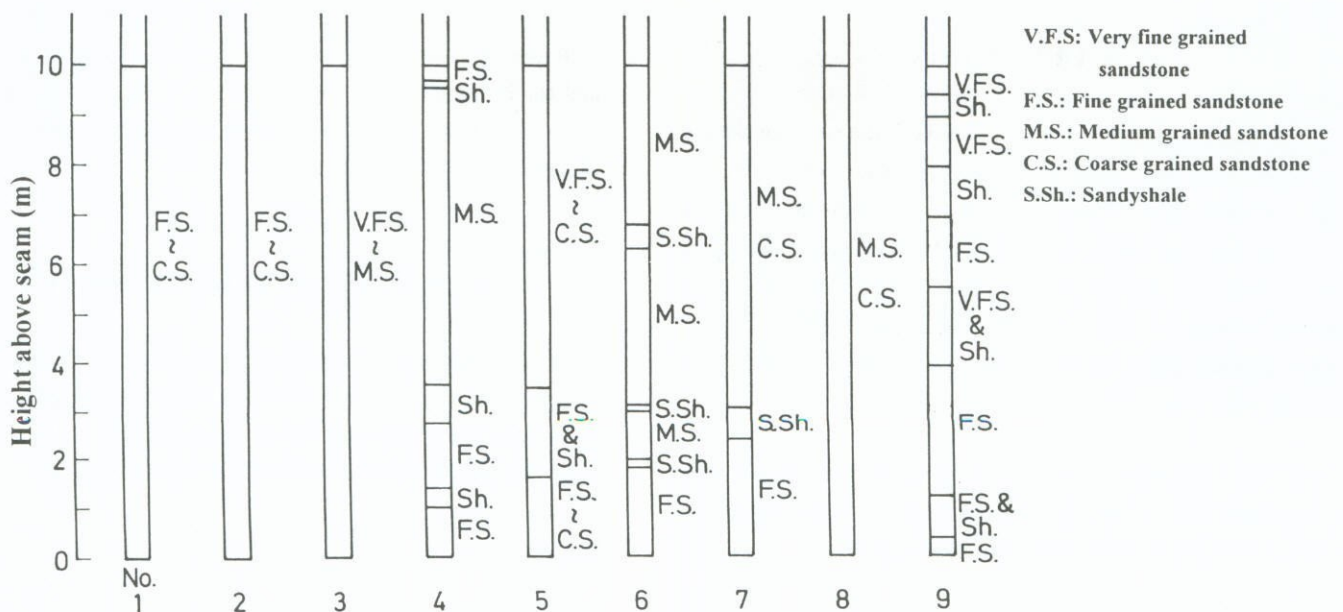


Figure 2 Lithology of the roof of the Second Upper Seam in the West-80 area.

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capacity longwall equipment was adopted in order to enhance longwall mining productivity. Here, rapid drilage and the effective maintenance of gateroads are of critical importance in regard to safety, economy, and efficiency. Along with standing supports such as steel arches or three piece sets, strata bolting system was used in order to increase drilage performance and reduce the cost of drilage and maintenance of gateroads.

Wind blast in the West-80 area

A big windblast occurred on the face of the West #11 longwall panel, as is shown in Figure 3. The West #11 panel began to retreat in 1995 in the Second Upper Seam coal and good production rates were achieved until wind blast occurred in the face. In this panel, the Second Upper Seam is at a depth of about 504 m below sea level, and there are no panels in the other two seams that are below it. The seam thickness varied between 4.0 m and 4.3 m. The seam is overlain and underlain by

sandstone, but underlain locally by shale.

The face was 150 m wide and cutting 2.8 m, leaving a floor coal that was 1.2-1.5 m thick unworked.

The windblast occurred after the face had retreated some 90 m. The air in the gob was compressed and forced out through the gaps between the powered supports. The force of the wind injured two people in the face. Longwall mining had to be stopped for about one month. It was apparent that the windblast was caused by falling roof rock in the gob which is related to periodic weighting from the overlying strata. Coal bursts also occurred in other parts of the mining area (Sugawara et al., 1987).

Roof strata behavior

The problem of periodic weighting has already been identified

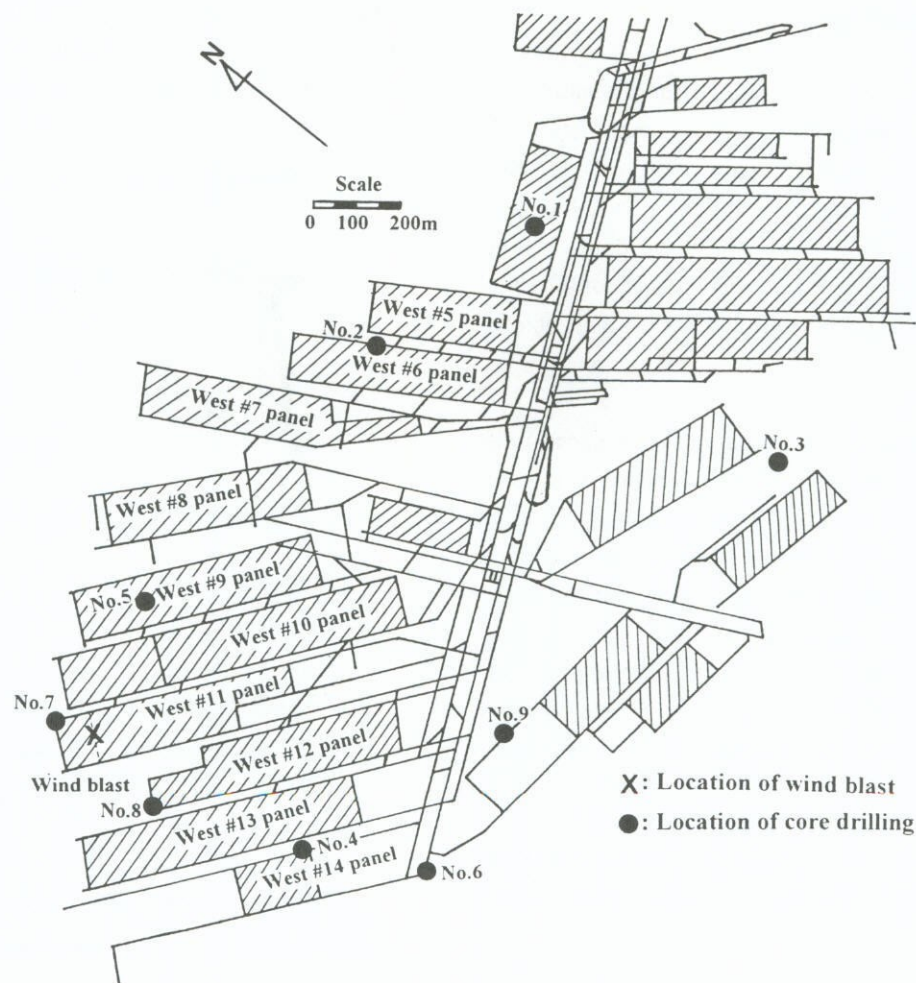


Figure 3 Panel layout and location of core drilling and windblast in the West-80 area.

as being related to the presence of hard-to-collapse strata or strong and massive strata in the immediate roof (Frith, 1996). The massive overlying strata cantilever over the supports into the gob and pivot about a point that is located ahead of the face. This can lead to fracturing of the strata ahead of the faceline, resulting in face falls and cavities developing to considerable heights above the face area. No amount of increased powered support capacity will prevent this problem from occurring. When the immediate roof consists of more friable or weaker strata such as shale, coal and mudstone, it will readily cave in behind the supports and bulk up sufficiently to fill the gob and isolate any failures in the overlying strata from generating a wind blast which would reach the mine workings. However, when the **thickness of the immediate roof reduced, there is insufficient bulking**, resulting in large, continuous voids in the gob extending back from the face. Any subsequent failure of the main roof, or even large scale slabbing from the main roof, would then have the potential to create a windblast on the face.

Countermeasures against wind blast

The colliery had been taking countermeasures against roof overhanging in the gob. In regard to a strong immediate roof, forced caving by blasting was used. Figure 4 shows the blasting pattern at the setup entry before starting the retreat longwall mining. The blast holes, 25 mm in diameter are drilled 1.8 m long vertically and with an upward inclination angle of $60-70^\circ$ and 1.5 m apart. The amount of explosive in each blast hole is $250\text{g} \times 4$.

After the face starts and advances, artificially induced roof caving occurs behind the supports. After that, caving conditions are observed and checked regularly. When there is no caving in the gob, the boreholes are drilled and blasted between the powered supports along the faceline, as is shown in Figure 5. The blast holes, 25 mm in diameter are drilled 1.8 m long with an upward inclination angle of $60-70^\circ$. The amount of explosive is the same as at the setup entry case. In addition, the blast holes are drilled from both the head- and tailentries ahead of the face. The holes, 25 mm in diameter are drilled 2.4-2.7 m long into the roof with an upward inclination angle of $60-70^\circ$, and about 0.5 m apart. In this blasting pattern, explosive charges of $250\text{g} \times 5$ or 6 in each hole are used to create fractures in the strong roof.

The disadvantages of this blasting method are as follows:

- 1) Rock loosening with this method is uneven: near the hole the rock is overfractured, while at a given distance it is destroyed by isolated cracks.
- 2) The effective range of the blast is small; 5-25 radii of the explosive charge.
- 3) Drilling operation is hard and dangerous.

HYDRAULIC FRACTURING TECHNIQUE

Hydraulic loosening is performed through boreholes by hydraulic moistening and fracturing. When water is pumped into a section of the borehole that is isolated by packers, some water penetrates into the surrounding strata. As the water pressure increases and reaches the maximum stress level that the strata can sustain, a crack is formed. If pumping is continued, the crack will extend.

However, with the hydraulic moistening of the roof, uniform and adequate saturation of the strata is quite difficult because of the relatively low permeability of the constituent strata. Permeability is reduced with the increasing depth of mining operations.

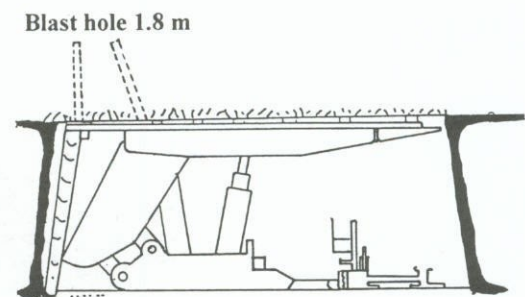


Figure 4 Blasting pattern at the setup entry.

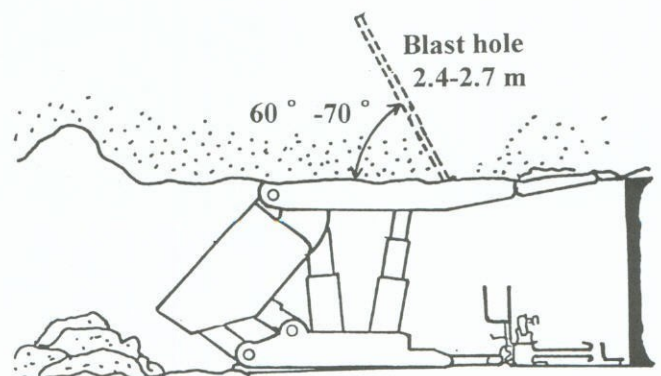


Figure 5 Blasting pattern in longwall mining.

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Hydraulic fracturing occurs in random segments of the hole, mainly through existing natural cracks. Water opens them even more and develops only these cracks, starting to move through those randomly oriented cracks that have with the widest openings.

Using present methods, it is impossible to control the rock loosening process, leading to uneven changes in the mechanical properties of the rocks.

The main structural difference between easy- and hard-to-collapse rocks is the stratification of easily collapsible rocks, with relatively weak contact between layers. The hard-to-collapse rocks, even though sedimentary, are massive and monolithic and have no distinct cleavages or bedding planes. If the hydraulic fracturing technique is used effectively and controllably, the instability of the longwall face which is caused by the dynamic effects of roof rocks can be limited by inducing artificially directed cracks in the hard-to-collapse rock mass and dividing it into many layers.

Hydraulic fracturing system

A new method for loosening hard-to-collapse roof rocks is based on the hydraulic fracturing technique that has been used in petroleum or rock engineering such as the extraction of geothermal energy from hot dry rocks or in-situ stress measurement. In general, two types of fractures are created by hydraulic fracturing, i.e., vertical or longitudinal and horizontal or transverse. **Creating vertical fractures has primarily been studied, however, forming horizontal fractures is useful and can effectively control the caving of hard-to-collapse roofs in longwall panels.** Forced horizontal fractures can be formed through one borehole, creating two, three, or more layers along the bedding planes, depending on the number of artificial cracks which are created on the internal walls of the borehole before fracturing.

Using the theory of elasticity or fracture mechanics one can create a condition for creation of the horizontal fracture. With the elastic theory, the conditions for the propagation of this horizontal fracture can be met if the internal water pressure equals the vertical stress plus the tensile strength of the rock. However, the propagation conditions depend on the existence of numerous natural cracks in the rock mass. Therefore, controllable crack initiation needs the treatment of cracks using

fracture mechanics. Figure 6 shows an artificial circumferential crack model of a borehole in order to create a horizontal fracture plane along the bedding plane in the hard-to-collapse roof (Keer, et al., 1977). A borehole is drilled under the magnitude of initial vertical stress P_z and an artificial circumferential crack is formed on the internal wall of the borehole. Water is pumped into a section of the borehole that is isolated by packers. As the water pressure increases, a fracture begins at the tip of the artificial crack. In order to simplify the analytical treatment, let the angle of artificial crack edge $\theta_c = 0$. Then, the breakdown water pressure P_c is estimated by the following equation:

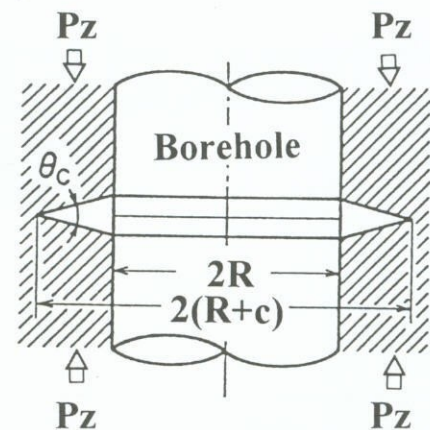


Figure 6 An artificial circumferential crack model of a borehole.

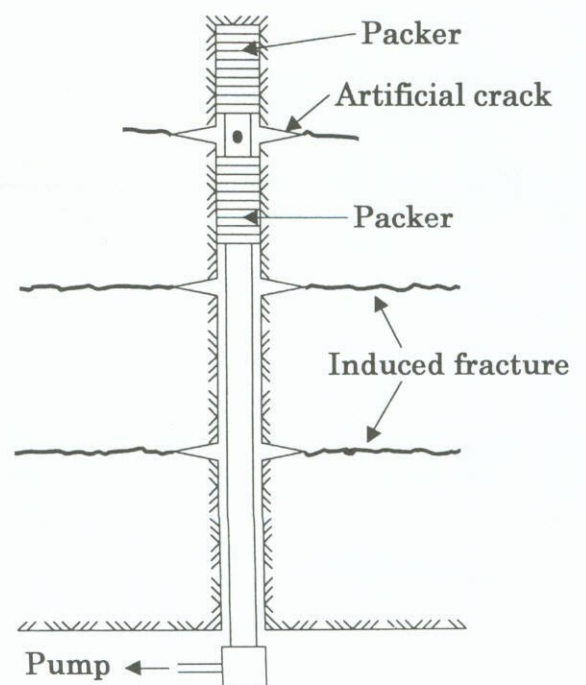


Figure 7 Hydraulic fracturing and moistening system.

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$$P_c = P_z + K_{Rc} / (k_c c^{1/2}) \quad (1)$$

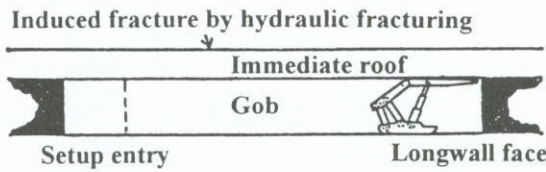
where K_{Rc} = fracture toughness; c = crack length and k_c = a constant depending on the ratio of crack length c to borehole radius R . Once formed, the fracture will continue to propagate as long as the pressure is greater than the stress normal to the plane of the fracture, or the vertical stress.

Figure 7 shows a hydraulic fracturing and moistening system. First, a long borehole is drilled into the roof vertically along the setup entry, the headentry, or tailentry. Next, one, two, or more

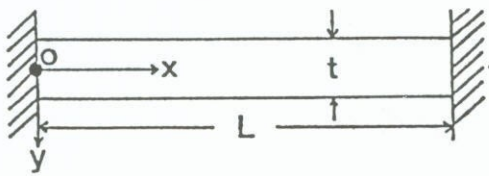
artificial circumferential cracks are made on the internal walls of a borehole at a given depth with slot cutting equipment or a water jet. Then, the packers are installed and, after that, hydraulic fracturing begins. Finally, the hard-to-collapse roof rocks are loosened by both fracturing and moistening.

Acceleration of roof caving in a gob with hydraulic fracturing

When the immediate roof is a hard-to-collapse one, a horizontal or transverse fracture can be created in the roof using a hydraulic fracturing technique, as shown in Figure 8(a). For



(a)



(b)

Figure 8 (a) Horizontal fracture in the immediate roof before first caving and (b) a built-in beam model.

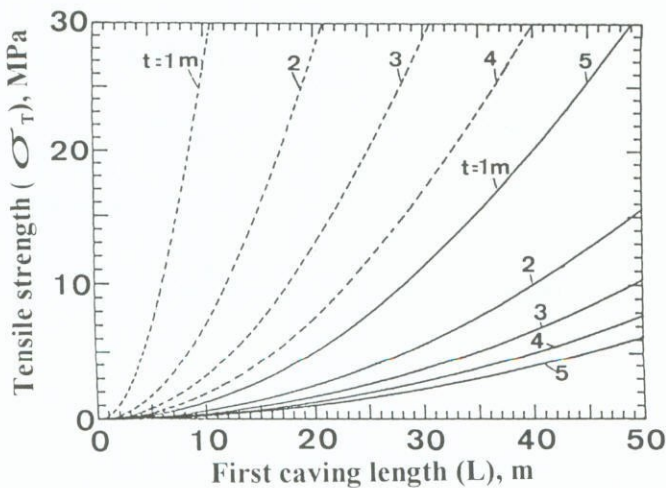
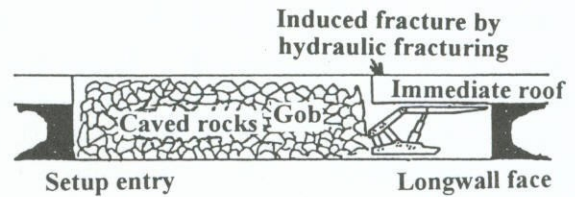
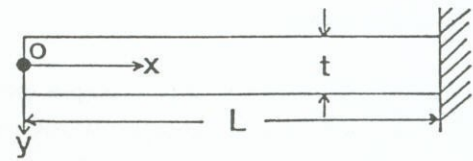


Figure 9 Relationship between first caving length and tensile strength of the immediate roof.



(a)



(b)

Figure 10 (a) Roof cantilever behavior and (b) cantilever model.

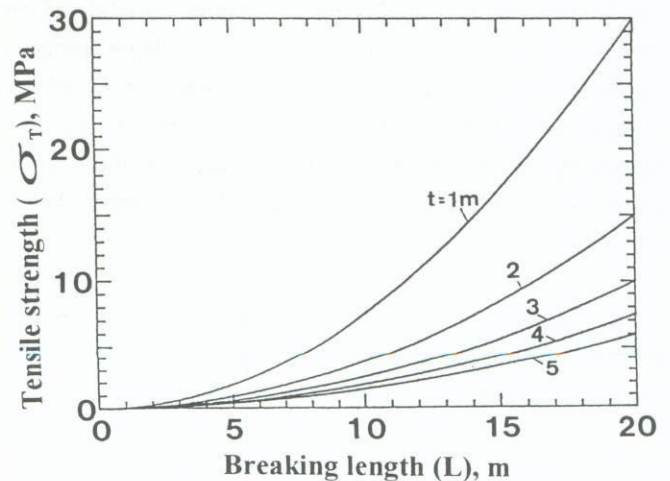


Figure 11 Relationship between breaking length and tensile strength of the immediate roof.

the immediate roof, the first caving length or weighting interval can be estimated using fundamental beam theory, as is modeled in Figure 8(b). The first caving length is determined by the following equation:

$$L = (2t \sigma_T / \gamma)^{1/2} \quad (2)$$

where L = first caving length for the immediate roof; t = thickness of the immediate roof; σ_T = tensile strength of the immediate roof, and γ = unit weight of the immediate roof. Of course, the immediate roof is greatly strengthened by horizontal stress, but to be conservative, the horizontal stress is assumed to be zero.

If the immediate roof is subjected to uniformly distributed pressure, p , due to water pressure in the induced horizontal fracture after hydraulic fracturing, the first caving length is estimated by the following equation:

$$L = [2t^2 \sigma_T / (\gamma t + p)]^{1/2} \quad (3)$$

Figure 9 shows the results from a parametric study using equations (2) and (3). The thinner the immediate roof, the smaller the caving length is. With a water pressure of 0.5 MPa, the caving lengths become much smaller. The reduction of the tensile strength of the immediate roof rock can be expected from moistening effects after hydraulic fracturing. The decrease in tensile strength leads to a shorter caving length.

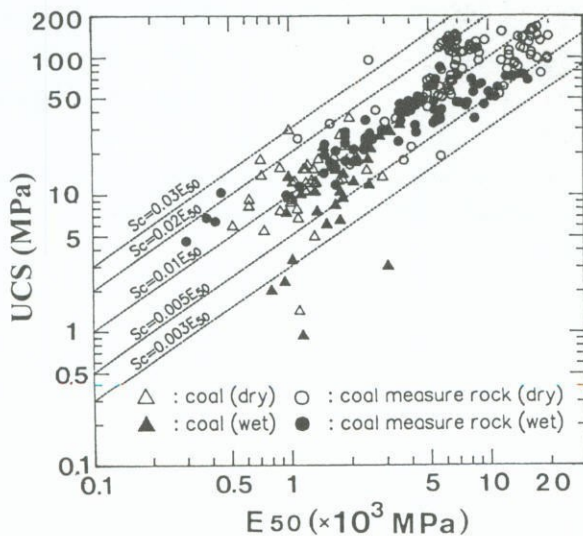


Figure 12 Relationship between tangent Young's modulus and uniaxial compressive strength of the Coal Measures rocks at Miike and Ikeshima Collieries.

After the first caving in the immediate roof, as the longwall face advances, the immediate roof, depending on the type of roof rock, caves in immediately with a short delay behind the powered supports. However, under the hard-to-collapse roof rocks, the immediate roof overhangs, remains for a considerable length of time and behaves like a cantilever. As the face advances, the length of the overhang increases. At some point, the roof cantilever will break and it will fracture due to tension at its fixed end. This behavior can be modeled, as is shown in Figure 10(a). Since the water pressure disappears after the first caving, it will not be considered here. Considering a cantilever model, as is shown in Figure 10(b), the breaking length or interval, L , is estimated by the following equation:

$$L = (t \sigma_T / 3 \gamma)^{1/2} \quad (4)$$

Figure 11 shows certain results using equation (4). Compared to the results in Figure 9, the breaking lengths become much smaller than the first caving lengths. Here, the strength reduction from moistening effects can be expected again.

Mechanical properties of Coal Measures rocks

Coal measures in Japan are Paleogene and consist primarily of sandstones, shales, sandyshales, mudstones, tuffs, conglomerates and coals.

Figure 12 shows the relationship between tangent Young's modulus at 50% of peak stress, E_{50} and uniaxial compressive

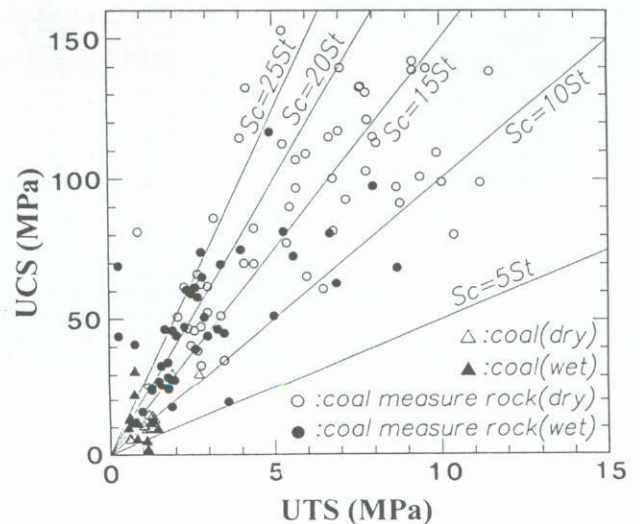


Figure 13 Relationship between uniaxial tensile strength and uniaxial compressive strength of the Coal Measures rocks at Miike and Ikeshima Collieries.

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strength, UCS of the Coal Measures rocks at Miike and Ikeshima Collieries. These data were obtained by uniaxial compression testing under dry and wet (saturated) conditions.

Figure 13 shows the relationship between UCS and uniaxial tensile strength, UTS, which was obtained by Brazilian tensile testing under dry and wet conditions.

These figures show that the strength properties cover a wide range and decrease under wet conditions. Some shale exhibits a slaking phenomenon when it comes in contact with water.

Figure 14 shows the mechanical property changes of Miike sandstone under different contents of water. The properties of the rock decrease with an increase in the water content.

Fracture toughness of strong sandstone

Fracture toughness tests were performed according to the ISRM suggested methods (ISRM, 1988).

The fracture toughness of Miike strong sandstone was determined by using chevron bend and short rod tests under dry and wet conditions. From these two tests, the anisotropy of fracture toughness can be estimated. Level II testing was done in all fracture toughness testing.

The sandstones, which are Miike-1 and -2, were taken from the roofs of the Upper Seam and the Second Upper Seam, respectively, in the West-80 area. These rocks were fine-grained sandstone. The physical and mechanical properties, including fracture toughness, obtained from laboratory tests are listed in Table 1. Other rocks are also listed for reference.

Figures 15 and 16 show the relationships between K_{CB} and K_{SR} and between K_{CB}^c and K_{SR}^c , respectively. K_{CB}^c and K_{SR}^c are the corrected fracture toughness. The value of K_{SR} tends to become greater than K_{CB} . The corrected fracture toughness is always greater than the initial value. Miike sandstone-1 shows some anisotropy but sandstone-2 does not. When saturated, the rocks decreased in fracture toughness, especially Miike sandstone. Using equation (1) and these data, an effective hydraulic fracturing system can be designed.

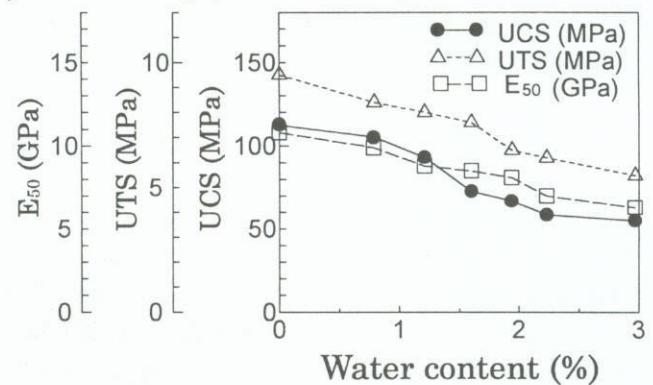


Figure 14 Mechanical properties under different conditions.

Table 1 Physical and mechanical properties of rocks.

Sample	Effective porosity (%)	Water content (%)	UCS (MPa)	UTS (MPa)	E ₅₀ (GPa)	K _{CB} (MPa·m ^{0.5})	K _{CB} ^c (MPa·m ^{0.5})	K _{SR} (MPa·m ^{0.5})	K _{SR} ^c (MPa·m ^{0.5})
Sandstone (Miike-1)	9.30	0.0 (dry)	118.3	8.2	8.2	1.56	2.77	1.43	1.79
		3.75 (saturated)	37.8	1.1	5.1	0.63	0.82	0.23	0.51
Sandstone (Miike-2)	9.67	0.0 (dry)	126.1	8.3	6.5	1.27	1.15	1.42	2.20
		3.92 (saturated)	47.2	2.1	4.1	0.35	0.62	0.33	0.56
Granite (Tokuyama)	1.65	0.0 (dry)	218.9	10.9	23.7	2.08	2.35	1.75	2.50
		0.62 (saturated)	194.1	9.1	22.0	1.95	2.18	1.79	2.40
Andesite (Karatsu)	12.05	0.0 (dry)	116.9	8.7	16.3	1.49	2.58	1.78	2.63
		4.59 (saturated)	79.1	4.9	11.1	1.10	2.07	1.30	2.13

Note : Sandstone Miike 1 and 2 were taken from the roof of the Upper Seam and the 2nd Upper Seam, respectively.

UCS : Uniaxial compressive strength.

UTS : Uniaxial tensile strength obtained by Brazilian tensile test.

E₅₀ : Tangent Young's modulus at 50% peak stress.

K_{CB} : Fracture toughness from chevron bend test.

K_{CB}^c : Corrected fracture toughness of K_{CB}.

K_{SR} : Fracture toughness from short rod test.

K_{SR}^c : Corrected fracture toughness of K_{SR}.

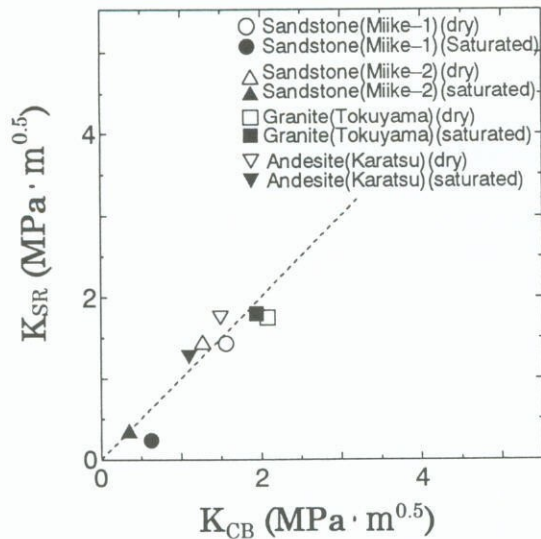


Figure 15 Relationship between fracture toughness K_{CB} and K_{SR} .

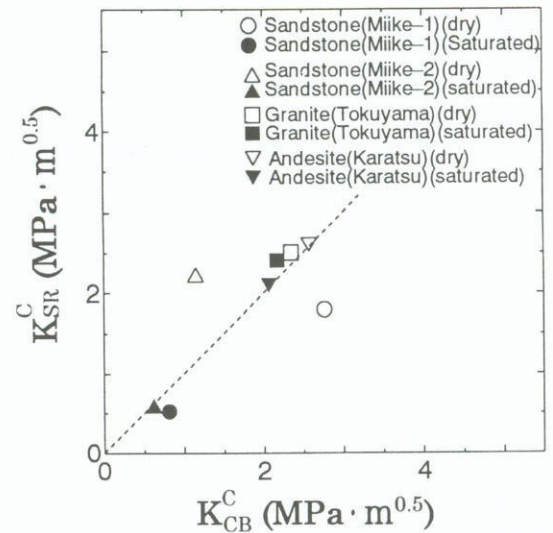


Figure 16 Relationship between corrected fracture toughness K_{CB}^C and K_{SR}^C .

CONCLUSION

Periodic weighting and windblast can be controlled by the acceleration of roof caving with hydraulic fracturing and the moistening technique. In the near future, the hydraulic fracturing system proposed in this study will have practical applications.

ACKNOWLEDGMENT

The authors would like to thank the staff of Miike Colliery and Kyushu Mine Safety and Inspection Bureau (KMSIB) for their cooperation in providing relevant information in the preparation of this paper.

Any opinions stated in this paper are those of the authors themselves and are not necessarily those of the colliery or the KMSIB.

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A Study of Periodic Weighting of Longwall Supports

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ABSTRACT

Periodic weighting of longwall supports occurs during retreat of the face under certain geologic conditions. Strong strata in the immediate and main roof tend to cantilever over the gob, weighting the supports periodically. The distance between weighting peaks, as well as the intensity of periodic weighting, is determined by the strength and thickness of roof members, their location relative to the seam, the frequency of jointing, and characteristics of the gob. In this paper the effects of varying several of these variables are investigated using block modeling. The models are calibrated against a published case history of periodic weighting in the Pittsburgh Seam. It is concluded that block modeling is a useful tool for correlating measured support pressure histories with expected modes of strata behavior in the roof, with potential for predicting shield behavior in new geologic environments.

INTRODUCTION

Background

Periodic weighting of longwall supports is a phenomenon occurring when the face area is exposed to recurring cycles of overhang and breakage of strong strata in the immediate and main roof. The most intense weighting occurs just prior to each caving of the strong stratum, when the length cantilevered into the gob is maximized.

Periodic weighting has doubtless been observed since the inception of longwall mining and occurs to some degree in many active mines. In early United States longwall mines in the Appalachian coalfield, Carman (1) describes "roof pressure cycles" caused by the "swinging and shearing off of the roof" behind the longwall face, peaking at 30- to 100-ft intervals of face advance. He correlated the weighting interval to strata characteristics and overburden thickness. Wilson (2) based his classic analysis of support loading on a statics approach, but

observed: "Faces subjected to periodic weightings may require an uprating of the supports."

Powered support leg pressures are readily monitored, and studies have been reported in the literature for 30 years or more. Gorrie and Scott (3) describe an early study performed in the United Kingdom that utilized Bristol recorders to monitor pressure in the hydraulic circuits of powered supports. In discussing presumed strata behavior causing difficult loading conditions for longwall supports, they note:

In the general absence of facilities to allow physical observations of the behaviour of the overlying beds in the waste behind a longwall face, these considerations were still only an expression of opinion largely based on the circumstantial evidence of results observed at the coalface.

In the last 20 years serious attention has been directed to the monitoring and study of longwall support leg pressures, enabling a more complete understanding of periodic weighting. Peng et al. (4) and Peng and Chiang (5, 6) began to systematically monitor and analyze periodic weighting cycles in a number of longwall mines in the eastern United States, defining the important parameters affecting periodic weighting and relating them to different types of stratigraphic sequences. It was recognized that the periodic weighting interval is related to the spacing of breaks in the overlying strata. A longer periodic weighting interval leads to a larger maximum periodic weighting resistance. Also, studies with four-leg shields showed that periodic weighting leads to a canopy load distribution biased towards the gob.

More recently, advances in longwall technology have made possible continuous, full-face, full-panel monitoring of shield leg pressures (7, 8, 9). Peng (10) describes how new longwall shields are currently delivered with pressure monitors and real-time display of shield pressure, enabling a complete record of periodic weighting events amenable to statistical evaluation. Peng has discovered that the mean and standard deviation of the



time-weighted average resistance (TWAR) are related in a predictable manner to the intensity of periodic weighting.

Objective

Shield leg pressure is much more readily monitored than strata movements over a retreating longwall face, and it would be desirable to garner as much information as possible from the excellent pressure data now available. Very good conceptual models of strata behavior have been developed to explain shield leg pressure records based on knowledge of the overlying lithology, basic beam theory, limited instrumentation results, and intuition. This paper explores a slightly different approach by using block modeling as a tool to help illustrate the effect of different strata behavior on support weighting.

PERIODIC WEIGHTING CASE HISTORY

An instructive case history of periodic weighting in the Pittsburgh Seam is provided by Peng et al. (4). This case history was selected for block modeling because of the complex strata behavior in the main roof.

A representation of the immediate and main roof behavior is presented in Figure 1(a). The immediate roof is a weak shale that caves immediately. The active portion of the overlying main roof is divided into two units: 1) an argillaceous shale approximately 18-ft thick (lower main roof), and 2) an upper limestone approximately 25-ft thick (upper main roof). Figure 1(b) shows a typical graph of changes in TWAR as a function of face advance, developed from measured support leg pressure data. From graphs of this type and observations at the face, it was inferred that the caving interval of the argillaceous shale was 8 to 10 ft and of the limestone was 55 to 85 ft.

MODELING METHODOLOGY

Block Modeling

The mechanical behavior of the strata over the longwall face is complex, involving block rotational and translational effects, interaction between blocks, and the action of the *in situ* stress field (11). If the spacing of incipient fractures delineating blocks can be assumed, block modeling enables a rigorous treatment of the mechanical behavior of interacting blocks of strata translating and rotating from their confined state over the solid panel to their final resting place in the gob.

Block models prepared using the distinct element method differ from continuum-based boundary element and finite element models traditionally employed in rock mechanics. They are uniquely suited to an analysis of periodic weighting mechanisms of longwall supports because (1) *blocky behavior* predominates, and (2) *very large displacements* occur during the caving process. Other methods lack the capabilities to consider these mechanisms to the requisite degree.

The Universal Distinct Element Code (UDEC) (12) is an off-the-shelf program based on the distinct element method. It was selected for this study over other distinct element codes for reasons of convenience.

Model Geometry

The geometry of the roof strata is readily implemented in the UDEC model. Figure 2 illustrates the general model configuration and finite difference mesh near the face area. Because the model is two-dimensional, block geometry is defined only in the plane normal to the face. The position of neighboring shields and their support contribution is not considered in the two-dimensional regime. For simplicity, the model is made entirely elastic, thus omitting the treatment of more complex rock failure mechanisms.

Fictitious joints define the roof cantilever that is allowed to cave as the roof is undermined by the advancing face. The timing of caving is prescribed as a function of face position and corresponds to the inferred geometry described in Figure 1(a). Part of the model caving sequence is shown in Figure 3 (a-f). Once caved, blocks are deleted and do not provide support to the intact cantilevered roof as gob. In actuality, bulking of the gob is expected to partially support the intact roof. Therefore, maximum convergence is predicted by the model. Strata above the cantilevered roof is assumed to be self-supporting in an arch spanning from ahead of the face into the gob.

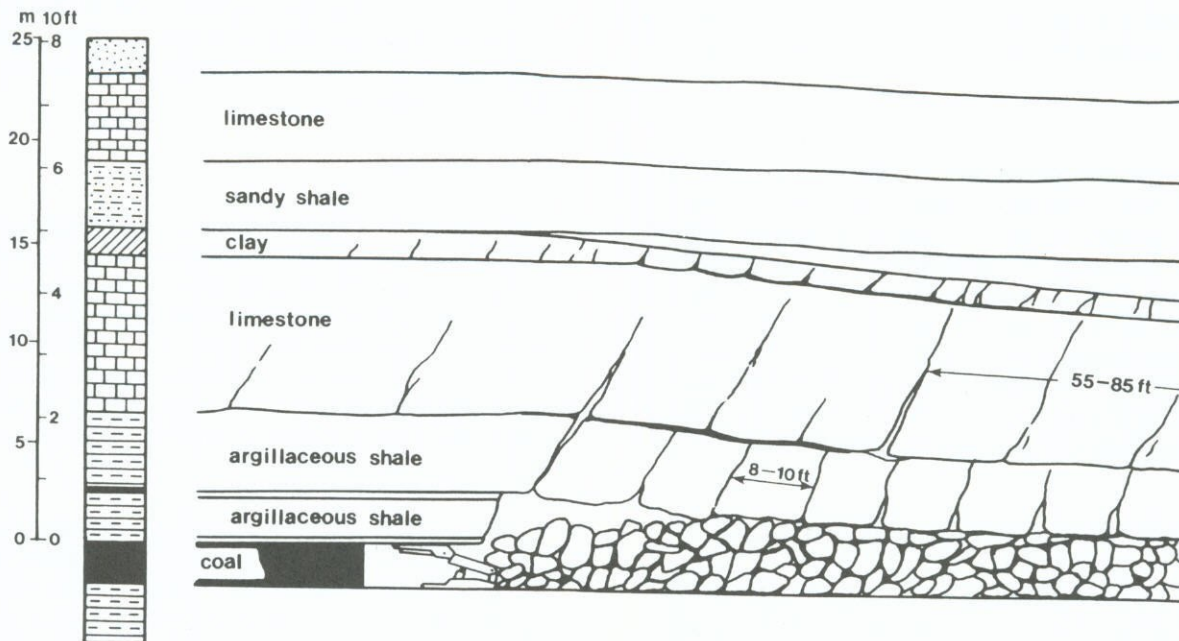
RESULTS

Convergence above the shield is monitored in the model as the face is advanced in 2.5-ft increments, representing the depth of the web cut. Equilibrium is established at each face position. Modeled convergence is converted to shield resistance by applying an estimated support stiffness. From these data a history of shield resistance as a function of face position is generated as shown in Figure 4, which most closely corresponds to the TWAR data measured in Figure 1(b).

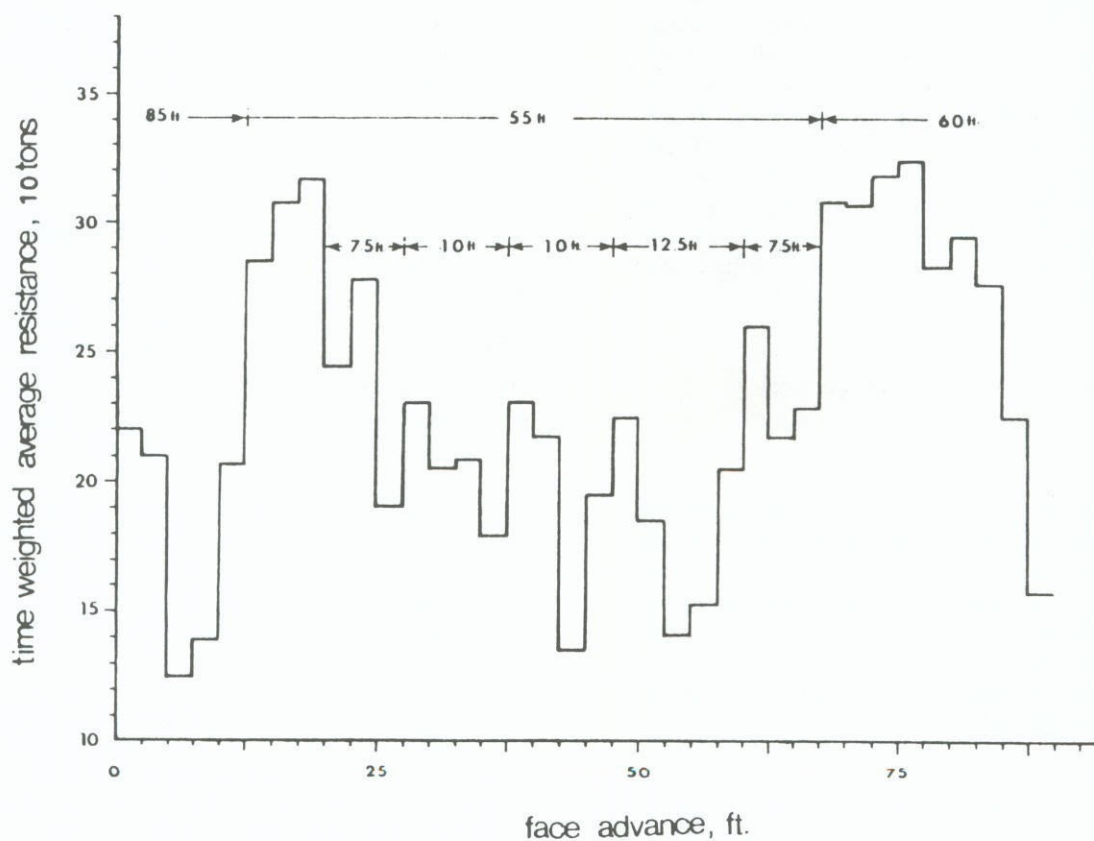
Observations by Peng et al. (4) suggest that periodic weighting is characterized by a significant increase in TWAR. The model results in Figure 4 reflect this sort of pronounced increase in shield resistance prior to reaching a peak approximately every 24 mining cycles, corresponding to the periodic caving of the thick limestone in the upper main roof.

A steady increase in shield resistance is complicated by the superposed loading cycles caused by caving in the immediate and lower main roof. It was observed that for every large periodic weighting, 2 to 4 smaller periodic weightings occurred. For the Pittsburgh Seam stratigraphy in Figure 1(a), this coincides with the more frequent caving of the argillaceous shale followed by longer-cycled caving of the overlying limestone. While the periodic weighting cycle arising from the caving of the argillaceous shale averages about 8 to 10 ft, the limestone weighting cycle averages almost 6 times as long, or about 60 ft.

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a) Geology and Structure of Immediate and Main Roof in the Pittsburgh Seam



b) Time-Weighted Average Support Resistance vs. Face Advance

Figure 1. Case History of Periodic Weighting in the Pittsburgh Seam (after Peng et al. 4). (Reproduced with permission from the author and the Society of Mining, Metallurgy, and Exploration (SME), Littleton, Colorado.)

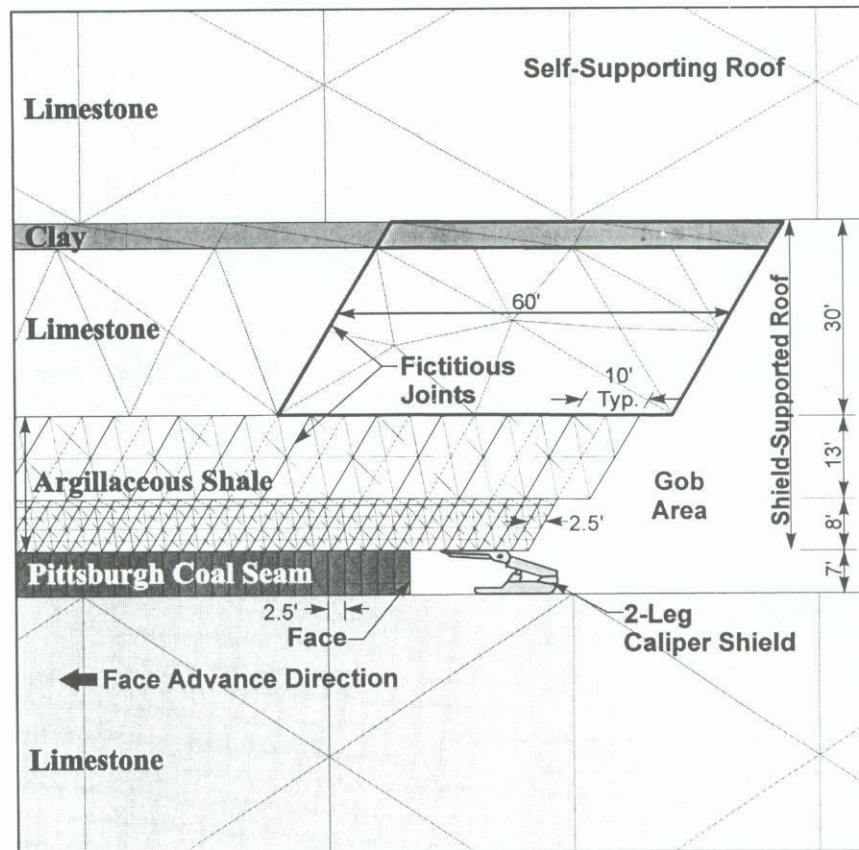


Figure 2. UDEC Model Configuration

The superposition of these cycles is subtly evident in the model and can be distinguished by smaller fluctuations of shield resistance prior to the arrival of the maximum periodic resistance in Figure 4.

The hypothetical effect of doubling the caving frequency of the limestone in the main roof is illustrated by the modeled results shown in Figure 5(a). The distance between peak loadings is reduced from approximately 60 to 30 ft. As a result, the intensity of peak resistance is reduced by approximately 16% or 52 T. Conversely, by halving the frequency of limestone caving, roof convergence eventually exceeds the capacity of the shields and forces them to "go on yield" until the limestone can cave, as illustrated in Figure 5(b).

CONCLUSIONS

Block modeling is an effective method of investigating different modes of strata behavior and their effects on support loading. While the use of block modeling does not solve the

fundamental problem of lack of physical observation, it provides an alternate tool permitting simulation of the caving of various strata sequences. This capability could be used as a method of illustrating strata mechanics in a mechanically satisfying manner, as an aid in interpreting shield pressure data, or most importantly as a predictive tool for new geologic environments.

Further improvements to the simplified models presented here can be readily implemented. These include 1) more complete treatment of the support action by modeling simulated shields using structural elements, and 2) introduction of a mechanically realistic gob which was omitted here for simplicity. The former would enable use of block models to directly simulate the behavior of shields subjected to periodic weighting in different geologic environments, and may ultimately prove useful as a shield design tool. Three-dimensional distinct element codes have been available for some time and could be used for simulating the behaviors of groups of shields. Ultimately, with advances in computing power, it may be possible to prepare models to evaluate and predict full-face and full-panel performance.

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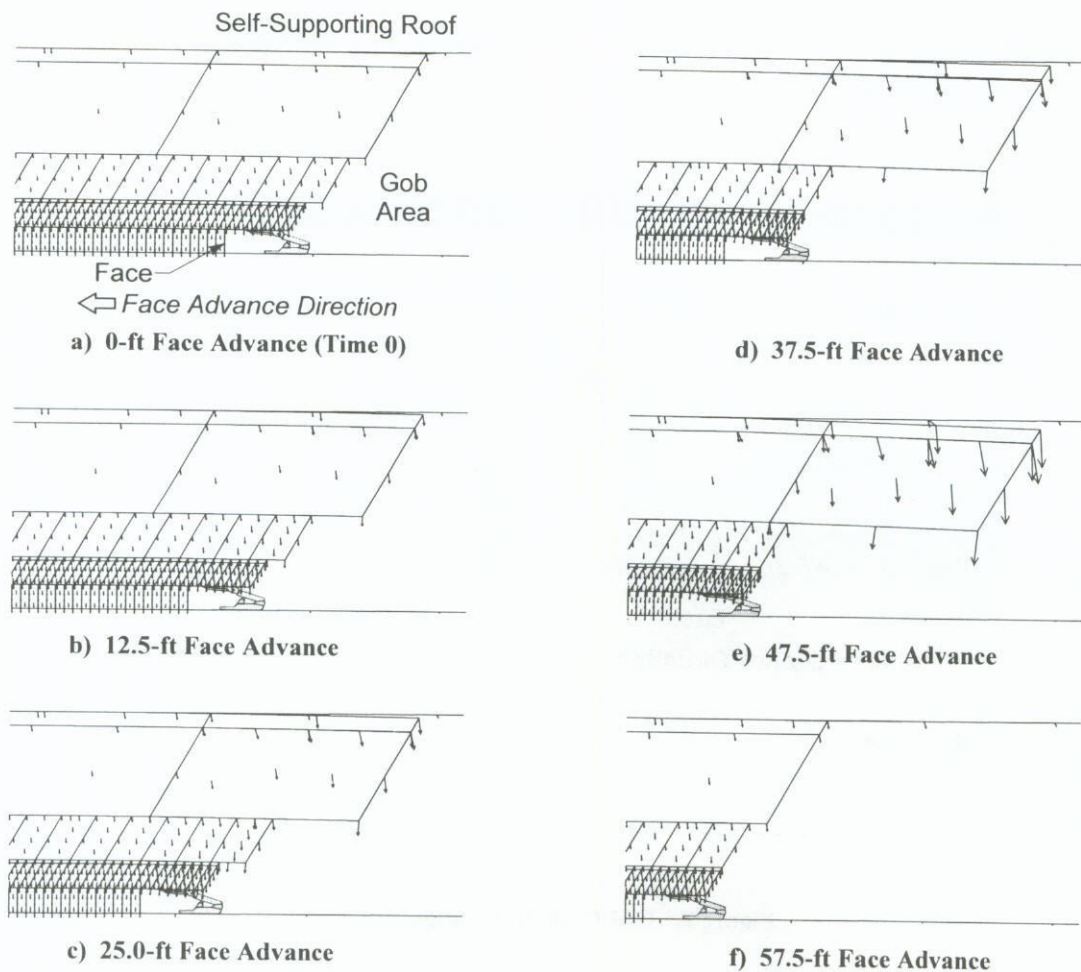


Figure 3. Sequence Showing Different Stages of Roof Caving with Face Advance

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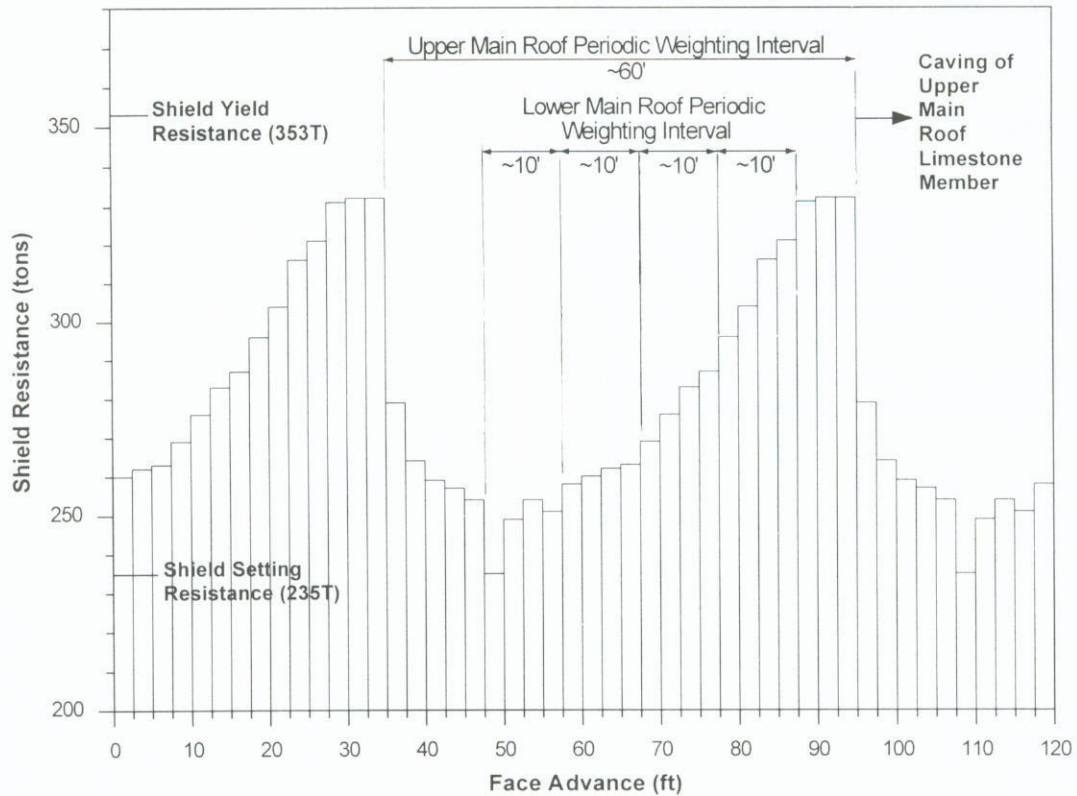
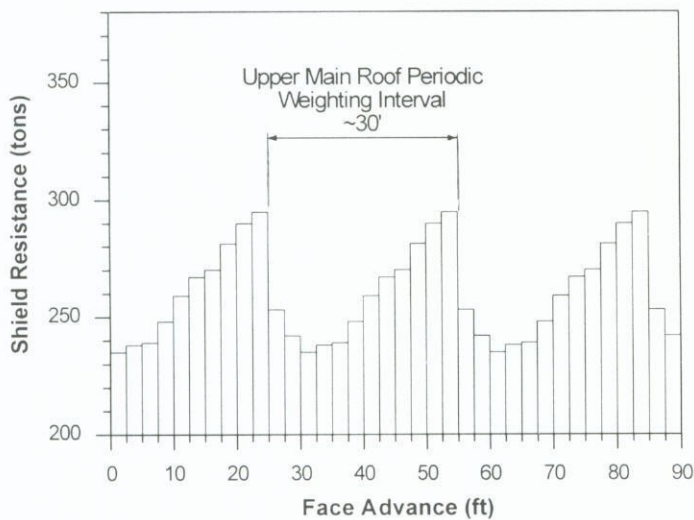
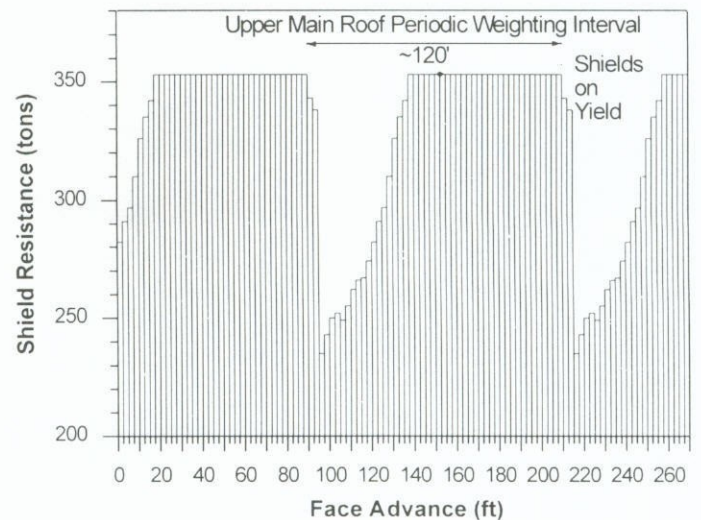


Figure 4. Variation of Modeled Shield Resistance with Face Advance



a) Caving Frequency = $\frac{1}{2} \times \text{Actual} = \sim 30$ ft



b) Caving Frequency = $2 \times \text{Actual} = \sim 120$ ft

Figure 5. Variation of Modeled Shield Resistance with Face Advance, Variable Caving Frequency in the Upper Main Roof Limestone Stratum

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Effects of Panel Mining Sequence and Retreat Direction on the Roof Stability of Longwall Headgate Entries

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ABSTRACT

The major functions of longwall headgate entries are to provide access ways to the outby submains or mains for coal transportation and to the longwall face for fresh air and for transporting men and material during longwall operations. Any roof control problems such as cutter roofs or roof falls in the outby headgate entries could result in significant production delays to the longwall face and may be a threat to miners' safety. Under a thinly laminated immediate roof and a high horizontal stress field, even a longwall headgate entry that has an angle of less than 45° to the maximum horizontal stress may experience roof deterioration in the outby headgate if the longwall panel mining sequence and retreat direction are not properly considered.

This paper attempts to analyze the causes of the roof control problems experienced in the longwall gate entries at our Cumberland and Emerald mines in Pennsylvania and to determine the optimal longwall panel mining sequence and retreat direction for the future mining area at Cumberland.

INTRODUCTION

Both Cyprus Cumberland and Emerald mines are located in Greene County, Pennsylvania and have been employing longwall mining in Pittsburgh seam since 1981 and 1983, respectively. The thinly laminated coal/shale immediate roof and high horizontal stress field are the two common features of the Pittsburgh seam that often create roof control problems during longwall mining.

Directional roof failure in the form of cutter roofs in the outby longwall gate entries has been a major challenge to both longwall operations. Since great efforts have been done in the past several years to predict (i.e. hazard mapping and regularly examining outby roof conditions) and control (i.e. installing cable trusses, cable bolts, or other supplemental roof support) roof failure and falls in the outby longwall gate entries, production delays to the longwall face have been tremendously

reduced or nearly eliminated during longwall retreat. However, installation of supplemental roof supports in the outby entries, especially headgate entries, during longwall retreat can result in significant production delays to the longwall face. To improve the overall roof stability and reduce or eliminate the supplemental roof support in the outby headgate entries, the causes and mechanism of entry roof failure and falls must be analyzed and completely understood.

For the headgate roof stability, both the development-induced and longwall mining-induced stress redistributions in the outby headgate entries in response to the coal extraction process and formation of caved zones are of major concerns. Due to the nature of longwall operations, the headgate entry cannot be supported with external standing supports in advance or during longwall retreat. Therefore, if the maximum horizontal stress plays a major role in the roof deterioration of the outby headgate entries during longwall retreat, a proper mine design, including longwall panel mining sequence and retreat direction, will be the most effective way of releasing the maximum horizontal stress concentration. If the vertical stress plays a major role, reducing the actual entry width (width of development + 2 times the rib yielding depth) may be more effective than changing the longwall panel mining sequence and retreat direction.

Review and analysis of the roof control problems experienced at both Cumberland and Emerald mines indicate that a directional roof failure in the form of cutter roofs in the outby headgate entries has been mainly caused by shear failure due to the maximum horizontal stress concentration during longwall retreat. Therefore, both the longwall panel mining sequence and retreat direction should be taken into account where the maximum horizontal stress plays a major role in the roof deterioration of the outby headgate entry during longwall retreat.

ROOF LITHOLOGY AND IN-SITU STRESS FIELD

Both a thinly laminated immediate roof and a high differential stress field are very common at Cyprus Cumberland and Emerald mines. As illustrated in Fig. 1, the immediate roofs at both mines

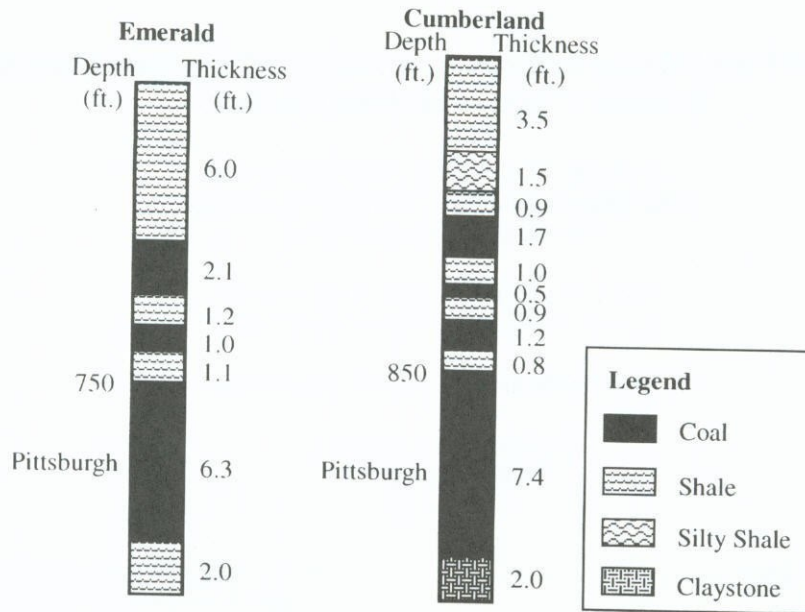


Fig. 1 Roof Lithology at Cyprus Cumberland and Emerald Mines

consist of thinly laminated shale and coal layers, and can easily fail due to either shearing or tension if they are not bonded together by bolts to form a thick beam. The maximum horizontal stress measured in Pittsburgh seam mines ranges from 3,000 to 5,200 psi, which is about 1.5 times the minimum horizontal stress and 5 times the vertical stress (1,2). The orientation of the maximum horizontal stress ranges from N65°E to N90°E or E-W in Southern Pennsylvania and Northern West Virginia. Underground observations at both Cumberland and Emerald mines also indicate a similar orientation for the maximum horizontal stress.

ROOF CONTROL PROBLEMS IN LONGWALL GATE ENTRIES

At Cumberland mine, the longwall panel mining sequence is from South to North and longwall retreat directions are from East to West in the eastern area and West to East in the western area. Since all the three entries in the longwall gate entry system are 17° to the regional maximum horizontal stress, they have been experiencing good roof condition during CM development. However, the crosscuts, especially those in the western area, developed at 60° to the gate entries have been experiencing some roof control difficulties on CM development.

The headgate entries immediately outby the longwall faces in the eastern area experienced difficulties in roof control during longwall retreat (Fig. 2). In addition to the primary roof support, cable trusses had to be installed from time to time as the secondary roof support to prevent roof failure and falls in the outby belt entry (headgate). Production delays to the longwall face due to dealing with the deteriorated roof were significant.

In the western area, the longwall panel mining sequence and retreat direction have been resulting in very good roof conditions in the outby headgate entry due to the longwall gob effect during longwall retreat. So far, two panels have been successfully mined without any significant roof control problems in the headgate entries.

At Emerald mine, the longwall panel mining sequence is from North to South and the panel retreat is from East to West, as illustrated in Fig. 3. Generally, the headgate entries have been in good roof conditions due to the relaxation of the maximum horizontal stress over the outby area. Two rows of wood cribs in an 8-ft spacing have been used as the supplemental roof support for the tailgate entries without any significant roof control problems. However, as the G panel's face approached the F panel's gob (Fig. 3), horizontal stress concentration or funneling was so significant and the face had to be shut down many times to truss the outby tailgate roof.

Both mines have been utilizing a 3-gate entry system with the crosscut spacing of 135 to 145 ft. For the development of each section or block, the linear footage of entries developed in the better orientation (17° or 26° to the maximum horizontal stress) is about 2.1 to 2.3 times the linear footage of crosscuts developed in the worse orientation (43° or 86° to the maximum horizontal stress). In addition, installation of supplemental roof supports in those crosscuts during longwall retreat could have no or less impact on longwall production.

Table 1 lists the frequency of roof control problems experienced at both mines during CM development and longwall retreat. In this study, three categories: rare, occasional, and often have been introduced to describe the frequency of roof

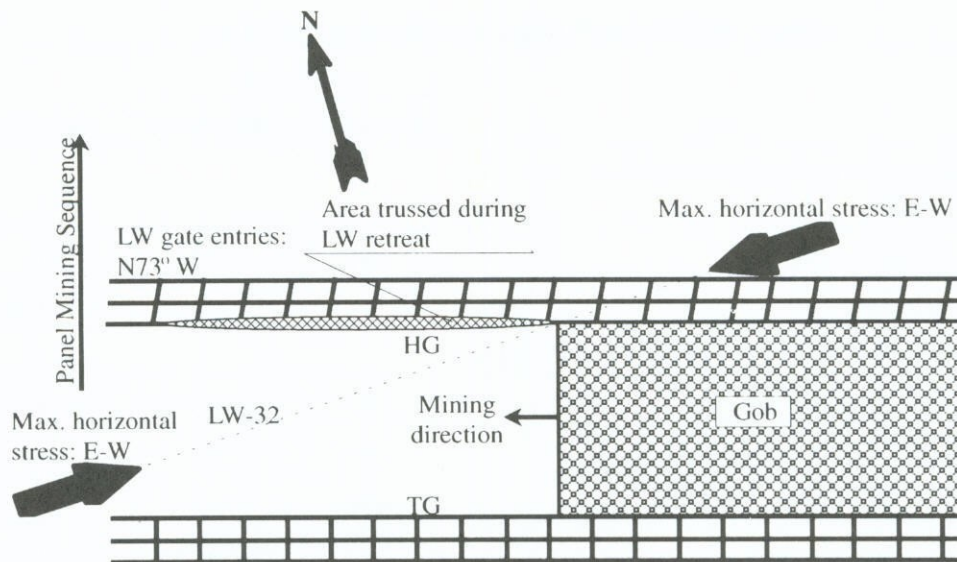


Fig. 2 Roof Control Problems Experienced in the HG of LW-32 at Cumberland Mine

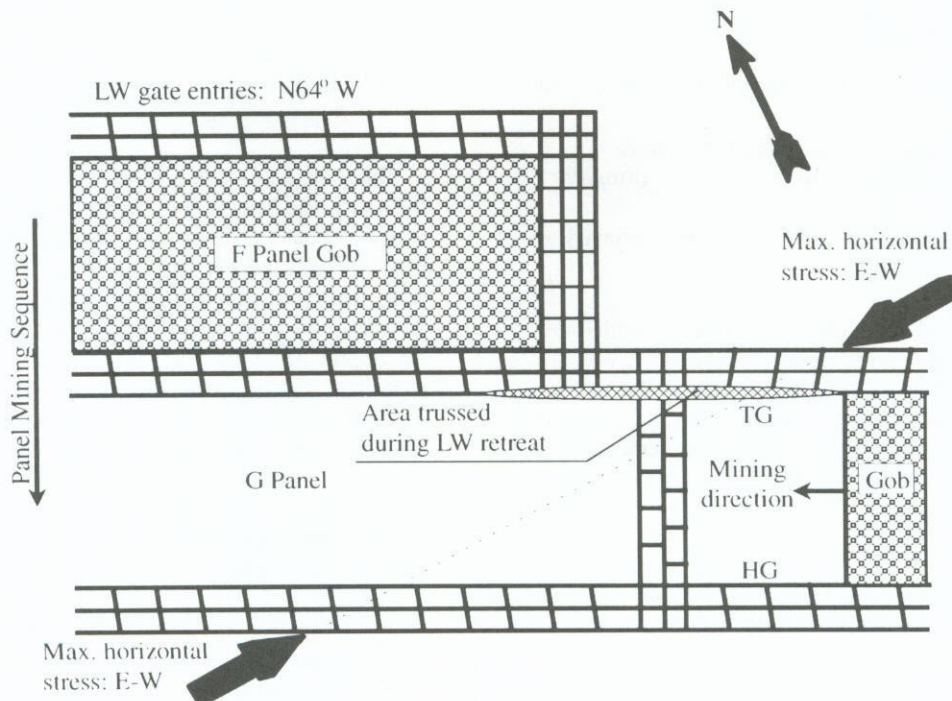


Fig. 3 Roof Control Problems Experienced in the TG of G Panel at Emerald Mine

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Table 1 Roof Control Problems Experienced at Both Mines

Item	Cumberland		Emerald
	East Panels	West Panels	
Frequency of roof control problems on CM development			
<i>LW gate entries</i>	rare	rare	rare
<i>Crosscuts</i>	rare	occasional	occasional
Frequency of roof control problems on LW retreat			
<i>First panel tailgate entry</i>	rare	occasional	occasional
<i>Second panel tailgate entry</i>	rare	rare	rare
<i>Headgate entries</i>	often	rare	rare
<i>Headgate crosscuts</i>	often	occasional	occasional

control problems and their impact on CM development footage and longwall production. Generally, the budgeted footage and production can be exceeded or achieved with the first two categories of roof control problems. However, if the roof control problems belong to the "often" category, significant delays to CM development and longwall production with a higher roof control cost could be expected. It should be noted that the roof control problems experienced at both mines were based upon the following roof control plans:

- Headgate entries at both mines: only primary roof support
- Headgate crosscuts: only primary roof support
- First tailgate entries at both mines: primary + supplemental roof supports
- Second tailgate entries at both mines: primary + supplemental roof supports

The roof control plans during longwall retreat at both mines are: whenever a roof deterioration in the outby longwall gate entries is noticed, supplemental roof supports in the form of cable trusses or cable bolts will be installed in the belt entry of the headgate, while other standing supports, sometimes cable bolts or cable trusses, will be installed for the outby tailgate entries and crosscuts.

ROOF FAILURE MECHANISM AND HEADGATE ROOF CONTROL

Roof Failure Mechanism

For longwall gate entries, two roof failure modes, tension and shearing, are often observed (6), depending upon stress fields (in-situ and mining-induced), roof lithology, and actual roof span. Tension failure in the form of excessive roof sagging can be caused by a great roof span due to either the original development or pillar yielding under mining-induced abutment

loads. Shear failure in the form of cutter roofs can be induced by a high differential stress field due to either entry development, mining-induced horizontal stress concentration, or topographic relief such as valley zones.

To prevent entry roof's tension failure, the actual roof span (developed entry width + 2 times rib yielding depth) during mining process must be limited to a certain value by:

- Designing a narrower entry
- Building a thicker roof beam
- Supporting or stabilizing entry ribs

In doing so, the tensile stresses at the entry roof center and the entry roof beam ends above the pillar abutments due to significant bending and excessive roof sagging can be minimized. Since the actual entry roof span depends not only the developed entry width but also the yielding depths of panel and pillar ribs during mining process, both the tributary load due to development and the abutment loads due to longwall mining play a critical role in entry roof's stability. Therefore, from controlling a tension failure point of view, vertical stress and the supported beam thickness are of major concerns.

To prevent entry roof's shearing failure, lateral movement along the bedding planes in the thinly laminated immediate roof must be minimized by:

- Increasing the friction force between bedding planes using pre-tensioned roof bolts
- Minimizing or eliminating the horizontal stress concentration in the outby headgate entry roof using a proper mine design

Results from both Finite Element modeling and field measurements (1-5) indicate that the most unfavorable roof conditions could be expected when longwall gate entries are developed in a great angle to the maximum horizontal stress. Underground observations at both Cumberland and Emerald



mines show that the best headgate stability can be achieved if the longwall panels are orientated with a small angle to the maximum horizontal stress and are retreated in such a way that the maximum horizontal stress relaxes over the outby headgate entry due to the newly formed gob. Therefore, from controlling a shearing failure point of view, the maximum horizontal stress is critical.

Headgate Roof Control

Generally, longwall headgate entries experience only the development and front abutment loads during their service lives. Therefore, primary roof support alone should be adequate to control the roof if a high horizontal stress field is not present. However, under a high horizontal stress field, if the newly formed gob cannot release the maximum horizontal stress in the immediately outby area of the headgate, roof deterioration may occur. Therefore, supplemental roof supports in the form of cable trusses or cable bolts have to be installed to prevent roof failure and thus roof falls during longwall retreat.

MINE DESIGN VS. THE MAXIMUM HORIZONTAL STRESS

Historical data of roof control problems at our Emerald and Cumberland mines indicate that the orientations of longwall panels and the maximum horizontal stress play an important role in the entry roof stability before longwall retreat, while the longwall panel mining sequence and retreat direction determine the overall entry roof stability during longwall retreat. In other words, the angle between the entry and the maximum horizontal stress determines the probability of entry roof failure on development. The horizontal stress concentration or relief in the immediately outby headgate roof due to the newly formed gob determines the probability of entry roof failure on longwall retreat. However, since the mining-induced stresses are usually 2 to 3 times the in-situ ones, the latter plays an even more critical role in the overall entry roof stability than the former. For example, if the newly formed gob can relax the maximum horizontal stress in the outby longwall headgate entries or crosscuts during longwall retreat, the degree or frequency of roof control problems will be less.

Even though the mains at both mines have been developed in the worst direction with respect to the in-situ stress field, the frequency of cutter roofs were not significant on CM development for majority of the mining areas.

Since the available space of the longwall headgate entry or belt entry is limited during longwall operations, installation of any supplemental roof supports to control the outby deteriorated roof will result in significant production delays to the longwall face. Therefore, the longwall panel mining sequence and retreat direction indicated in Fig. 4 that inherently increase production delays and roof control cost should be carefully evaluated against early production requirement for any new mining areas or mines where the maximum horizontal stress is significant.

The best headgate roof condition during longwall retreat can be achieved if the longwall panel mining sequence and retreat direction indicated in Fig. 5 are employed. Generally, due to the nature of longwall mining, longwall gate entries have the longest linear footage per panel and are the most difficult ones to control on longwall retreat among mains, longwall gate entries, and crosscuts. Therefore, longwall gate entries should be always designed with a small angle to the maximum horizontal stress to minimize the probability of roof failure on development. Most critically, both the longwall panel mining sequence and retreat direction should be designed such that the newly formed gob relaxes the maximum horizontal stress concentration in the outby headgate entry roof during longwall retreat.

SUMMARY AND CONCLUSIONS

Based upon this study, it can be concluded that the roof control problems in the immediately outby headgate entries at Cumberland's eastern mining area were mainly caused by the maximum horizontal stress concentration due to the newly formed gob. Since the longwall mining-induced stresses in the outby headgate entry roof are usually 2 to 3 times the in-situ ones, employing a proper mine design to release such a stress concentration in the outby area is the most effective means of maintaining a stable headgate entry during longwall retreat. Therefore, if the longwall panel mining sequence and retreat direction illustrated in Fig. 5 is used for the new mining area at Cumberland mine, good roof conditions in the headgate entry can be expected during longwall retreat.

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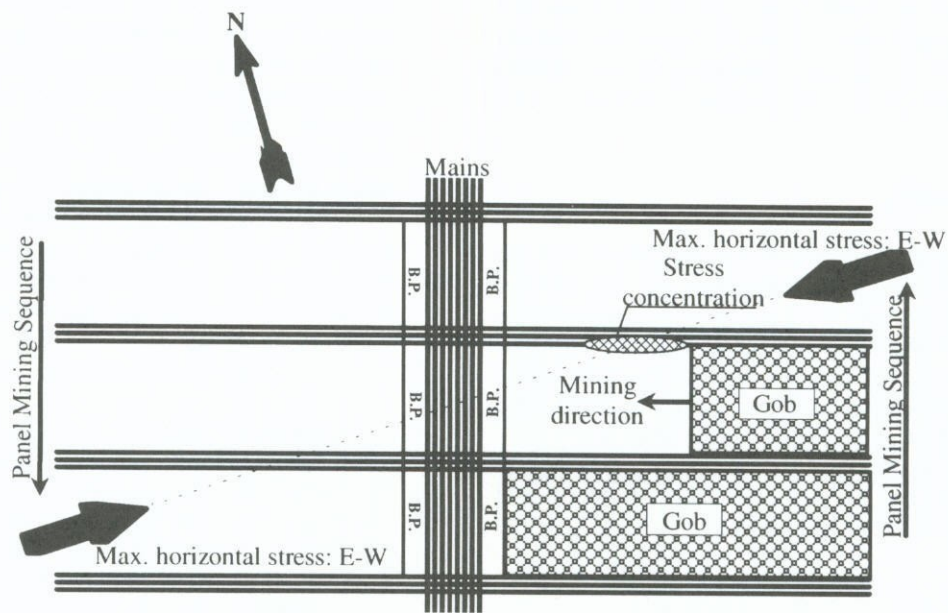


Fig. 4 Horizontal Stress Concentration over the Outby HG During Longwall Retreat

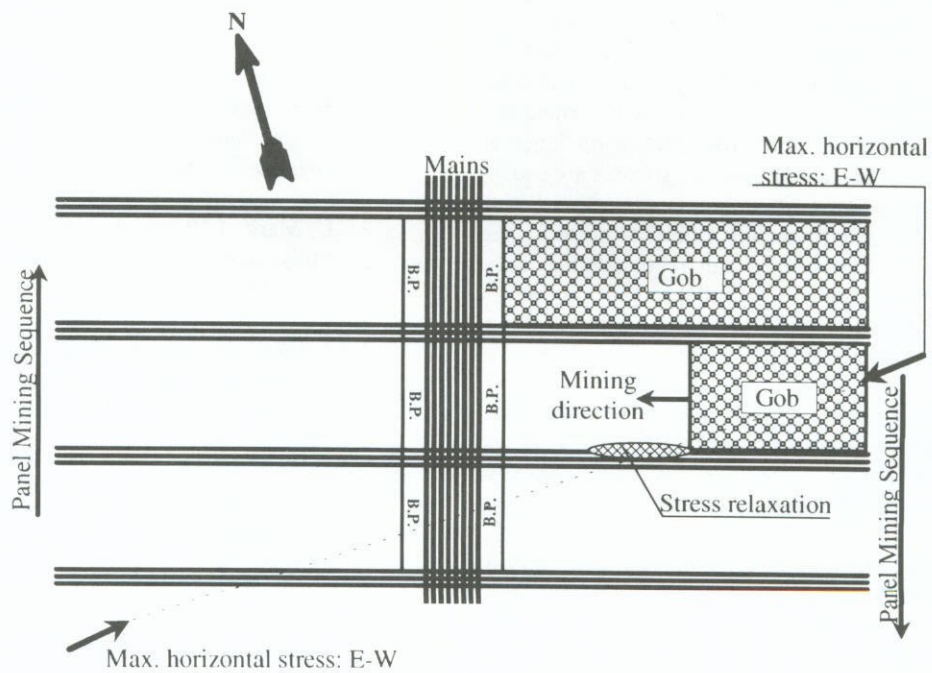


Fig. 5 Horizontal Stress Relaxation over the Outby HG during Longwall Retreat

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Controlling Roof Beam Failures From High Horizontal Stresses in Underground Stone Mines

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ABSTRACT

The U.S., Australian, and United Kingdom coal and the Canadian hard rock mining industries have long recognized the significance of high horizontal stresses as a factor affecting the stability of roof and rib conditions in underground mines. Recently, a growing segment of the U.S. underground stone mining industry has also begun to recognize that horizontal stresses occur in some of its more than 90 mines. Considering the typically high strength and massive nature of limestone, this fact is a revelation in itself. High horizontal stresses produce extensive and sudden rock failures and, in some cases, resulted in injuries to mine workers. Through the years diverse control strategies have been proposed and experimented with. Reorientation of mine entries to reduce stress concentrations have proven successful and are widely accepted in practice. Other solutions, like rock reinforcement, are poorly understood and less accepted in practice. It is the purpose of this National Institute for Occupational Safety and Health (NIOSH) study to develop a better fundamental understanding of these ground control strategies under high horizontal stress conditions through a series of field and laboratory studies. To this end a design technique is presented which provides stone miners with a method for making stability assessments. The consequences of widening rooms, changing geology and horizontal stresses, and different rock bolts on roof beam failures are discussed.

INTRODUCTION

Excessive roof beam sag is considered the major cause of roof failure in U.S. underground stone mines. Roof beams can sag in response to gravity forces that pull subjacent beams away from overlying stable beams. Generally, these roof beams are formed in response to geologic characteristics, such as stratigraphic bedding, and are found in many stone mining provinces. Roof beams of varied thickness have been observed to span rooms ranging from 7 to 16 m (23 to 53 ft), some without any roof support. This can only occur if the beams are sufficiently strong/stiff, massive, and thick. Unfortunately, blasting fractures

the immediate roof beam, adding to the frequency of roof failures.

The strength aspects of this failure criterion is generally easily met by most limestones. Typically, intact (solid) pieces of limestone are very strong, often having compressive strengths of 70 to 210 MPa (10.1 to 30.4 thousand psi or Ksi) and tensile strengths as high as 15 MPa (2.2 Ksi). Limestone has also been found to have a high stiffness, with elastic modulus (E) ranges from 13.8 to 41.4 GPa (2 to 6 million psi or Mpsi).

Not all limestone is massive; in fact, most limestone has horizontal layering that forms beams typically ranging from less than 0.15 to 0.6 m (0.5 to 2 ft) thick (Figure 1). Variations in roof beam thickness are caused by horizontal bedding planes of varying lengths and spacing. Some of these beams contain different numbers of vertical and subvertical discontinuities (vertical joints or fractures and subvertical crossbed planes). These fractures allow local sections of the immediate roof or rib to separate, forming prisms or wedges. These local wedge instabilities can suddenly fail without warning and are especially common in rib falls.

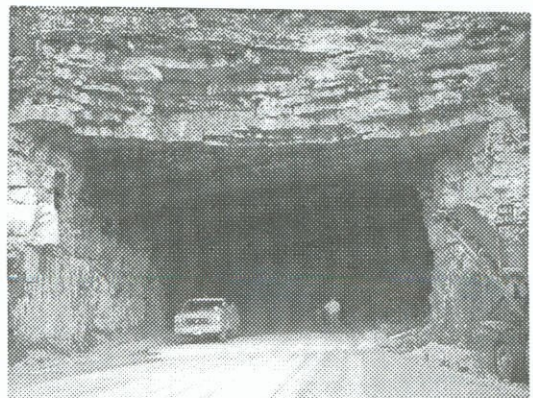


Figure 1 - Limestone mine portal showing a roof composed of numerous flat lying beams separated by bedding planes and ranging in thickness from 0.3 to 0.6 m (1 to 2 ft). It is estimated that greater than 95% of all underground stone mines have dips less than 10°.

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When high horizontal stresses are applied to the ends of thin beams comprising the immediate roof, a buckling failure can occur. The approximate value of the critical load inducing the buckling failure can be determined by Euler's formula (Eq. 1):

$$P_{cr} = \frac{\pi^2 \cdot E \cdot I}{L^2} \quad (\text{Eq. 1})$$

where P_{cr} = Critical load
 E = Elastic Modulus
 I = Moment of Inertia
 L = Length of beam

The value of the critical stress, σ_{cr} , corresponding to the critical load, P_{cr} , which induces buckling failure with respect to different end conditions is determined as follows:

$$\sigma_{cr} = \frac{\pi^2 \cdot E \cdot t^2}{12 \cdot L_e^2} \quad (\text{Eq. 2})$$

where σ_{cr} = Critical stress
 t = Thickness of beam
 L_e = Effective length of beam; for pinned end $L_e = 1 \cdot L$ and fixed end $L_e = 0.5 \cdot L$.

Using Euler's formula for both pin- and fixed-ended beams provides some general parameters related to stone roof beam stability. The strength of beams with different geometries and stiffnesses can be examined by plotting the critical stress at different beam slenderness ratios and elastic moduli. The critical stress for pinned and fixed-end beams is calculated by multiplying the beam length by 1 and 0.5, respectively. Figure 2 shows this relationship for three different elastic moduli: 13.8, 27.6, and 41.4 GPa (2, 4, and 6 Mpsi). The relative potential for buckling failure can be determined by evaluating the position of the calculated critical stress value. If the value plots below the strength/stiffness line, the beam has a low potential for buckling. Conversely, if the value plots above this line, the beam has a high potential for buckling.

As horizontal stresses are increased, roof beam deflection is induced. This deflection causes a maximum displacement in the center of a mined room and can be measured as roof beam sag. At the critical stress state, beam failure is thought to commence. The manner in which failure occurs has become a subject of much study. Roof beams can fail in shear as well as tensions. Multiple low angle shear failures have been recognized at several underground stone mines by the authors, as well as others (Emery, 1964; Parker, 1996; and Petersen, 1996). These failure surfaces are generally oriented perpendicular to the application of the load. Also, some roof beams have been observed to crack at their bottom center from the elevated tension levels in this portion of the deflecting beam.

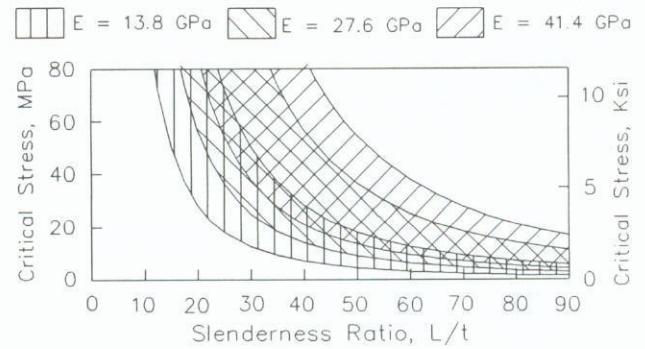


Figure 2 - Critical stress conditions for beams of various geometries and stiffnesses from Euler's formula.

These calculations clearly show that increasing the stiffness and/or the thickness of these beams is expected to dramatically increase their strength and hence the stability of the roof. Certain types of rock bolts can act to join thin layers, enhancing roof strength by increasing its flexural stiffness. Therefore, strategies to increase roof beam strength can minimize roof beam failures from moderate to high horizontal stresses and lessen the exposure of underground stone miners to hazardous conditions.

STRESS CONDITIONS IN U.S. UNDERGROUND STONE MINES

Because vertical stress is a function of the overburden, most underground stone mines are not deep enough to have high vertical stress problems. However, excessive levels of vertical stress are sometimes encountered in benching or multiple layer mining operations where pillar sizing and pillar positioning could produce local high stress concentrations. Unlike vertical stresses, horizontal stresses are directly related to overburden removal or tectonic forces especially in many near-surface conditions.

In the eastern U.S., it has been found that the magnitude of the horizontal stresses often exceeds the vertical stresses by a factor of at least two and can be much greater in shallow mines. Additionally, the ratio between the primary and secondary principal horizontal planes of stress ranges between 1.5 and 1.95 based on 26 measurements at 25 eastern U.S. coal mines (Mark and Mucho, 1994). These same measurements show that the orientation of the principal horizontal stress plane exhibits an ENE to E-W orientation in the northern Appalachian Basin. Because almost 20 underground stone mines operate within this same geologic province, it is expected that they will experience similar regional stress conditions. During 1995 and 1996, almost one-half of the underground stone mines in the U.S. were visited by the authors (Figure 3). High horizontal stress conditions were noted in western Pennsylvania, eastern Kentucky, West Virginia, and southwestern Virginia.

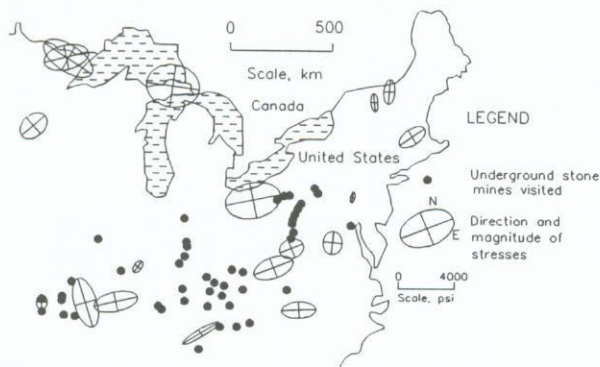


Figure 3 - Location of U.S. underground stone mines visited during this investigation and measurements of in situ stresses at nearby mines and quarries (Bickel, 1993).

Although these horizontal stress trends appear to be consistent over most of the eastern U.S., local variations can occur due to geologic structures (faults, igneous dikes, etc.), stratigraphic features (Parker, 1966), and topographic features (Molinda et al., 1992). For example, Parker (1966) found that horizontal stress at the White Pine Copper Mine in the Upper Peninsula of Michigan varied from an apparent small degree of tension to 13.8 MPa (2,000 psi) of compression. Closer to a fault, stresses rose to 31 MPa (4.5 Ksi). In a deep limestone mine in Kentucky, Parker (1996) measured pressures in roof beams in excess of 96.6 MPa (14 Ksi). Therefore, local confirmation of stress field magnitude and orientation are necessary because stresses can change rapidly. This can be accomplished through measurement or by mapping the orientation of failure patterns developed in response to high horizontal stresses (stress mapping).

Stress mapping uses observational techniques to understand the local horizontal stress conditions. If high horizontal stress conditions exist, they will manifest themselves through several characteristic failure patterns. These patterns have been noticed and discussed by others (Parker, 1973; Gale 1986; and Mucho and Mark, 1994). Many of the same conditions noted by these researchers were observed by the authors at several underground stone mines. The following conditions were particularly noteworthy:

1. The strike of low-angle shear failure planes with associated rock powder oriented perpendicular to maximum horizontal stress direction.
2. Oval or elliptical roof failures oriented perpendicular to maximum horizontal stress direction (Figure 4).
3. Thin delaminated stone layers separated from the immediate roof and oriented perpendicular to maximum horizontal stress direction.
4. Joints and shear-type faults oriented parallel to maximum horizontal stress direction.



Figure 4. - If the roof is not reinforced properly with rock bolts, the beams can progressively fail higher into the roof until a stable roof beam is encountered. Here the roof fall has assumed an elliptical shape.

HIGH HORIZONTAL STRESSES AND ROOF BEAM BEHAVIOR

It has already been shown that the critical stress necessary to induce buckling failure within an Euler beam is dependent on the geometric characteristics and stiffness/strength of the beam. Unfortunately, the Euler beam is an oversimplification of the conditions found in most U.S. underground stone mines. For example, limestone roof beams are neither pinned nor fixed in the mechanical sense. More importantly, limestone roof strata typically contain multiple layers. The writers have found that many failures appear to originate in the lowest beams, or the beams closest to the mine opening (see case study discussed later in this report). This is probably due to the concentration of stresses in the lower beam and constraints to bending in the upper beams, and the fact that stress is highly concentrated in the lower beam at room corners.

Parametric Studies of Beam Thickness, Beam Strength, and Applied Horizontal Stress

Analyzing the complex interaction of multiple beams cannot be accomplished with analytical tools. Therefore, numerical models were employed to examine the interaction of far field applied stress conditions, local beam stiffness characteristics, and unique boundary properties. The finite difference code called FLAC was utilized because it contains realistic capabilities to model beam boundaries through the use of interfaces.

A grid 150 elements square was constructed that defined a bed of limestone 14 m (46 ft) in thickness and surrounded by weaker and thicker clay/shale member. The limestone has a stiffness of 27.6 GPa (4 Mpsi) surrounded with a clay/shale with a stiffness of 6.9 GPa (1 Mpsi). A room 13.7 m (45 ft) in width and 9.1 m

inducing gaps in the modeled roof. The location, areal extent, and thickness of these gaps are the most critical observational data available to identify potential roof instabilities.

All material models were elastic to conform with Euler's stress-based failure criteria and to simulate elastic-beam bending phenomena. Euler's stress-based failure criteria assumes that beam failure is initiated when critical stress initiates beam buckling. However, the elastic numerical model continues to bend the beam until the unbalanced forces dissipate through the finite difference grid through a process called relaxation. Failure conditions are considered to be stress-based, relying on the critical stress condition to define beam stability. Beam horizontal stress conditions were calculated as the average stress along a vertical plane cutting the center of the beam.

Plots of average maximum horizontal stress conditions for five different beam geometries and four applied horizontal stress conditions are shown in Figure 6. In this analysis, the critical stress is calculated for a material with a stiffness of 27.6 GPa (4 Mpsi) using a fixed-end condition. The fixed-end condition is thought to more closely represent the end conditions found in underground mines than the pinned-end condition.

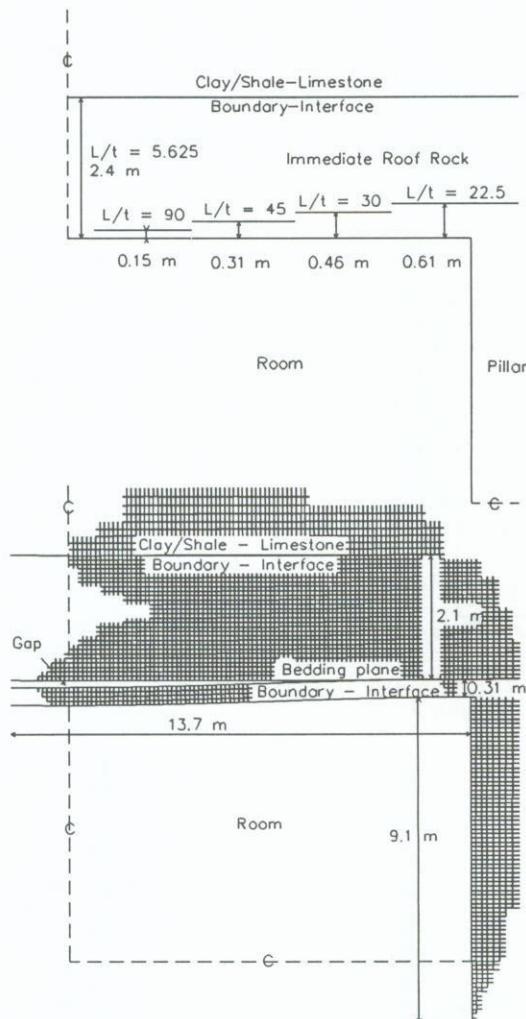


Figure 5 - Dimensions of five beams used in parametric investigations. At high horizontal stresses and slenderness ratios, the beams began to sag, creating a gap in the modeled roof.

The emphasis of these parametric investigations was to examine various shaped beams under varying regional horizontal stress conditions. In this study, five beam slenderness ratios were examined: 5.6, 22.5, 30, 45, and 90 (Figure 5). At high horizontal stress levels, the immediate roof beam began to sag,

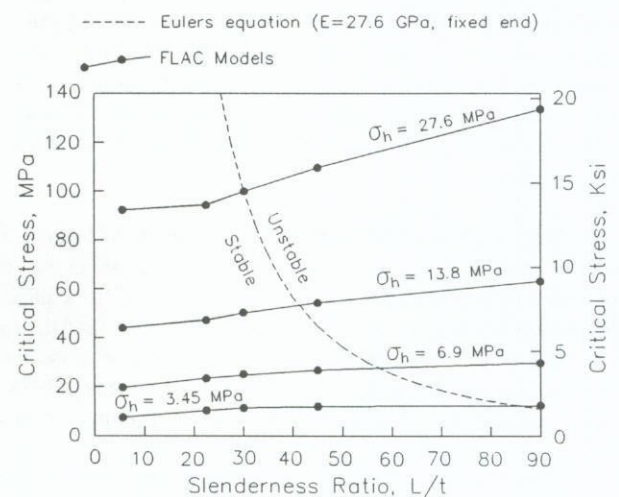


Figure 6 - Potential stability to buckling failure of differently shape roof beams with a varying horizontal stress field, ranging from 3.45 to 27.6 Mpa.

This new analysis supplies a very instructive view of the relationship between beam strength, beam geometry, and the regional horizontal stress field. If two of the conditions are held constant, the third can be altered to determine how it can potentially influence the onset of buckling failure. For example, if the mine is subjected to a 13.8 MPa (2 Ksi) horizontal stress field and the beam has a stiffness of 27.6 GPa (4 Mpsi), then a 0.3 m (1 ft) thick beam in a room 13.7 m (45 ft) wide (slenderness ratio=45) will plot in the unstable zone, indicating a high potential for buckling failure. Conversely, if the beam is 0.6 m (2 ft) thick, the intersection point of the 13.8 MPa and the 22.5 slenderness ratio conditions place this beam in the stable zone.

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indicating a low potential for buckling failure. The following is a list of general trends identified from the parametric analysis:

1. At low regional horizontal stresses (< 3.45 MPa), beam buckling is highly unlikely in strong (high stiffness) rocks with slenderness ratios of less than 90. In this case, localized wedge failures due to gravity forces may be a much more significant factor.
2. At moderate regional horizontal stresses (6.9 to 13.8 MPa), beam buckling is likely to occur in most limestone rocks with intermediate stiffness characteristics and slenderness ratios of greater than 50.
3. At high regional horizontal stresses (> 27.6 MPa), beam buckling is likely to occur under almost any strength limestone rocks with slenderness ratios of greater than 30.

CONTROLLING HIGH HORIZONTAL STRESS THROUGH REINFORCEMENT: A CASE STUDY

Roof bolting has become an accepted means of maintaining roof stability in some stone mines with high horizontal stress conditions. In studying the role of reinforcement, the authors have recognized two basic approaches: 1) support the broken rock mass after the failed rock stabilizes, or 2) reinforce the roof beam to prevent buckling failure from occurring. Different roof support systems can be employed to achieve either of these objectives. The design guidelines presented here stress the second philosophy, i.e., prevention of roof beam buckling failure from occurring.

Available roof reinforcement systems consist of both active and passive types. An active system attempts to pull the roof together by tensioning the bolt head. Conversely, a passive system relies on the sag of the roof beam to actively load the roof bolt. Two major considerations in deploying roof reinforcement are 1) suspension, pinning, or tying wedges or prisms of rock to a more competent member above or 2) reinforcing or beam building by joining individual thinner beams together to increase the effective beam thickness. In some cases where vertical jointing is common, blocks thickened by reinforcement can form the stable Voussoir Arch (Parker, 1996). Other important factors are bolt spacing, length, and capacity, all obviously tied to local stress, geologic, and mining considerations. The options available to achieve the above design issues are many. The types of products includes both mechanical and resin point anchor bolts, full-beam active and passive bolts (combinations of these two systems), and friction stabilizers, such as split-sets and swellex bolts, which rely on radial and frictional forces for anchorage. Because roofbolts in the underground stone industry are generally used on a spot basis, the selection process is complicated by the need to determine how, when, and where to use roof reinforcement. The following section will outline a case study where roof reinforcement was used to control high horizontal stresses.

Site Conditions

In-depth field studies were conducted at an underground operation in Pennsylvania mining the Loyahanna Limestone. The limestone averages 21 m (70 ft) in thickness and is overlain by soft claystones and shales. Horizontal bedding planes are generally not laterally persistent in the Loyahanna Limestone; therefore, it is sometimes necessary to create the roof line with blasting techniques. Immediate roof beam thicknesses were measured that ranged from 0.15 to 0.5 m (0.5 to 1.5 ft). Local practice is to leave approximately 2 m (6 ft) of limestone in the roof. This mine had experienced some problems with controlling a zone of failed roof that had a distinct directional trend to it. It had also periodically used roof support to control this ground with varying degrees of success. During this study, four types of support were used and their performance was evaluated: 1.59-cm (5/8-in) mechanical point anchor bolts, 2.22-cm (7/8-in) fully grouted resin bolts, 2.54-cm (1-in) torque-tensioned resin bolts, and 3.49-cm (1-3/8-in) friction stabilizer bolts (Figure 7). All bolts used in this study were 2.4 m (8 ft) in length.

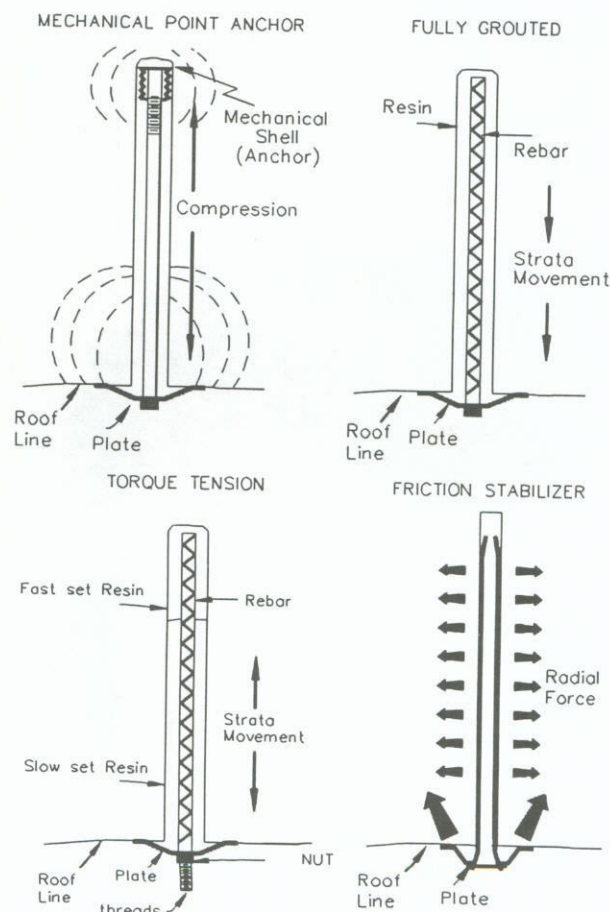


Figure 7. - Four common roof support systems: mechanical point-anchor, fully grouted resin, torque tension resin, and friction stabilizer bolts.

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The characteristics of each of these supports are defined by their load-bearing capacity, stiffness, and postfailure characteristics. These characteristics were measured by conducting pullout tests (Figure 8) in the Greenbrier Limestone at NIOSH's Lake Lynn Underground Laboratory. The 1.59-cm (5/8-in) mechanical point anchor bolts displayed a low level of stiffness as the bolts elastically deformed approximately 0.5 cm (0.2 in), attaining a load of approximately 10 metric tons (11 tons). At this point, the bolts stretch and the anchors began to move or slip. These bolts displaced 14 cm (5.5 in) before failing. The 2.22-cm (7/8-in) fully grouted bolts are much stiffer, achieving approximately 22 metric tons (24 tons) of load at only 0.5 cm (0.15 in) of elastic deformation, primarily at the bolt head. One of the fully grouted bolts failed at this point; the other achieved over 4 cm (1.2 in) of plastic yield and gained an additional 10 metric tons (11 tons). The 2.54-cm (1-in) fully grouted torque-tension bolts were very stiff, achieving almost 35 metric tons (39 tons) of load at just over 0.5 cm (0.15 in) of elastic deformation. Both torque-tension bolts displayed very little plastic yield, displacing a total of only 1 cm (0.39 in) prior to failing between 36 to 37 metric tons (39 to 41 tons). The friction stabilizer bolts, i.e., split-sets and swellex bolts, rely on radial and frictional forces for anchorage. They are excellent at stabilizing wedges because of their ability to provide immediate resistance to deformation as a result of their high initial stiffness. This unique anchorage, however, does not provide very high load resistance, ranging between 4 and 12 metric tons (5 and 14 tons) (Scott, 1989; and Stillborg, 1992). These bolting systems can deform continuously for upwards of 5 cm (2 in) under constant load.

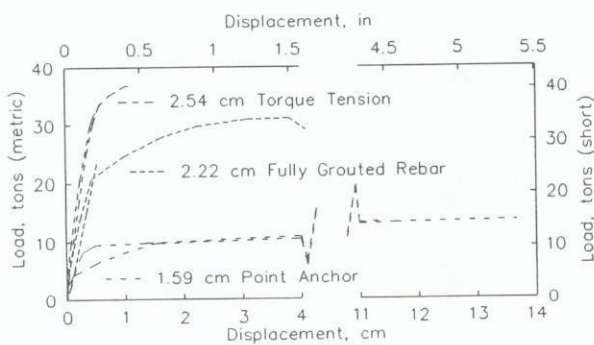


Figure 8. - Pull tests conducted on mechanical point anchor, fully grouted, and torque-tension bolting systems in limestone roof at the Lake Lynn Laboratory.

Measurements of Horizontal Stresses

Effective and efficient roof support systems require an understanding of how high horizontal stress conditions control the behavior of roof strata. An understanding of roof behavior can be accomplished by observational and measurement techniques used to analyze roof rock performance. In addition to stress mapping, 11 in situ stress measurements were conducted with the hydraulic fracturing method at the underground study site. The borehole where the tests occurred was examined with a fiber-optic probe (Figure 9). The roof in this area is composed of alternating sequences of limestone, claystone, and limey

sandstones. The first 1.8 m (6 ft) contains a massive limestone with occasional bedding laminations. Between 1.8 and 2.7 m (6 and 9 ft) the first of numerous red claystone members occurs. Above these red bed members is another competent limestone unit. A relatively thick unit of red claystone is found between 4 and 5.2 m (13 and 17 ft). A massive calcareous sandstone unit is found high in the hole (5.2 to 5.9 m or 17 to 19.5 ft).

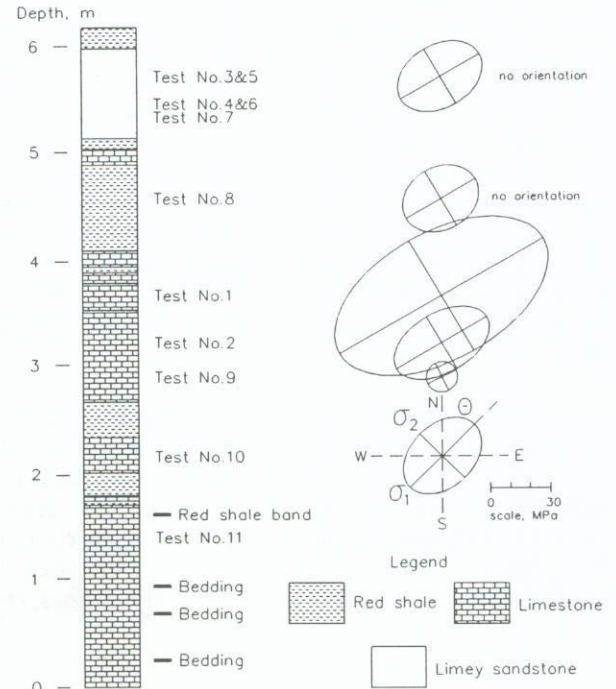


Figure 9. - Stress ellipses from six successful hydraulic fracturing tests within the roof of a Loyallhanna Limestone Mine.

Of the 11 tests, six produced enough data to estimate stress field orientations and magnitudes. Measurements of principal horizontal stress ranged from approximately 10 to 60 MPa (1.4 to 9 Ksi) with an average of approximately 25 MPa (3.6 Ksi). Of the five unsuccessful tests, three were due to our inability to propagate a fracture using pressures of almost 69 MPa (10 Ksi). These are very high pressures and attest to the strong nature of the rock and the high in situ stress conditions. The other two unsuccessful tests were related to either problems with the fracture tool or hydraulic packers.

Of the six successful tests, stress field orientations could be determined from only four tests. Orientations were measured from the direction of vertical propagation produced during fracturing (Figure 9). All four vertical cracks propagated in the N45 to 65E direction. With the exception of test No. 9, all stress field values indicate that elevated horizontal stress conditions occur within this portion of the mine. In the case of test No. 1, the stress field was found to be extremely high. The reasons for this are still under investigation, but are undoubtedly related to the limestone beam-bending process, the location of the softened (fractured) zone within the immediate roof, the stress concentrations around the mine opening, and the variations in beam stiffness, i.e., stiffer layers store high levels of stress. The

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orientations match reasonably well with the regional stress field found in other mines within this region (Figure 3).

Roof Failure Measurements

To understand better how these local horizontal stress conditions affect roof behavior, measurements of roof sag and the associated roof failure were made. The immediate roof began to fail in the middle of July approximately 90 m (300 ft) from the end of a previous directionally controlled roof fall. This original failure took on the appearance of a series of low-angled shear planes cutting or ripping the rock. The orientation of these planes was perpendicular to the orientation of the horizontal stress field as determined from previously discussed stress mapping and stress monitoring activities. Seven sag monitors were quickly placed along the projected failure trend. Two of seven monitors used the 20-point sonic probe; the other five were prototypes of a recently developed NIOSH monitor called the Remote Monitoring Safety System (RMSS) (Iannacchione et al., 1997).

Data collected from three of these monitors are shown in Figure 10. Monitor No. 7 collected sag measurements for almost 70 days prior to total roof collapse. During this time, the roof sagged in three distinct phases. The first phase was marked by a slow but steady sag in the lower roof beam. At approximately 40 days there was a sudden increase in the sag of the beam. The third phase indicated that the beam sag velocity lessened, but

ended in total roof failure. A total of approximately 5 cm (2 in) of roof sag occurred prior to roof collapse. Data from monitor No. 3 showed a much different trend. Unlike monitor No. 7, this instrument was placed close to an existing failure; therefore, significant beam sag could have already occurred. The area began to "cut or rip" on July 26, rapidly extending the zone of failed roof. The roof associated with monitor No. 3 went from stable to unstable in a matter of 5 hours.

Monitor No. 4 was purposely placed slightly away from the main failure trend. The magnitude of sag measured from this instrument was 1/10 the magnitude of the instruments within the failure trend. However, these measurements did show that beam bending and associated shearing extended significant lateral distances on the order of 6 m (20 ft) from the fall's edge and 12 m (40 ft) from the center of the fall. This monitor also showed that while sag was initiated in the lowest beam, beam separations quickly moved much higher in the roof.

Rock Bolt Performance

Friction stabilizer bolts 2.4 m (8 ft) in length were installed in portions of the directional roof failure trend. Unfortunately, many of these bolts failed in the lower 1/3 of the bolt or at the location of the first major beam separation. Some of the bolts were stretched apart while others were sheared, indicating considerable shifting along the beam separations. In either case, these bolts were not of sufficient strength to resist the considerable beam sag.

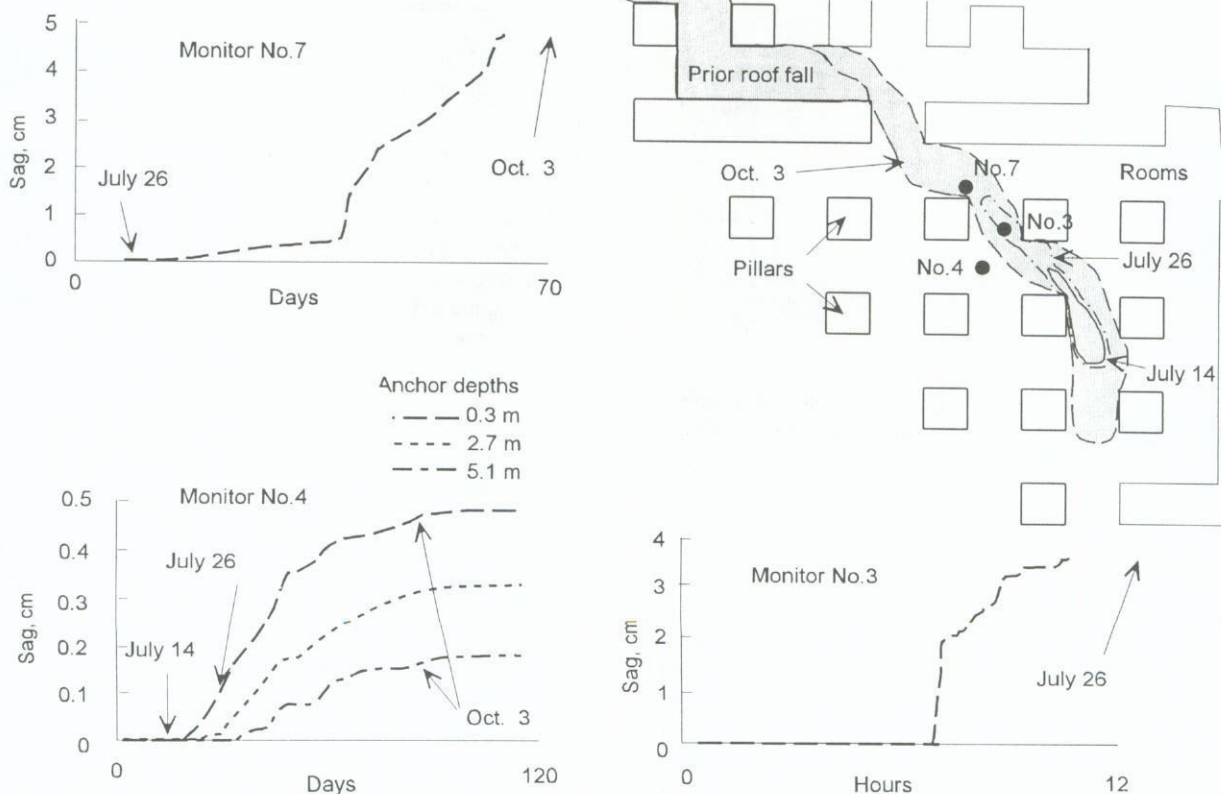


Figure 10. - Roof behavior associated with the large roof fall caused by high horizontal stresses.

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The next bolt tried was a 1.59-cm (5/8-in) mechanical resin-assisted point anchor bolt 2.4 m (8 ft) in length. These bolts were similar in strength to the friction stabilizers, so they would have probably suffered from a similar fate. However, poor installation prevented this bolt from being placed in the high horizontal stress locations. The mechanical resin-assisted anchors were prone to spinners and could sometimes be pulled from their holes by mechanical scalars. It is thought that the limestone hole and the steel bar were both too smooth to allow the resin to bond well between the bolt and the rock, while the mechanical expansion shell seemed to have trouble anchoring into the limestone.

At this point, a new directionally controlled failure began to develop in another room. Mine management decided to try a much stiffer support, the 2.54-cm (1-in) torque-tension resin bolt. These bolts were quickly installed along the projected trend of the new directional failure zone. The fast-set resin along the upper bolt allowed the bolts to be tensioned to an estimated 5 metric tons (6 tons). This tensioning had the added benefit of pulling some of the loose lower beams together. Six RMSS's and three hollow inclusion cells were installed in this area to examine roof behavior. Additionally, a 10-m (30-ft) production bench was extracted to within 14 m (45 ft) of the bolted entry. Through the course of the 8-month monitoring period, stresses in the lower beams have changed as the roof was first loaded then unloaded as the bench extraction zone approached. Although the roof has shown some signs of localized cracking between the bolt heads, no beam collapses have occurred. Additionally, roof sag monitors have shown that very little sag has occurred in the bolted roof area. To this date, no torque tension resin bolts have failed.

Recently, the mine began using 2.22-cm (7/8-in) fully grouted resin bolts in new development areas. The reasons for this are twofold. First, the 2.54-cm (1-in) torque tension bolts must be manually inserted into the bolt holes. Because the bolts are very heavy, long-term usage could cause injury to the operator installing them. Second, because roof bolts are placed soon after mining, minimal separations exist in the roof. Hence, there is little need to tighten the beams together. To date, these bolts have performed without failure in areas where high horizontal stress conditions are thought to have existed.

ROCK REINFORCEMENT PERFORMANCE AND SELECTION

After reviewing the case study data, the question now is why some bolts performed better than others and what can we learn from this study to help in making the best selection in the future. In the case study, it seemed evident that stiffness and load capacity were the controlling factors in successful rock reinforcement performance. Clearly, rock reinforcement acted to strengthen the immediate roof by increasing the effective beam thickness. This enhanced reinforcement action was accomplished in our field test by higher capacity bolts, but it could have also been accomplished by closer bolt spacing. The critical point is that the thinner beams are tied together and act like thicker

beams. Therefore, the critical stress needed to induce buckling is increased, decreasing the potential for buckling.

Although the concept of a stress-based failure criterion is straightforward, its application in controlling high horizontal stresses is not. After all, there is little evidence to suggest that rock reinforcement can significantly change the stress conditions within a roof beam. When rock reinforcement passes through adjacent layers, it does little to change the stresses that would have existed in the beam if the support were not there. This is true until failure or softening takes place. What the support does do is resist beam separation and reduce beam sag. Therefore, to analyze rock reinforcement performance it is necessary to utilize a deformation-based failure criterion.

Controlling Critical Roof Beam Sag Through Roof Reinforcement

To achieve a deformation based failure criterion, a critical sag magnitude needs to be established. It would be ideal to develop this critical sag magnitude based on many observations under a complete suite of geometric and strength characteristics. Unfortunately, there are limited experience and data in this area. Therefore, critical sag magnitudes need to be developed on a case-by-case basis. In this case study, 5 cm (2 in) of total sag preceded a major roof collapse. However, the lower beams began to fail prior to the main beam roof failure. It seems reasonable to assume that beam failure from buckling had begun in the lower beam at one-half of the total sag, or approximately 2.5 cm (1 in). Let's assume that this value is the critical sag magnitude and use it to assess the performance of unsupported and support roof beams.

In the parametric investigations discussed earlier, elastic beam sag occurred in beams with different slenderness ratios and under different horizontal stress conditions. The maximum vertical beam sag for each condition was calculated and displayed in Figure 11. If a match is made between conditions where a beam has a high potential for buckling, as defined by the critical stress failure criteria, and maximum vertical beam sag, then all conditions that produced more than 2.54 cm (1.0 in) of sag would represent a failure beam. This value correlates well with the sag magnitudes measured in the test site, validating the numerical model's performance.

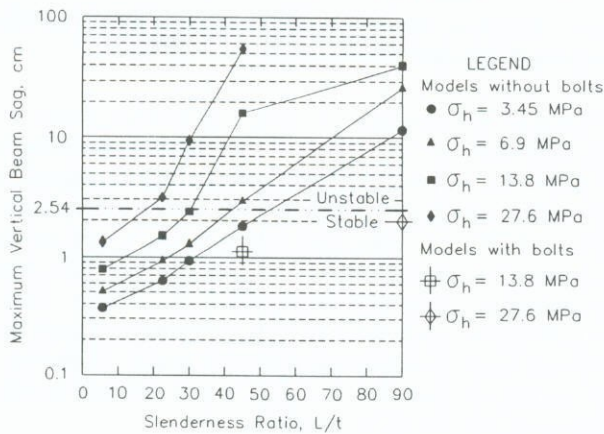


Figure 11. - Maximum vertical beam sag for several beam geometries and applied horizontal stress conditions with and without roof bolts.

To help examine the role played by rock reinforcement in controlling critical roof beam sag, numerical simulations were again used. This time, structural elements were placed in the model to simulate rock bolts. All four supports discussed earlier were evaluated. Material properties representative of these supports were chosen to simulate the material properties observed in the pull-out tests. Mechanical point anchor bolt performance was simulated by attaching the two end-points of the support elements to the finite difference grid. The two resin bolts were glued to their adjacent elements by placing a softer material with properties similar to resin grout between the modeled limestone and steel reinforcement. The friction stabilizer was modeled by removing the resin grout and allowing the steel to become attached directly to the modeled limestone.

These four rock reinforcement systems were run under two conditions presented previously where buckling failure was predicted by both the critical stress and critical displacement failure criteria. These two conditions represent a moderate failure potential, where horizontal stress was 13.8 MPa (2 Ksi) and the slenderness ratio was 45, and a very high failure potential, where the horizontal stress was 27.6 MPa (4 Ksi) and a slenderness ratio 90. All four bolts under both test conditions reduced maximum vertical beam sag to below the 2.5 cm (1 in) critical displacement failure criteria. That is approximately 1 cm (0.4 in) for the moderate failure potential conditions and 2.1 cm (0.8 in) for the very high potential failure conditions.

Selecting Effective Roof Reinforcement

Although the beams have been stabilized against buckling through rock reinforcement, some of the simulated bolts in the model were at or near their yield strength. Therefore it would be inappropriate to say that any kind of bolt could stabilize the beam under these conditions. In fact, our case study showed just the opposite. This is a direct result of the unique manner in which different bolts resist beam bending.

One way to evaluate the individual bolt performance is to examine their load distribution. For example, a mechanical point-anchor bolt compresses the beams between the anchor and the plate (Figure 7). Therefore, as the bolts resist sag the load is evenly distributed along the axis of the steel bar. The bar near the plate or in the threaded anchor section can stretch and fail when excessive loads are encountered. When this happens, the entire bolt loses all reinforcement value. The resin bolts distribute load in a very different fashion. Their resistance to sag is concentrated along that section of the bar where the beam separation is occurring. If the bar would happen to fail at this location, the rest of the intact bolt would still provide some resistance to separation at other locations in the bolted horizon. The friction stabilizers are unique in that they can gain pullout resistance in response to beam separation or lateral shifting. However, loss of contact with the host rock along the bolt axis can reduce its load-bearing capacity.

In addition to examining the influence of bolting on resisting beam sag, numerical simulations were used to evaluate the stability of the individual bolts. This was accomplished by determining the ratio of the axial load to the yield strength of each bolt at or near the beam's critical stress conditions (Figure 12). With a slenderness ratio of 45 and the horizontal stress field of 13.8 MPa (2 Ksi), the point anchor bolts were within 30% of achieving their maximum strength. Both fully grouted and torque tension resin bolts had more than 60% of their strength remaining, while one of the friction stabilizer bolts was at its yield strength. It is assumed that some bolts may fail when their yield strength is achieved and some additional yield takes place. In the case of the point anchor and friction stabilizer bolts, it may take additional sag to develop further plastic deformation before failure occurs. When the slenderness ratio was increased to 90 and the horizontal stress was doubled to 27.6 MPa (4 Ksi), a different set of stability conditions was observed. The point anchor bolts were now at or close to yield. The resin bolts still had at least one-half of their strength remaining, while friction stabilizer bolts were in yield.

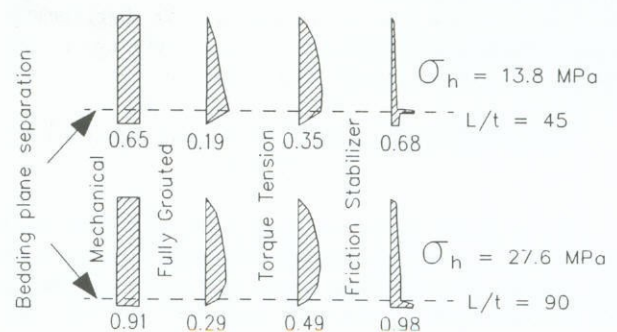


Figure 12. - Profiles of the average ratio of axial load to yield strength for four different bolt types placed under roof conditions that could potentially cause buckling failure of the lower roof beams.

The model results match well with field observations. These observations indicate that no resin bolts failed in the field test,

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while considerable numbers of point anchor and friction stabilizer bolts failed. Bolt stability was directly related to load capacity. In some high horizontal stress conditions, the 6 to 12 metric tons (7 to 14 ton) of load capacity measured in the point anchor and friction stabilizer bolts was not enough to withstand loads generated by the deforming beams.

SUMMARY AND CONCLUSIONS

Roof beam behavior in underground stone mines with high horizontal stress conditions has been characterized using data gathered from the literature, mine visits, and field test sites. These roof beam behavior characteristics are:

1. Roof beams subjected to high horizontal stresses will deform or sag more than those only subjected to gravity loading.
2. Horizontal stresses are not dependent only on overburden. Shallow underground stone mines can have a wide range of horizontal stresses.
3. Horizontal stresses in western Pennsylvania, West Virginia, southwestern Virginia, and eastern Kentucky can be several times greater than the vertical stresses.
4. Observations of roof beam failures associated with high horizontal stresses have been recognized in several underground stone mines by the authors, as well as several other investigators.
5. Measurements of principal horizontal stress at one underground stone mine ranged from approximately 10 to 60 MPa (1.4 to 9 Ksi), with an average of approximately 25 MPa (3.6 Ksi) oriented in the ENE direction.
6. Buckling failure will occur once critical stress conditions exist within roof beams subjected to high horizontal stresses and may appear as tensile or shear failures.
7. Critical stress conditions are influenced by beam stiffness/strength and geometry.
8. At one underground stone mine, approximately 5 cm (2 in) of total roof sag preceded a total roof collapse. It is assumed that roof layers began failing at approximately one-half of this total, or 2 to 3 cm (0.8 to 1.1 in).
9. Elastic numerical models agreed with field data when Euler's stress-based failure criterion was applied. These models were then used to examine the interaction of unique roof beam geometries with far field applied stress conditions.
10. Numerical models with roof bolts suggest that elastic beam sag could be reduced to 2 cm (0.8 in) or less at applied horizontal stress at or below 27.6 MPa (4 Ksi) and slenderness ratios of less than 90.

A design technique for layered limestone roof rock is presented based on the use of both stress- and deformation-based failure criteria. The stress-based failure criterion uses the Euler formula to evaluate the critical stress for buckling of beams of varying stiffness/strength and geometry subjected to different horizontal stresses. The stability of roof beams is also achieved using a deformation-based failure criterion calibrated with field data. The deformation-based failure criterion is used to evaluate the role of rock reinforcement in stabilizing roof beams. Roof bolt selection is based on bolt strength, length, and installation time. Conclusions drawn from this investigation are –

1. At low regional horizontal stresses (< 3.45 MPa), beam buckling is highly unlikely in strong (high stiffness) rocks with slenderness ratios of less than 90. In this case, localized wedge failures due to gravity forces may represent the more hazardous condition.
2. At moderate regional horizontal stresses (6.9 to 13.8 MPa), beam buckling or shearing is likely to occur in most limestone rocks with intermediate stiffness characteristics and ratios of greater than 50.
3. At high regional horizontal stresses (> 27.6 MPa), beam buckling is likely to occur under almost any strength limestone rocks with slenderness ratios of greater than 30.
4. Rock bolts act to reinforce the roof beams, increasing the effective thickness of the beams. These thicker layers produce roof beams that are more stable against buckling failure.
5. Two important rock bolt selection criteria under high horizontal stress conditions are bolt stiffness and load capacity.

The design technique presented provides stone miners a method for making stability assessments of roof beams in high horizontal stress conditions. The consequences of widening rooms, changing geology, and horizontal stresses can be examined. Also, provisions have been made for considering the role of rock reinforcement and selecting an appropriate roof bolt.

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Roof Control Under Conditions of Shallow Depth and High Horizontal Stress Field - A Case Study

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ABSTRACT

Traditionally, roof falls are usually attributed to bad roof strata and high vertical stress related to the overburden depth. However, roof falls under shallow overburden depth are often caused by a high horizontal stress field. Recent studies have proven that the high horizontal stress field, if existing, plays a major role in roof stability. Also the roof conditions are related to the entry directions under a horizontal stress field. Questions are: How to orient the entry to a favorable direction to prevent the roof falls? If the entry cannot be oriented, how to support it? Through a case study in mine A, this paper analyzes the horizontal stress effect on roof stability. The guideline for the entry orientation is given based on the results of analysis. Additionally, suggestions are made for those entries that cannot be oriented to a favorable direction.

INTRODUCTION

Mine A is a room and pillar mine under a shallow overburden depth. Serious roof falls have been experienced recently. To control the roof falls, methods, including using different bolts and changing panel orientations have been tried. However, nothing has worked effectively. By careful review of roof fall history and examination of the geological and mining conditions, it is concluded that the horizontal stress is a very important factor pertaining to the roof problems. In order to evaluate the mine layout and roof control, Jennmar Finite Element Analysis (JFEA) system (Stankus and Guo, 1996) is used to simulate the stress conditions under different panel orientations. Based on the analysis, the panel layout and roof support plan are recommended.

MINING AND GEOLOGICAL CONDITIONS

The panel layout of Mine A is shown in Fig. 1. The 1st East Main is oriented N30°E with a 7-entry system. The 2nd North Main is oriented N30°W with a 7-entry system. The panels left of 2nd North Main are oriented in the E-W direction. The East Submain is oriented N30°E with a 7-entry system. The panels left of the East Submain are oriented N30°W with a 7-entry system. The angle between the entry and cross-cut is 60°.

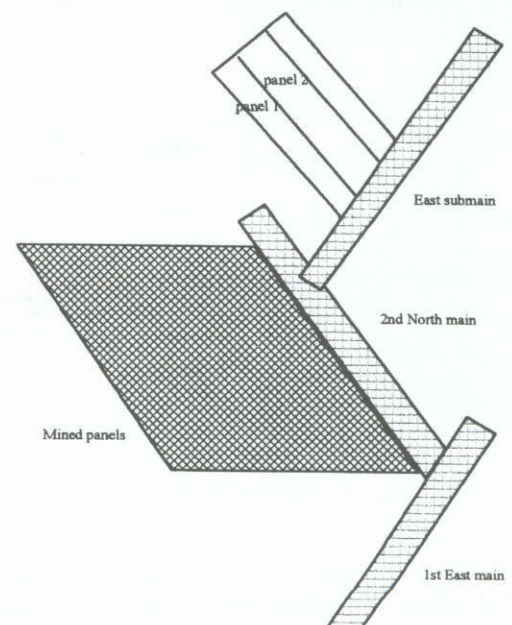


Fig. 1 Current Panel Layout of Mine A

Shown in Fig. 2 is the general lithologic column. The immediate roof primarily consists of shale or shale

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with inter-bedded sandstone. The immediate floor is a claystone layer.

ROOF FALL ANALYSIS

By examining the mine maps on which the historic roof falls were marked, they are summarized as follows:

- In the 1st East Main, a total of 19 roof falls occurred, 16 of them occurred in the entries and only 3 occurred in the crosscuts.

- In the 2nd North Main, a total of 47 roof falls occurred, among them 15 occurred in the right hand crosscuts, only 1 occurred in the left hand crosscuts, and 31 occurred in the entries.

- The panels left of 2nd North Main were successfully mined out without any roof fall.

- In the East Submain, 1 roof fall occurred in the entry.

- At the left of East Submain, 6 roof falls occurred in the first panel, among them 4 were in the entries, 1 in the left side crosscut, and 1 in the right side crosscut.

- Because of severe roof falls in the second panel, it had to turn at SM35+00 to make the panel parallel to the East Submain. However, after the turn, the roof conditions did not improve. Finally, the section was abandoned and moved to the East Submain.

Based on the roof fall occurrence, it is concluded that the roof falls are related to the in-situ horizontal stress. The relationship between the frequency of roof falls and the angle (α) of in-situ major horizontal stress direction to the mining direction is listed in Table 1.

Table 1 Roof Fall Frequency

			α	roof fall frequency
1st East Main	Entry		52	16
	Crosscut	left	68	2
		right	8	1
2nd North Main	Entry		68	31
	Crosscut	left	8	1
		right	52	15
East Submain	Entry		52	1
	Crosscut	left	68	0
		right	8	0
Panel 1	Entry		68	4
	Crosscut	left	8	1
		right	52	1

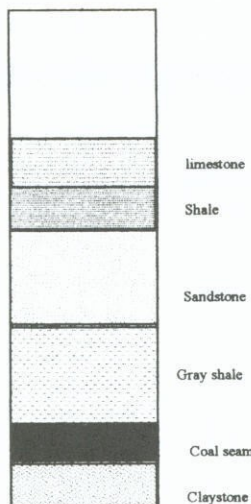


Fig. 2 General Lithologic Column

Fig. 3 shows the results of stratascoping two boreholes. Basically the first 0"-23" is comprised of a darker soft shale layer with laminations. From 23"-35", the roof is medium shale with obvious bedding planes. The hole was fragmented and some separations were observed. From 35"-77", the roof is a uniform light shale with some laminations. After 77" there are frequent vivid calcium bands approximately 3" or 6" all the way to top of the borehole.

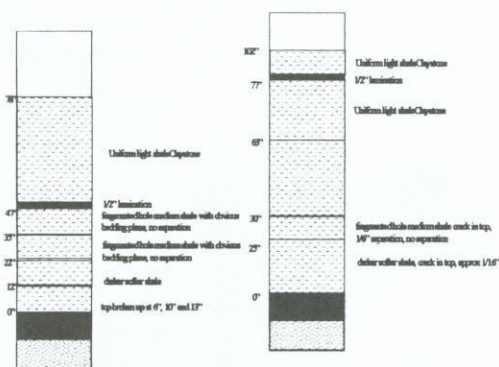


Fig. 3 Results of Stratascoping

The overburden depth is 150'. Horizontal stress information is not available. However, based on experience in this area, the in-situ horizontal stress is approximately:

Major horizontal stress: 1800 psi
 Minor horizontal stress: 700 psi
 Orientation: N82°E

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From Table 1, it can be seen that the total number of roof falls is 73. Only 3 of them occurred in the entry or crosscut which is 8° angled with the major horizontal stress, 33 of them occurred in the entry or crosscut which is 52° angled with the major horizontal stress, and 37 of them occurred in the entry or crosscut which is 68° angled with the major horizontal stress. The distribution of the roof falls vs. opening angle to the horizontal stress direction is shown in Fig. 4. Obviously, the orientation of the entry is a significant factor affecting the roof stability.

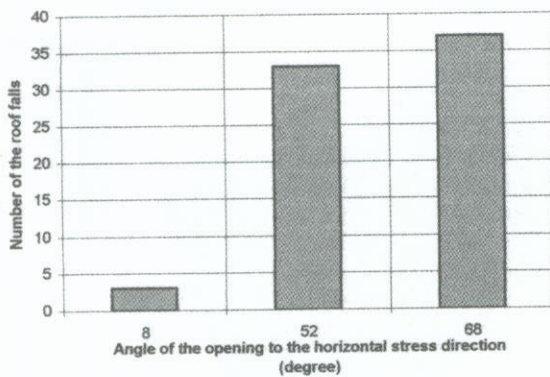


Fig. 4 Roof Fall Distribution

FINITE ELEMENT ANALYSIS

In order to design the mine layout and roof support, five 3-D finite element models were established to simulate the effect of the angle (α) of the major horizontal stress direction to the mining direction. The angles (α) for five models are 0° , 22.5° , 45° , 67.5° , and 90° . The horizontal stress field applied to the models is:

Major horizontal stress: 1800 psi
Minor horizontal stress: 700 psi

Shown in Fig. 5 is the 3-D finite element model including the three-entry system.

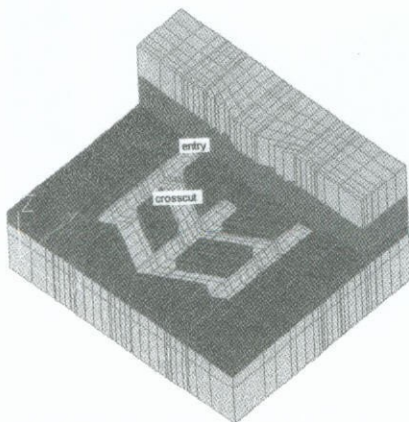


Fig. 5 3-D Finite Element Model

Fig. 6 is a plane view of the model at seam level. The stresses at the points marked on the Fig. 6 will be analyzed in detail.

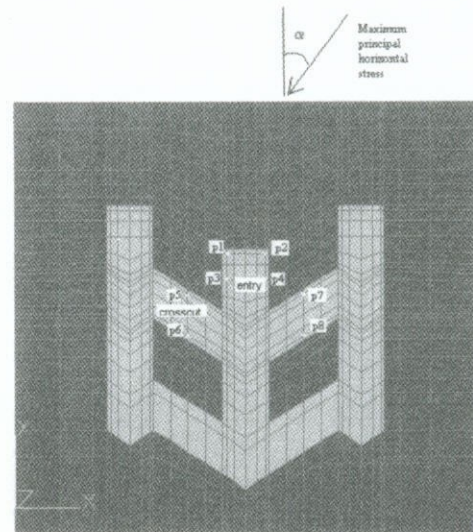


Fig. 6 Plane View of the Model at Seam Level

Stress Distribution

Shown in Fig. 7 is the 3-D shear stress (Von-Mises) distribution on the roof line level when $\alpha=0^\circ$. In this model, the entry is parallel to the major horizontal stress and the crosscut is 60° angled with the major horizontal stress. It can be seen that the stress in the crosscut is higher than that in the entry. The maximum stress in the crosscut is 2950 psi. The maximum stress in the face area is 2450 psi. The maximum stress in the entry is 1630 psi.

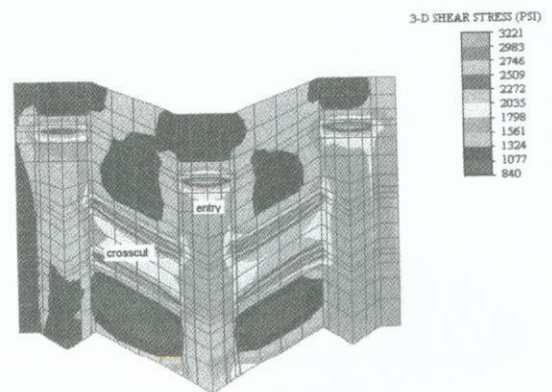


Fig. 7 3-D Shear Stress Distribution When $\alpha = 0^\circ$

Shown in Fig. 8 is the 3-D shear stress (Von-Mises) distribution on the roof line level when $\alpha=22.5^\circ$.

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In this model, the entry is 22.5° angled with the major horizontal stress; the crosscut on the right side is 37.5° angled with the major horizontal stress; and the crosscut on the left side is 82.5° angled with the major horizontal stress. It can be seen that the maximum stress occurred at the right corner of the intersection with a magnitude of 3197 psi. The stress is 2620 psi in the right crosscut and 3080 psi in the left. The maximum stress in the face area is 2509 psi. The maximum stress in the entry is 2100 psi.

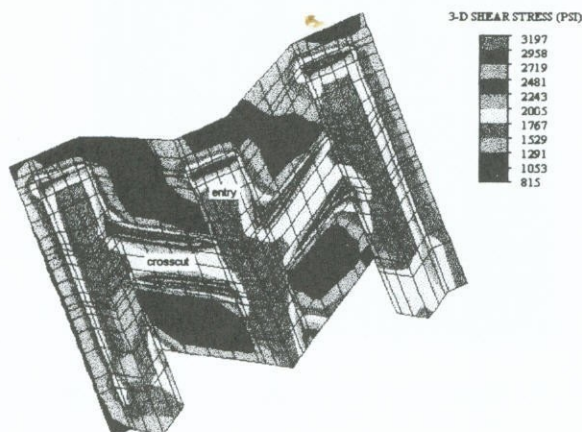


Fig. 8 3-D Shear Stress Distribution
When $\alpha = 22.5^\circ$

Shown in Fig. 9 is the 3-D shear stress (Von-Mises) distribution on the roof line level when $\alpha=45^\circ$. In this model, the entry is 45° angled with the major horizontal stress; the crosscut on the right side is 75° angled with the major horizontal stress; and the crosscut on the left side is 15° angled with the major horizontal stress. It can be seen that the maximum stress occurred at the left corner of the intersection with a magnitude of 3236 psi. The stress is 2800 psi in the right crosscut and 1850 in the left. The maximum stress in the face area is 2580 psi. The maximum stress in the entry is 2550 psi.

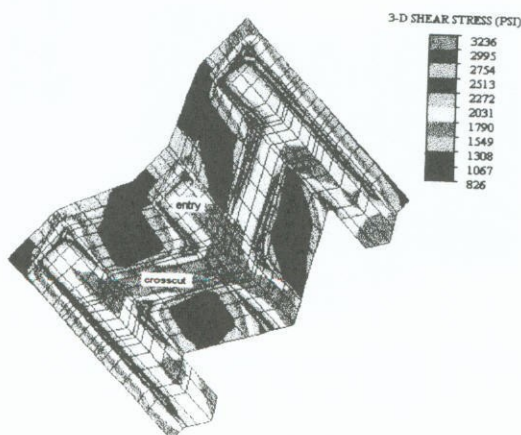


Fig. 9 3-D Shear Stress Distribution
When $\alpha = 45^\circ$

Shown in Fig. 10 is the 3-D shear stress (Von-Mises) distribution on the roof line level when $\alpha=67.5^\circ$. In this model, the entry is 67.5° angled with the major horizontal stress; the crosscut on right side is 7.5° angled with the major horizontal stress; and the crosscut on left side is 52.5° angled with the major horizontal stress. It can be seen that the maximum stress occurred at the lower left corner of the intersection with a magnitude of 3308 psi. The stress is 1520 psi in the right crosscut and 2500 psi in left. The maximum stress in the face area is 2570 psi. The maximum stress in the entry is 2700 psi.

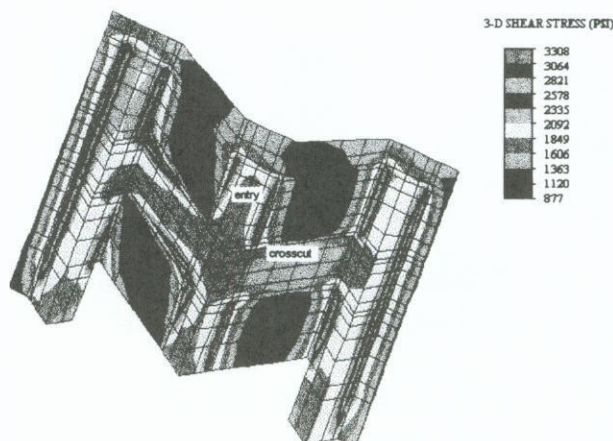


Fig. 10 3-D Shear Stress Distribution
When $\alpha = 67.5^\circ$

Shown in Fig. 11 is the 3-D shear stress (Von-Mises) distribution on the roof line level when $\alpha=90^\circ$. In this model, the entry is perpendicular to and crosscut is 30° angled with the major horizontal stress. It can be seen that the stress in the entry is much higher than in the crosscut. The maximum stress in the entry is 2750 psi. The maximum stress in the face area is 2500 psi. The maximum stress in the crosscut is 1920 psi.

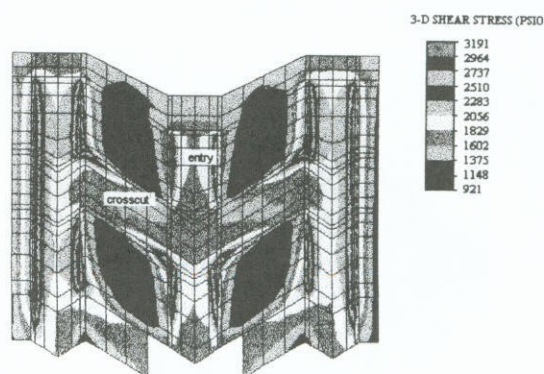


Fig. 11 3-D Shear Stress Distribution
When $\alpha = 90^\circ$

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Effect of the Angle (α) of Major Horizontal Stress Direction to the Entry Direction

Shown in Fig. 12 is the 3-D shear stress changes with the angle (α) of the major horizontal stress to the entry direction at points 1 (P1) and 2 (P2) (see Fig. 4) in the face area. Overall the stress increases as α increases. The stress at point 1, which is intersected by major horizontal stress, is always higher than that at point 2.

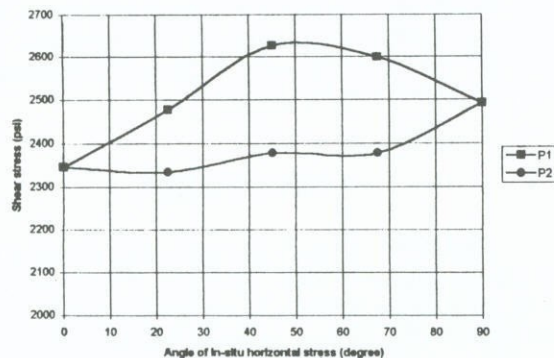


Fig. 12 3-D Shear Stress change vs. Angle (α) at Points 1 and 2

Shown in Fig. 13 is the 3-D shear stress changes with the angle (α) of the major horizontal stress to the entry direction at points 3 (P3) and 4 (P4) (see Fig. 4) in the entry. Overall the stress increases as the α increases. However, from 0° to 50° , the stress is affected by α significantly. After 50° , changing α has minimal effect on the stress in the entry, which means the roof stability does not change dramatically.

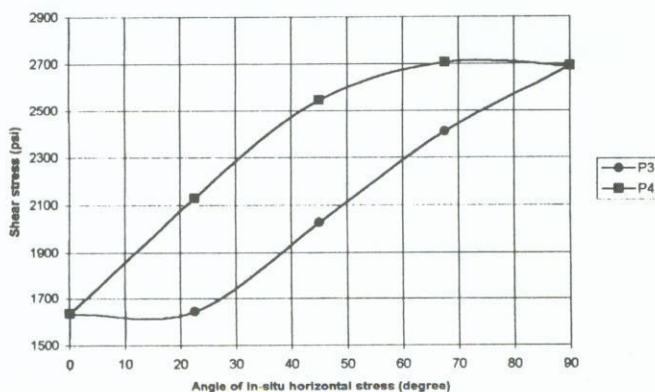


Fig. 13 3-D Shear Stress Change vs. Angle (α) at Points 3 and 4

Shown in Fig. 14 is the 3-D shear stress changes with the angle (α) of the major horizontal stress to the entry direction at points 5 (P5), 6 (P6), 7 (P7), and 8 (P8) (see Fig. 4) in the crosscut. Overall the stress decreases as α increases. However, from 0° to 50° , the stress is not affected by α significantly. After 50° , a little increase of α can decrease the stress dramatically.

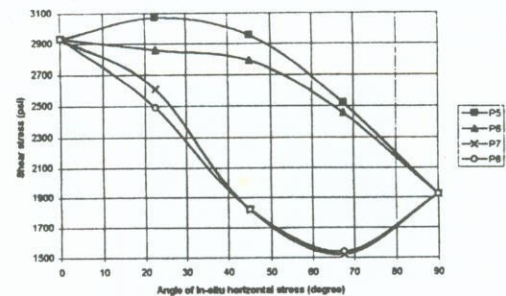


Fig. 14 3-D Shear Stress Change vs. Angle (α) at Points 5, 6, 7 and 8

MINE LAYOUT AND ROOF SUPPORT

Mine Layout

In the active mining area, the East Submain is oriented $N30^\circ E$ which is 52° to the major horizontal direction. From Fig. 12 it can be seen that the shear stress in the entry roof is almost at a maximum. In the soft roof strata condition, roof falls are expected. This has been experienced in 1st East Main.

The sections on the left side of East Main are oriented at $N30^\circ W$ which is 68° to the major horizontal stress direction. The stress is higher than that in the East Submain thus the roof condition should be worse.

The guideline for panel layout is shown in Fig. 15. In the lighter area, where the mining direction is in the range of $N58^\circ W$ to $S42^\circ W$ and $N42^\circ E$ to $S58^\circ E$, the entry roof is in good condition. Supplemental support may not be needed if the roof is properly supported with primary support. In the darker area, the roof is in a bad condition. Here, supplemental support may be required, especially in the soft roof strata condition.

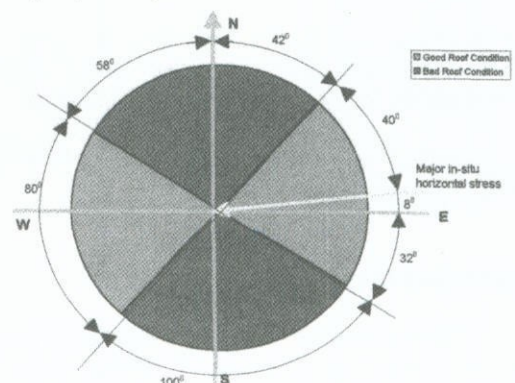


Fig. 15 Guideline for Panel Layout



Roof Support

Fig. 16 is the 3-D shear stress distribution at cross-cut P3 - P4 (see Fig. 5) when the mining direction is perpendicular to the major in-situ horizontal condition.

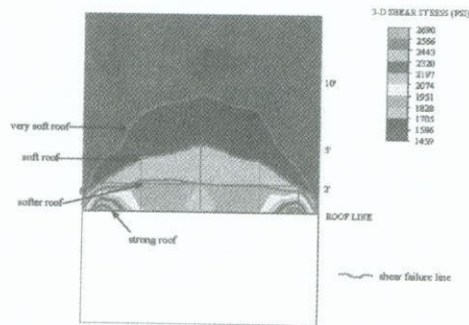


Fig. 16 3-D Shear Stress Distribution at Crosscut P3-P4

The possible roof shear failure line predicted is also shown in Fig. 16. Based on the different roof strata conditions, the roof falls are analyzed as follows:

- In the strong roof condition, the roof may not fail but may experience some small shear failure in the roof corner.
- In the softer roof condition, shear failure may extend to the 2' level and cross through the roof horizontally. If the roof is not properly supported or before it is bolted, the first 2' of immediate roof may fail.
- In the soft roof condition, the shear failure may extend to the 5' level and cross through the roof. In this case, even though the roof is bolted using less than a 5' bolt, the roof will break and fail above the bolted horizon. Proper supplemental support will be required.
- In the very soft roof condition, the shear failure may extend to 8' above roof line level. Again the supplemental support is necessary in this case.

Based on the analysis, the following roof support plans are suggested:

Primary Support

The results of the stratascope show that there are a number of obvious bedding planes and laminations which are prone to shear failure. The results of finite element analysis show the possible shear failure line. In order to control the shear failure

caused by horizontal stress, bolt pre-tension is required. This would be especially true when the mining direction is in the darker area (see Fig. 15). Therefore, a 4' INSTAL type bolt is recommended in conjunction with a T-5 channel.

Supplemental Support

As described earlier, if the mining direction is in the darker area (see Fig. 15) and the roof strata are soft, supplemental support will be necessary. The JM tensioned cable truss system is recommended which has been proven to be very effective in controlling the high horizontal stress.

CONCLUSIONS

Based on the study of historic roof falls, the high horizontal stress field is identified as an important factor contributing to the roof instability in Mine A. The results of the finite element analysis provide a guideline for orienting the panel in a favorable direction to avoid the high horizontal stress concentration. Accordingly, the primary and supplemental roof support plans are recommended for those entries that cannot be orientated in a favorable direction.

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- Stankus, J.C. and Guo, S., 1996, "Computer Automated Finite Element Analysis - A Powerful Tool for Fast Mine Design and Ground Control Problem Diagnosis and Solving," *Proc. 5th Conference on the Use of Computers in the Coal Industry*, edited by S.D. Thompson, R.L. Grayson, Y.J. Wang, West Virginia University, pp. 108-155.



Assessment of Roadway and Yielding-pillar Performance During Retreat Longwall Extraction at a United Kingdom Deep Mine

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ABSTRACT

During the last 5 years a United Kingdom colliery has utilised a yielding-pillar configuration to extract a relatively thick coal seam at a depth of 640 m. As part of ongoing research into the siting of roadways adjacent to extracted longwall panels, several field monitoring sites were established at differing locations to investigate roadway stability and yielding-pillar performance. Characteristic roadway deformation has been identified together with strata softening pathways which indicate the temporal increase in the height of softening into the roadway roof. In the assessment of such a complex problem numerical modelling, using FLAC, has been shown to contribute vital additional information, and provide further insights into the mechanisms controlling strata behaviour.

INTRODUCTION

The mine has over 5 years of practical experience employing yielding-pillars between longwall retreat panels. In earlier layouts, tailgate's were driven with 10 m pillars to previously formed goafs. The last of these suffered extreme deformation on retreat. Severe deterioration in the condition of both the pillar, and floor with visible signs of roof instability led to a reassessment of pillar widths, which resulted in the adoption of the current 15 m system. An appropriate yielding-pillar is designed to avoid two undesirable conditions, (i) too small a pillar width which risks roof instability from goaf override, and (ii) too large a pillar width which promotes high stresses in the proximity of the tailgate.

An observational approach to the investigation has been applied using instrumentation to monitor strata behaviour. This 'design-by-measurement' approach, used in the introduction of both roofbolting and cablebolting to the

United Kingdom coal mining industry, is described by Hurt *et al.* (1) and Kent *et al.* (2).

The monitoring results have been used to highlight the performance of the pillars and adjacent roadways throughout drivage and face retreat. Consideration was given to the changing strata conditions, installation of secondary reinforcement and the remedial measures implemented along the length of the pillars. The results are presented within the context of examining the lateral interaction effects caused by mining close to previous excavations. Direct comparison of monitoring results between the two pillar sizes is difficult because of variations in the support and reinforcement installed, and the differing timing of mining activities undertaken in each tailgate. Therefore case history results, to assess the variable performance of the current 15 m pillar, are described with an appreciation of the key factors controlling deformation.

A numerical modelling approach, using FLAC, has provided further insights into mechanisms controlling the strata deformation. The FLAC model was validated using in-situ measurements prior to performing sensitivity analyses on critical input parameters. Model properties were based on triaxial test data obtained from roof core samples. Parametric modelling was used to investigate several effects, these included variations in pillar width, pillar confinement, bedding properties and loading conditions.

SITE CHARACTERISTICS

Geometry

The deep mine in question is currently extracting 4 m of coal from a total combined seam thickness of 6.5-7 m by retreat longwall, at a depth below surface of 640 m. The described study included a programme of strata/pillar deformation and stress monitoring undertaken within a tailgate, separated from adjacent goaf by a 15 m yielding-pillar, as indicated in figure 1. Panel sequencing involved alternate extraction of longwall's in different districts to enable drivage of tailgate's alongside adjacent goafs. The

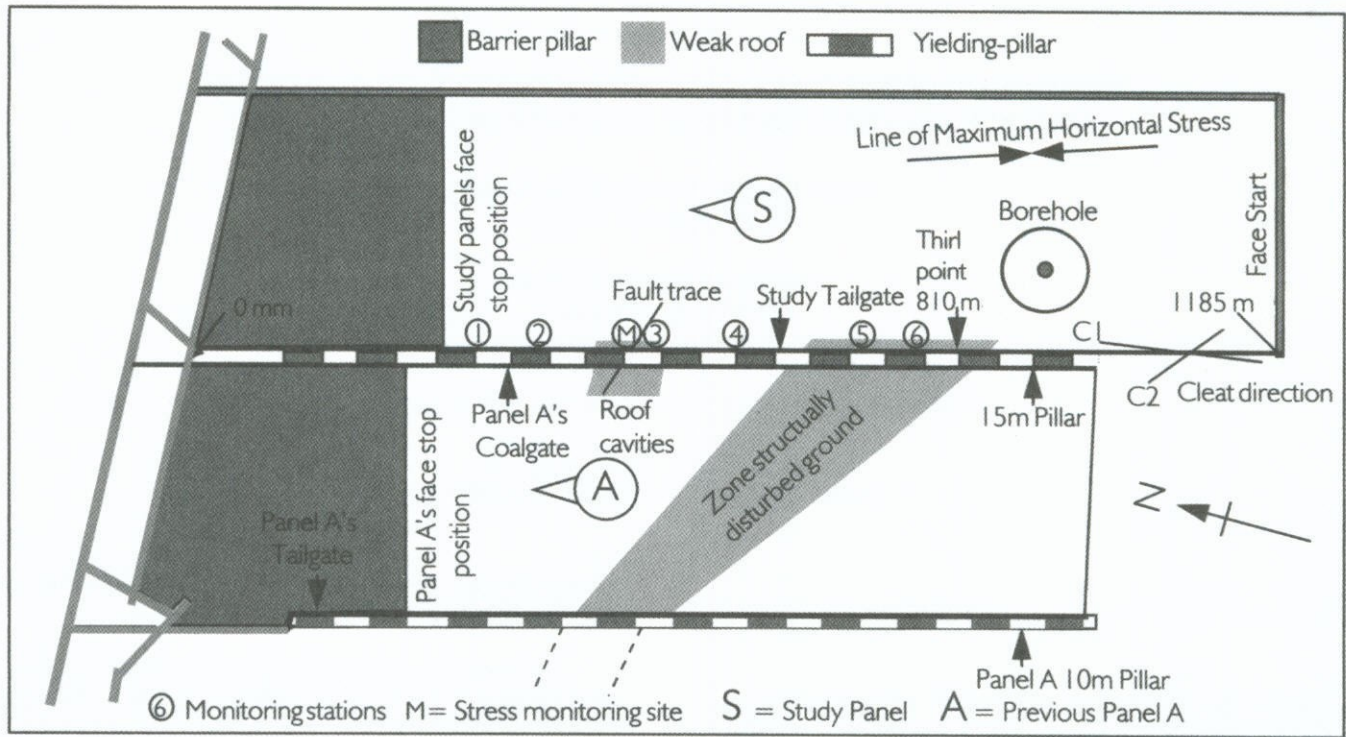


Figure 1. District layout showing monitoring sites, location of gate roads and structural features.

tailgate was developed following complete extraction of the adjacent panel, being driven from both face and outbye ends terminating in a thirl point at the 810 m mark. The previous panel A shown in figure 1, was designed using a 10 m pillar between itself and the adjacent panel. Panel A's tailgate suffered only low displacements on drivage, but large deformations following the onset of retreat. During early retreat these large deformations influenced the behaviour of the whole tailgate, irrespective of its distance from the faceline.

Lithology and Structure

The immediate roof horizon is characterised by a weak clay-parted mudstone with a minimum thickness of 2 m. This weak horizon often suffers extensive shear leading to large levels of roof dilation and rapidly extending heights of softening.

Triaxial testing of roof core samples taken from the tailgate indicated the uniaxial compressive strength of the mudstone's as being between 28 and 35 MPa. From roof core data, and borehole information, the thickness of the mudstone increased from approximately 2 m near the face start position to > 4 m at the tailgate's outbye end. Due to the presence of this roof strata approximately 0.2 m of coal was left on drivage to form the immediate roof. Three metres into the roof the strata coarsens to silts and sands with two prominent sandstone beds positioned at 7 and 9 m into the roof, as shown in figure 2.

The primary cleat in the tailgate has a bearing of between 354-360° compared with a gate bearing of 343°. This can result in weak rib conditions, and correspondingly high rib displacements, similar characteristics have been described by

previous authors, particularly Vervoort (3). Because of the high horizontal stress ratio the practice of mine layout has been to orientate the tail and coalgates close to the orientation of maximum horizontal stress, as shown in figure 1. This avoids the adverse conditions in the coalgates caused by drivage at high angles to the maximum horizontal stress as described by Gale and Blackwood (4).

Structurally an area of weak faulted and disturbed ground was anticipated between the 600 and 850 m marks following conditions experienced in the previous panel. Such disturbed ground was recognised as potentially a high deformation region and supplementary rockbolt support was used to control high displacements on drivage, as discussed by Barratt (5). A second zone of weak roof was identified around the 435 m mark caused by a fault having a WNW/ESE trend, which resulted in numerous cavities in the immediate roof on drivage.

A summary of the recorded in-situ stress magnitudes obtained from Hollow Inclusion (HI) cells are given in table 1.

General support details

The tailgate was supported by 4.5 m x 3.05 m flat topped delta steel arches set at 0.7 m centres. Supplementary support was provided by rockbolting using, 8 x 2.4 m fully encapsulated 35 Tonne roofbolts and 3 x 1.8 m rib bolts at each side also placed at 0.7 m spacings. Additional reinforcement was provided, in places on drivage, by 4 m 54 Tonne Flexible Strand Roofbolts (FSR's), 2 at 0.7 m spacings. These were placed at the face of the heading in some circumstances and as outbye remedial support in other circumstances. In all cases FSR's were set with at least 2.5 m



Study tailgate Description	Lithological Log	UCS MPa	E GPa	UCS E ^{-10^-3}
Siltstone				
Sandstone		91.4 37.9	21.6 10	4.21 3.79
Siltstone Fine, occasional Sand		59.54 62.74	14.5 18.4	4.08 3.4
Sandstone Fine		89.99	23.3	3.84
Siltstone Sandy laminae.		62.16	18.8	3.30
Siltstone		48.5	13.0	3.72
Siltstone Fine muddy laminae		50.68 51.93	13.5 15.3	3.74 3.38
Mudstone dark.		52.9	14.5	3.62
Mudstone		57.48	12.4	4.62
Mudstone Silty, laminated.		43.86 49.39	12.8 9.85	3.42 5.01
Mudstone				
Coal Massive				
Roof Horizon				

Figure 2. Study tailgate roof lithology.

Location	Max Horiz.	Min Horiz.	Vert	Stress Rat.	Bearing Max
Nearby Colliery #1	21	9	16	2.33	344°
Nearby Colliery #2	27	11.5	17	2.35	332°
Previous T/G. †	17.5	11	16	1.59	315°

Table 1. Measured in-situ stresses

Stress magnitudes in MPa. † This measurement was carried out from a previous solid driftage at a depth of 10.1 m and is expected to approach virgin conditions.

of resin encapsulation. Eight metre 60 Tonne twin birdcaged cablebolts provided remedial support to sections of the tailgate which encountered either high roof displacement rates, or significant softening above the bolted height. Previous experience at the mine has shown the importance of remedial reinforcement in controlling strata deformation around roadways. Following tailgate development, and prior to face retreat, "Hercules" cribbing was placed 0.3 m from the pillar ribside with a maximum centre-line spacing of 0.7 m, generally installed at no less than 55 m ahead of the face.

Monitoring system details

The roof and rib displacements were monitored in detail along the length of the tailgate at 6 stations, by wire and sonic extensometers anchored up to 15 m into the strata. Stations

2, 3 and 5 included horizontal wire extensometry recording displacements in the solid and pillar ribside. Table 2, below, provides a summary of the monitoring programme, (the final reading value being the distance the station was ahead of the face at its last reading).

In addition roof displacements were measured by standard dual height telltales, with anchor points at 2.1 m and 5.0 m which were installed along the length of the tailgate at 20 m intervals. The use of telltales in the deep mines of the UK is practically universal, their design and use has been described by Bigby and Egan (6). In cablebolted areas cablebolt telltales were also installed these having anchor points positioned at 7 and 9 m into the roof.

Station	Meter mark	Height into roof	Depth into pillar	Depth into solid	Final Reading
Stn 1	284	6.5 (s) 6.5 (s)	-	-	230
Stn 2	360	15 (w)	10 (w)	5 (w)	57
Stn 3	540	11 (w)	-	10 (w)	125
Stn 4	601	6.5 (s)	-	-	95
Stn 5	702	13 (w)	6 (w)	-	139
Stn 6	737	13 (w)	-	-	102

*Distances in metres, Extensometers : w=wire s=sonic

Two 3D stress monitoring cells were installed following the absolute stress measurement by overcoring at the 510 m mark. The first cell was positioned 7 m into the pillar and 5 m above the tailgate roof, the second was installed 5.2 m into the solid ribside at a height of 4 m above the roadway roof. This exercise occurred following completion of the tailgate driftage, but prior to the start of face retreat. The main purpose of the stress measurement exercise was to examine the vertical and horizontal stress retention in the pillar on its formation, and then to monitor the stress changes as the face approached.

MONITORING RESULTS

Stress measurements

The results of the overcoring using one HI cell and one ANZI cell are given in table 3, along with a comparison of results from similar overcores carried out on two previous panels which employed a 10 m yielding-pillar. These overcore results indicate the yielding nature of the pillars, suggested by a reduction in horizontal stress and vertical stress from cover load. The failure of the overcore in the solid ribside, through massive debonding, was suggestive of installation into already yielded strata. Such a response indicates significant load transfer from the yielding-pillar to the solid ribside, where an extensive yield zone was being generated. The results of the stress monitoring programme are presented in figure 3, and confirm this behaviour, but also indicate the pillars ability to carry further loads on retreat.

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Location	Vert.	Horiz. *	Max Horiz.	Minor Horiz.	Pillar Width
Study Panel	12.9	8.6	13.3	9.5	15m
Previous panel A	9.5	2.3	4.0	2.6	10m
Previous panel B	10.9	4.8	12.8	5.6	10m

Table 3. Measured stresses above yielding-pillars

*Refers to the horizontal stress perpendicular to the roadway length. Italics refers to Secondary Principal Stress in the horizontal plane. All values in MPa.

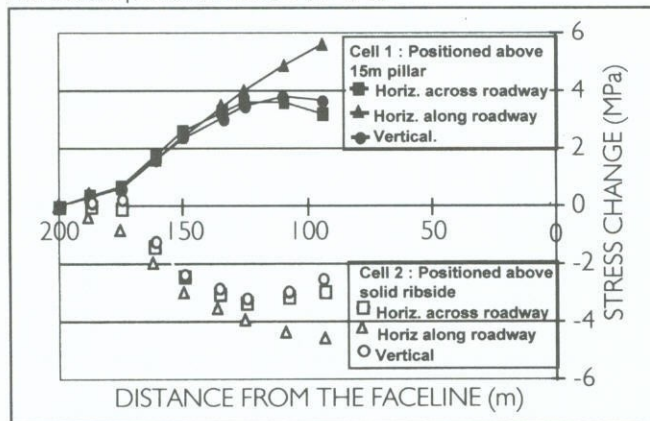


Figure 3. Results from tailgate stress monitoring on retreat.

Roof Behaviour

Figure 4 shows the total maximum recorded roof displacements on retreat for the study tailgate and the adjacent (panel A's) coalgate. The lower margins of the shaded areas represent the total displacements following drivage. Deformation data for the adjacent coalgate (on drivage) has been used for comparison purposes, to identify structural and geological influences and provide a basis for interpretation of the tailgate/yielding-pillar behaviour. Roof displacements recorded both by tell-tale devices and extensometers, show a characteristically wide variation throughout both drivage and retreat.

Also shown in Figure 4 is a summary of the installed additional and remedial support, indicating that this additional reinforcement was targeted in those areas where roof instability was anticipated. Typically these areas were reinforced by FSR's on drivage. The timing of "Hercules" crib placement was variable, some were installed up to 500 m ahead of the faceline whilst others were placed only 55 m ahead of the retreating faceline. Minor peaks in the displacements along the tailgate reflect the influence of panel A's extraction, at the 250 and 1015 m marks caused by effects of horizontal stress notching.

The adjacent coalgate, unlike the tailgate was essentially driven into virgin ground and provides an opportunity to identify the inherent structural disturbances and variations in immediate geology and stress. Areas of increased potential for yield are represented by sections of the tailgate adjacent to which the coalgate suffered high displacements, for

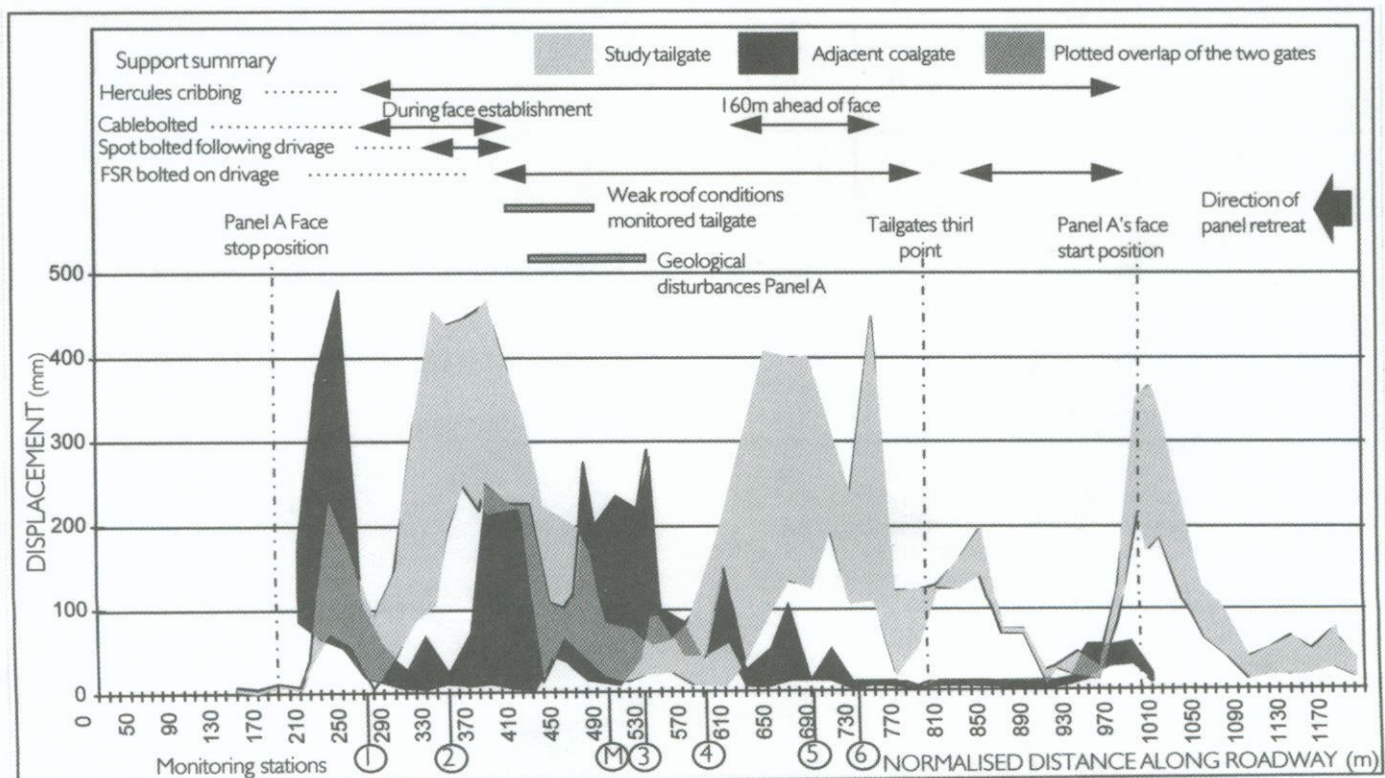


Figure 4. Comparison of telltale results from the study tailgate and adjacent panel coalgate with the tailgate's support summary. The shaded areas represents the displacement during retreat and the lower margins the total displacement on completion of drivage

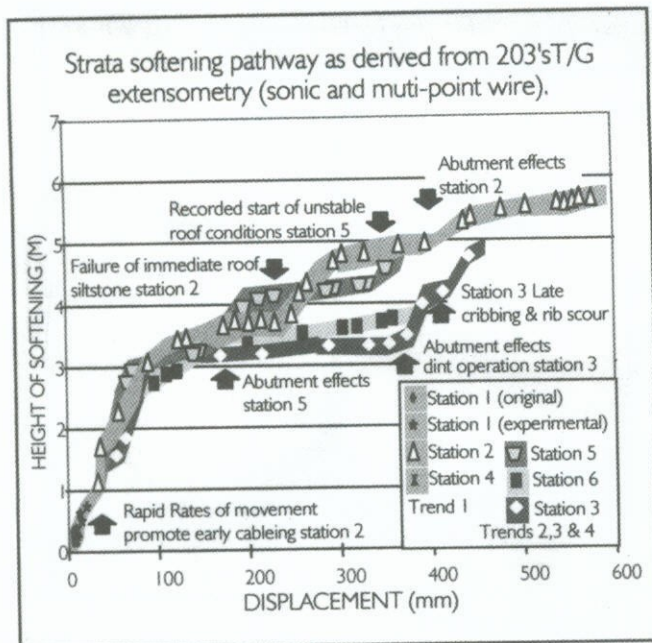


Figure 5 Strata softening pathway from the study tailgate.

whatever reason. The subsequent stress redistributions are likely to significantly influence the performance of the yielding-pillar and tailgate, this is reflected in the inverse relationship between the two gate road's roof displacements (Figure 4). For example at station 2, (low displacements in the coalgate) during drivage of the tailgate the height of softening extended high into the roof (>3.5 m) above the primary bolted horizon of 2.4 m, as illustrated in Figure 5. This early progression of softening high into the roof later promoted high total roof displacements during face retreat. Interpretation of the tailgate, and the yielding-pillar performance, should be considered as a combined-system since they obviously interact with each other.

Solid Ribside Behaviour

Solid ribside deformations in the tailgate were monitored at stations 2 and 3, these have been used to summarise its performance. Where the roof of the adjacent coalgate had suffered low displacements on drivage the extension of failure into the tailgate's solid ribside developed in a progressive manner from an initial state of low displacement and minimal depth of yield to the opposite. This behaviour was recorded at station 2, where on drivage the recorded movement at a depth of 5 m into the ribside was < 5 mm.

This contrasts with station 3 (540 m mark) where the coalgate suffered high roof movement on drivage, with a recorded 60 mm of displacement 6 m into the ribside on drivage, consistent with the solid ribside having yielded to a considerable depth. Subsequently on retreat the extension of the depth of failure was minimal, the main effects were seen in the immediate 1-1.5 m where deformation was already high. The high deformation state of the pillar at station 3 is further supported by the debonding of the stress overcore taken at the 510 m mark. Subsequent stress monitoring during retreat, see figure 3, indicated that the load carried by the solid ribside was reducing before a point of stabilisation 110 m ahead of the face suggestive of a significant yield zone.

Pillar Behaviour

The interaction between the deformation of the roof and solid ribside described above indicates the significant influence that the variable behaviour of the pillar and coalgate have on the stability of the tailgate along its length. The integrity of the tailgate's roof and its overall stability is controlled by the yielding-pillar, whose role is to yield whilst sustaining sufficient residual strength to avoid excessive deformation. From the monitored results it is possible to define two extreme behavioural modes in the response of the yielding-pillar, between which there are transitional states which describe the behaviour of the combined-system.

The two extremes (late stage and early stage yielding) are controlled by the pillar's ability to yield. Superimposed on this are complexities caused by variations in geology, structure, the strength of the individual strata and bedding interfaces and the pillar's loading history. Stations 2, 3 and 5 provide the data to assess the variable behaviour of the pillar. The roof and pillar deformations for both stations 2 and 5 are compared in figure 6.

Late stage yielding - Station 5

Characterised by very low displacements in the adjacent coalgate on drivage. This was mirrored by low stable roof displacements in the tailgate on drivage. The height of softening (shown in Figure 5), was constrained close to the bolted horizon of 4 m prior to effects of the face being imposed. Such behaviour maybe partially attributable to the installed FSR bolts which have a high system stiffness.

Station 5's pillar extensometer was lost 139 m ahead of the face, so the total abutment effects were not recorded. Results prior to this loss, indicated failure of the immediate ribside only, the pillar not having deformed significantly. Cablebolting was however required at station 5, approximately 160 m ahead of the faceline, because of sudden high telltale readings above the bolted height. Telltales were maintained throughout and recorded a further 50 mm of movement prior to face passage following cablebolt installation. This suggests the pillar did enter a high deformation mode, but very close to the faceline bringing sudden loading to the standing support and cribline.

Intermediate yield - Station 2

The differences between the observed behaviour of station 2 and 5 are likely to stem from two fundamental differences, they are:

1. No FSR was installed at station 2 on drivage
2. The area surrounding station 2 was stress relieved, particularly in the horizontal plane, and closely influenced by structural discontinuities.

The latter induced a rapid softening high into the roof on drivage requiring immediate remedial support with the use of 8 m double birdcaged cablebolts. This early strata deformation was then instrumental in promoting progressive deformation of the yielding-pillar, recorded by extensometer to a depth of >5 m. Subsequently the pillar stabilised in terms of deformation, with practically no further displacements on retreat between 139 m ahead of the face

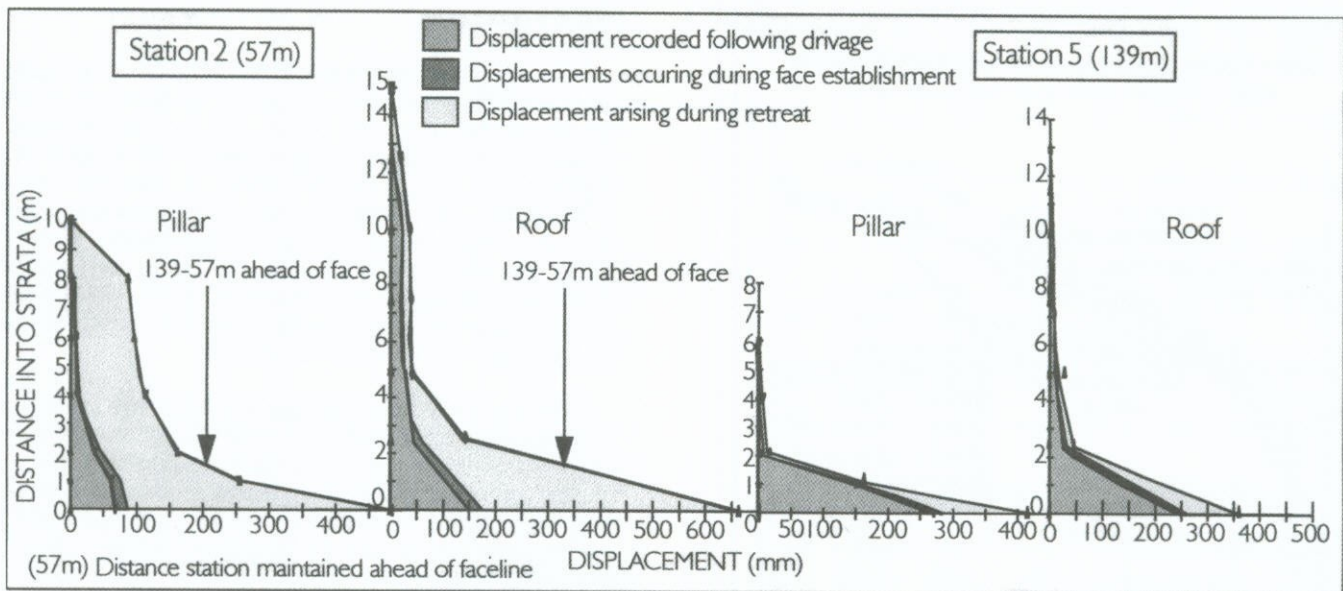


Figure 6. Comparison of yielding-pillar and roof deformation as recorded at stations 2 and 5.

and the final reading 57 m ahead. The stabilisation is reflected by elimination of further roof dilation, and is suggestive of pillar yielding accompanied by a favourable stress redistribution.

Early stage yielding Station 3

This is characterised by the adjacent coalgate having suffered high displacements, whilst on drivage the tailgate experienced good conditions (with low total roof displacements and a moderately constrained height of softening, see Figure 5). The ground was also disturbed structurally which promoted a larger yield zone width on drivage of the coalgate. Following the tailgate's drivage, most of the ground which was to make up the pillar had already undergone deformation and entered into yield, as suggested by the stress overcore. The relatively de-stressed conditions promoted a stable lower stress environment for the tailgate, reflected in low roof displacements. A further consequence was an increased yield zone depth into the solid ribside, which promoted high rib displacements. The stress recorded from the overcore site indicated the pillar was carrying a vertical stress approximately 4 MPa below cover load which is indicative of a yielded condition, however additional load was later carried on retreat (see figure 3).

Observations of the pillar indicated high levels of rib displacement at the 540 m mark, close to station 3. These high displacements necessitated a rib scour followed by replacement of 4 legs on the solid ribside, soon after drivage and long before any abutment effects were imposed. Roof behaviour had been very good on drivage and throughout retreat until this point when the station was 125 m ahead of the face. As the face effects were accentuated by both an extensive dint operation and late cribbing, bulking of the immediate 2.5 m of roof followed. From examination of Figure 5 these were closely associated with the final two stepwise increases in the height of softening, which then extended above the bolted height of 4 m. The indications are that the tailgate's stability was particularly sensitive to these operational mining activities.

Retreat Effects

The effects of retreat on the tailgate are illustrated by figures 5, 6 and 7. Figure 5 shows the effect not only of increased stresses associated with the face abutment but also the effect other mining operation's stresses associated with the face abutment had on the height of softening, (HOS), with particular emphasis on stations 2, 3 and 5. Indications are that at station 3 the combined effects of a large dint operation, late cribbing and a rib scour, promoted a rapidly rising trend in the HOS close to the faceline. These conditions were visibly identifiable in the immediate roof by significant localised bulking. The high deformation conditions ahead of the face at station 5 were of particular concern since the HOS had already extended above the bolted horizon.

Figure 6 compares the behaviour on retreat of the roof and pillar at stations 2 and 5, (intermediate and late yield). Station 2 shows high deformations at greater depth during retreat, followed by a later stabilisation. Station 5 exhibits displacements particularly in the pillar which were confined to the immediate 2 m of strata, without any indication of entering into a high deformational mode within the recorded period, but without any discernable stabilisation.

The abutment stresses influenced behaviour up to a position 300-350 m ahead of the faceline, Figure 7. The solid ribside's reaction contrasts to that of the roof and pillar since the abutment load is transferred to the solid, away from the pillar, Figure 3, and roof. Tending to confirm this there exists an apparent strong correlation between stabilisation of the pillar's deformation and stabilisation of roof displacement. The behaviour recorded both inbye and outbye in the tailgate on retreat with the 15 m pillar, differs notably with that recorded in panel A's tailgate. At no instance was there an event near the faceline which promoted a deformational response along the entire length of the roadway as occurred with the 10 m pillar on the previous panel. Station 2, Figure 7, was typical for all stations, the face abutment effects being clearly discernable as affecting deformation between 250 and 350 m ahead of the faceline.

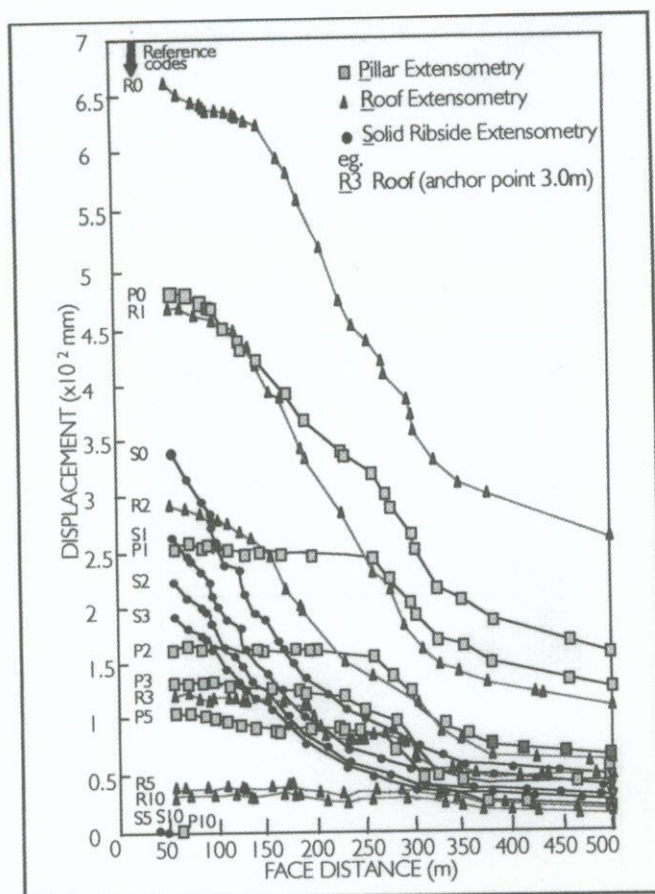


Figure 7. Comparison of face effects on the yielding-pillar, solid ribside and roof as recorded at station 2.

Variations in the pillar condition, along the length of the tailgate were noted over short distances. Conditions varied from a severely spalled and fractured rib, with cantilevered roof towards the pillar, to an intact pillar ribside with low displacement. The high frequency of these variations suggests that the relative condition of the coalgate cannot solely account for the resulting tailgate conditions, although they may exert a fundamental influence. When the tailgate was driven other mechanisms will have influenced pillar behaviour; these may include, for example, slip along bedding interfaces and variable bulking of the adjacent panel's goaf. In order to investigate factors which may influence pillar behaviour a numerical modelling approach was adopted, and is described below.

NUMERICAL MODELLING

The use of numerical modelling to analyse geomechanical problems is well established. Garratt (7) summarises the application of numerical modelling to reinforcement design in the UK coal mining industry. The approach taken here was to utilise modelling to promote the understanding of mechanisms associated with the observed deformation behaviour, rather than to make precise predictions.

Many of the current applications of yielding-pillars have been designed through applied practical experience, using identifiable characteristic responses recognised through monitored instrumentation, Hendon *et al.* (8). A similar

approach has been applied in the UK, but associated with it has been the cautious application of validated numerical models. Consideration of pillar sizing and performance along with issues related to the design of yielding-pillars in the UK have been examined, using numerical modelling by British Coal (9).

Model Generation

FLAC (10) was used to model the tailgate-pillar system. To simplify the model, lines of symmetry were used and a more idealised lithological section chosen. The model was expanded in later runs so that the whole of the pillar was modelled with both the tailgate and coalgate included. This allowed variations in the loading and boundary conditions which would mimic the true sequence of drivage and panel extraction on both sides of the pillar which was not possible with the original model.

Modelling of the tailgate-pillar system in its entirety is a large and complex problem, especially simulation of caving during longwall extraction. In order to limit the scope of the problem and concentrate on the behaviour of the pillar, the model grid was established with boundaries relatively close to the area of interest. The loading and boundary conditions were then varied to simulate the true sequence of drivage and panel extraction.

Material properties were derived from triaxial test data from the mine. Intact and residual strength failure envelopes were derived for the various rock samples. The ubiquitous constitutive model was used in conjunction with a 'FISH' routine allowing definition of the rocks strength envelope through piecewise linear approximations dependant on confining stress. In-situ stresses were used to define the initial pre-mining stress condition in the model.

Model Verification

Model output has been compared with monitoring results to ensure confidence in the generated model prior to undertaking any sensitivity or parametric analysis. This validation process ensures the failure modes, deformation and reinforcement characteristics being modelled correspond realistically to those measured. In this instance confidence in the applicability of the methods and strata properties used had been established in an earlier investigation of roof reinforcement at the mine. Figure 8 shows graphically the way in which the validation procedure was achieved. The measured strain gauged bolts provide a load/height profile which can be correlated with the loads generated in the modelled bolts. The maximum loads occur in the immediate 1 m of roof where shear along the weak bedding planes is greatest. From the two profiles there is a clear correlation in the modelled and measured behaviour.

A similar procedure was followed for both modelled and recorded roof displacements and stresses. The results suggest that the short and longer-term deformation and failure characteristics including HOS, following drivage, are accurately described in the pseudo-time behaviour of the model.

The results of numerical modelling have been used to

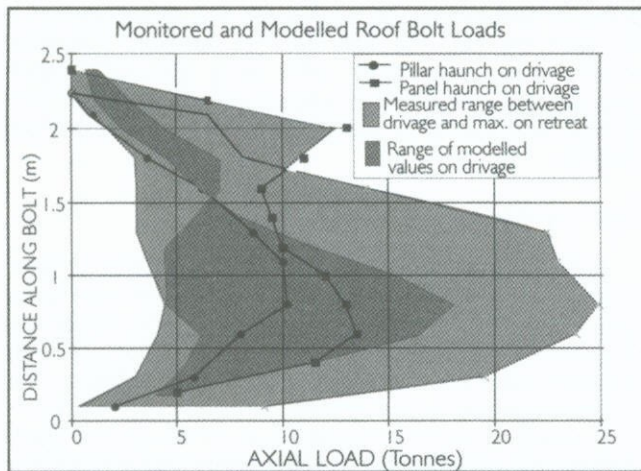


Figure 8. Model verification : Comparison of bolt loads.

provide further insight into potential mechanisms controlling the yielding-pillar performance. Modelling has been used therefore, to investigate factors such as pillar width, variation in immediate lithology, the location of bedding planes and differences in model generation and loading conditions. Some key aspects of the modelling are summarised below. Particular attention has been made to consider variations which could potentially alter the measured behaviour underground.

Different pillar widths have been modelled to assess their influence on potential for yield. The pillar widths examined included 10 m (previously used on an adjacent face), 15 m (the current size) and 20 m. Velocity loading allowed the model to examine the post failure behaviour of the different pillars, their relative residual strengths, and variable modes of failure.

Figure 9 shows the average vertical stress carried across the pillar and the post-failure stress maintained following peak pillar load. The 10 m pillar showed the lowest pillar strength, and in terms of numerical timesteps was the first to reach its peak strength and yield.

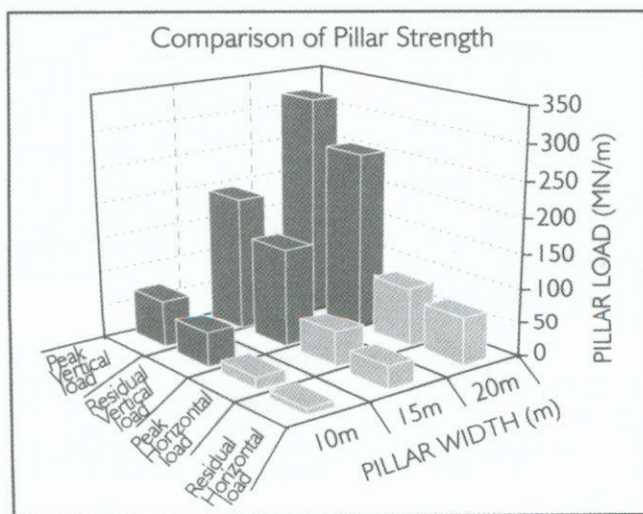


Figure 9 Modelled yielding-pillar peak and residual loads.

Shear strain contours were used to analyse the different deformational styles caused by changes in pillar width. Zones of shear extended from above the adjacent panel across the centre of the 10 m pillar to encroach close onto the tailgate, whereas the 20 m pillar showed signs of an intact pillar core, with increasing stress being carried following continued loading. Figure 10 shows the increased potential influence of goaf override over a 10 m pillar as opposed to the more stable 15 m pillar.

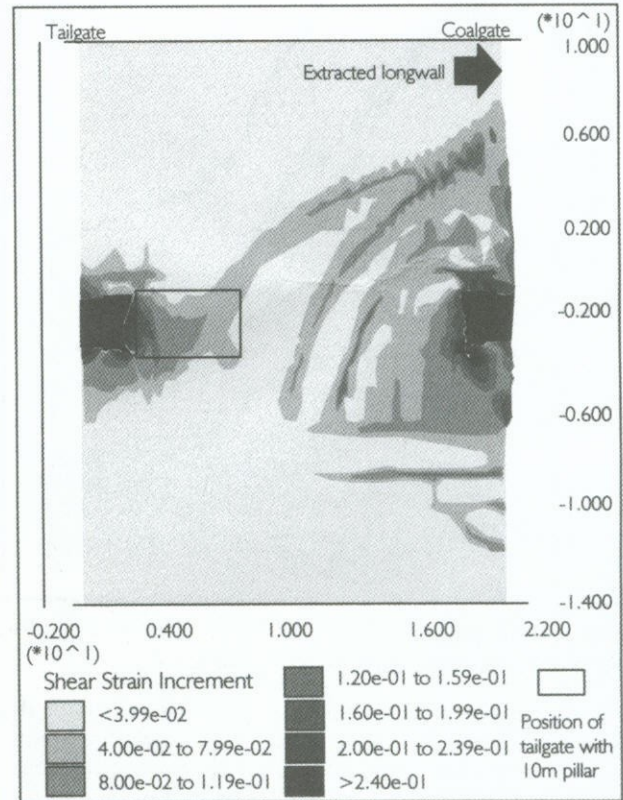


Figure 10 Modelled Shear Strain Increment following formation of a 15m yielding-pillar.

The model was established using interfaces between the individual bedded units. In order to assess the use of such artificial surfaces a model was run with and without the presence of FLAC interfaces separating the bedded units. Figure 11 shows the results which indicate the extensive shear surrounding the tailgate when the FLAC interfaces are not present. The orientation of these bedding planes which cross the preferential direction of shear serve to limit its extension into the pillar, the result of which is lower displacements and a reduced depth of yield.

In addition to sensitivity analyses on critical input parameters the effects of reinforcement and variation in boundary and loading condition were simulated. The effect of panel extraction, for example, was examined by simulating the possible range of boundary stresses. This included an increase in the vertical stress and a reduction in the horizontal restraints to represent the anticipated reductions in horizontal confinement as the roof of the extracted panel caved. To represent drivage of the roadways a 'FISH' routine was used to soften the ground prior to extraction, reducing the bulk

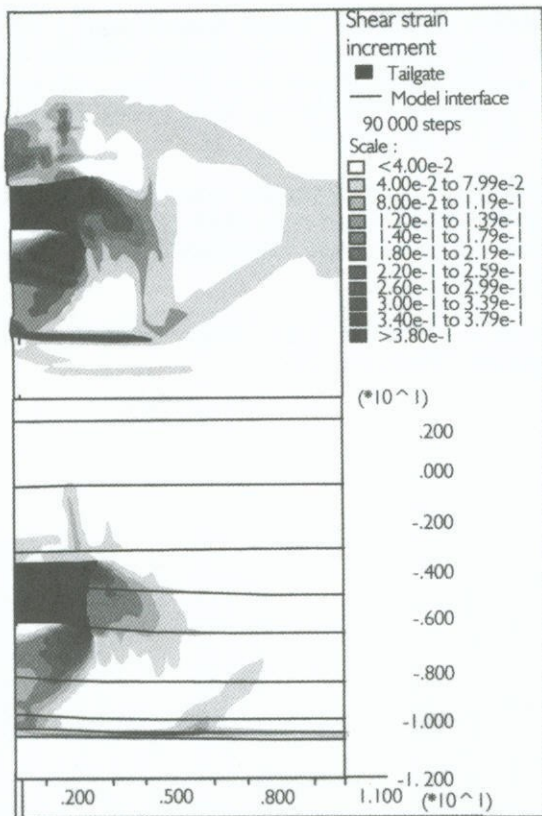


Figure 11. Modelled Shear Strain surrounding the tailgate with different model interface generation.

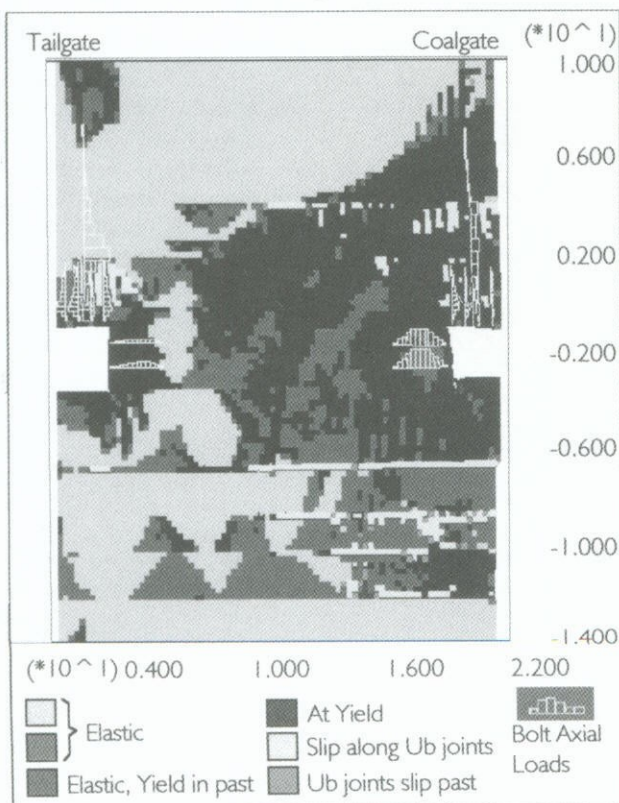


Figure 12. State plot showing the extent of failure within a 15 m pillar soon after drivage of the tailgate

and shear modulus and then sequentially reducing the stresses within those zones. Figure 12 illustrates behaviour similar to that recorded from station 2 (intermediate yield). It examines failure within a 15 m pillar soon after drivage of the tailgate, the indication is that the pillar has not yielded in its entirety at this stage, but did so following further loading.

The above examples show the application of modelling to assist in the interpretation of the recorded behaviour. Importantly validation of the model provides confidence in this particular application. Site specific modelling must, however, be considered in further applications in view of the complexity of any design situation. Used effectively modelling provides a powerful analytical tool, where complex behaviours can be simulated by making realistic assumptions.

CONCLUSIONS

Deformation and stress monitoring has provided valuable information to assess the performance of a yielding-pillar configuration. The immediate lithology, presence of structural features, previous deformation/stress history and extent of reinforcement all influence the performance and stability of the tailgate-pillar system. The influence of the first driven roadway has been shown to alter subsequent deformation and loading behaviour, in particular it significantly affects the timing and extend of yield within the pillar on its formation.

Given the conditions at this mine, the chosen panel layout and excavation sequencing it is considered that the 15 m pillar as opposed to the former 10 m pillars, offers greater stability, and from monitoring, provides more favourable tailgate conditions. Despite sections of the 15 m pillar deforming close to the retreating faceline, the excessive displacements experienced in tailgates using a 10 m yielding-pillar were avoided and events near the faceline which produced deteriorating conditions along the whole length of the roadway were not evident. Stress measurement and monitoring in such conditions proved important to assist understanding of the pillar's performance, to ensure its stability is maintained throughout retreat and help verify the results of numerical modelling.

The potential influence of a weak bedding plane and changes in the immediate lithology have been modelled. Both have been identified as possible mechanisms promoting softening and deformation within the pillar. Reduced bedding properties promoted a reduction in both the pillars vertical and horizontal stress retention leading to a reduction in the pillar's strength. The nature of the extraction sequencing adopted does result in difficult conditions in the support of the tailgate, ultimately however, the optimum pillar design and roadway conditions obtained are likely to be strongly influenced by the surrounding geology.

Numerical modelling has been used to gain an insight into the mechanisms affecting the strata deformation. Only through careful verification can such a numerical model provide a confident insight into these mechanisms. The user must be aware of the influence variation in model generation can have on the modelled strata behaviour, which this study has highlighted. The extent to which the pillar has yielded soon after its formation will depend significantly on the

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nature of the adjacent panels caving the loading history and the initial magnitude of horizontal stress. It is the variation of these within the model which has provided substantiation of the transitional states of pillar yield which have been described from analysis of the monitored results at this site. The selective nature of the modelled area illustrates the way in which complex geometries which would be difficult to understand with conventional analytical techniques, can be represented without requiring a complex modelling approach. The study has shown this can be achieved by careful sequencing and variation of the models boundary and loading conditions.

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Practical Stress Modeling for Mine Planning

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ABSTRACT

As part of the initial investigation and validation of a new boundary-element formulation for stress modeling in coal mines, the underground stresses and displacements at two multiple-seam coal mines with unique stress problems are modeled and predicted. The new program, LAMODEL, calculates stresses and displacements at the seam level and at requested locations in the overburden or at the surface. Both linear elastic and non-linear seam materials can be used; and surface effects, multiple seams, and multiple mining steps can be simulated. In order to most efficiently use LAMODEL for accurate stress prediction, the program is first calibrated to the site-specific geo-mechanics based on previously observed stress conditions at the mine. For this calibration process, a previously mined area is "stress mapped" by quantifying the observed pillar and strata behavior using a numerical rating system. Then, the site-specific mechanical properties in the model are adjusted to provide the best correlation between the predicted stresses and the observed underground stress rating. Once calibrated, the model is then used to predict future stress problems ahead of mining. At the two case study mines, the calibrated models showed good correlation with the observed stresses, and also accurately predicted upcoming high stress areas for preventive action by the mines.

INTRODUCTION

Mine planners have a variety of modeling methods, both empirical and numerical, available for analyzing pillar stresses and determining safe pillar sizes for various mine geometries and geologic structures. Empirical methods emphasize the collection and interpretation of case histories of pillar performance. ALPS and ARMPS are two such empirical programs which are derived from large databases of real-world pillar studies and which can be used for determining pillar sizes for single-seam longwall and retreat room-and-pillar mining, respectively (1, 2). The Virginia Polytechnic Institute and State University recently developed a comparable empirical program called MSAP for sizing pillars for

multiple-seam situations (3). These empirical programs are closely linked to reality and very user-friendly, and for many "typical" mining geometries, they work extremely well.

However, it is difficult to apply these empirical programs to mining situations beyond the scope of the original empirical data base. Therefore, when complicated stress conditions arise from complex single or multiple-seam mining geometries, numerical modeling techniques such as finite-element, boundary-element, discrete-element, or finite-difference are usually applied. In general, these numerical, or analytical, design methods are derived from the fundamental laws of force, stress and elasticity. Their primary advantage is that they are very flexible and can quickly analyze the effect of numerous geometric and geologic variables on mine design. Their primary disadvantage is that they require difficult-to-obtain, and/or controversial, information about material properties, failure criteria, and post-failure mechanics. In the following paper, the solid foundation of empirical pillar design and in-mine observation is combined with the flexibility of numerical modeling to provide a practical technique for mine planning in difficult situations.

LAMODEL

In order to analyze the displacements and stresses associated with the extraction of large tabular deposits such as coal, potash and other thin vein-type deposits, the displacement-discontinuity (DD) variation of the boundary-element technique is frequently the method of choice. In the displacement-discontinuity approach, the mining horizon is treated mathematically as a discontinuity in the displacement of the surrounding media. Using this technique, only the planar area of the seam needs to be discretized, or gridded, in order to obtain the stress and displacement solution on the seam. Often, this limited analysis is sufficient, since in many applications only the distributions of stress and convergence on the seam horizon are of interest. Also, by limiting the detailed analysis to only the seam, the DD method provides considerable computational savings over other techniques which discretize the entire

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body (such as finite-element, discrete-element or finite-difference). It is a direct result of this computational efficiency that the DD method is able to handle large areas of tabular excavations as needed in many practical coal mining problems.

A displacement-discontinuity program incorporating a laminated media has recently been developed by NIOSH. This new program is called LAMODEL. Traditional displacement-discontinuity programs use a homogeneous isotropic elastic formulation which simulates the overburden as one solid material. In contrast, the LAMODEL program simulates the geologic overburden stratifications as a stack of layers with frictionless interfaces. Specifically, each layer is homogeneous isotropic elastic and has the identical elastic modulus, Poisson's Ratio, and thickness. This "homogeneous layering" formulation does not require specifying the material properties for each individual layer and yet it still provides a realistic suppleness to the mining overburden that is not possible with the classic, homogeneous, isotropic, elastic overburden model. From our experience, this suppleness provides a more accurate strata response for modeling local deformations, inter-seam interactions and/or surface subsidence. The LAMODEL program calculates stresses and displacements at the seam level and at requested locations in the overburden or at the surface. Both linear elastic and non-linear seam materials can be used. The program also has the ability to analyze: 1) the interseam stresses resulting from multiple-seam mining; 2) the effects of topographic relief on pillar stress and gob loading; 3) the stress changes during mining through multiple mining steps, and 4) the surface subsidence.

INITIAL MATERIAL PROPERTY GENERATION

As mentioned earlier, one of the most difficult parts of using a numerical model is determining the correct (most accurate) material properties for input. After developing numerous D.D. models and then comparing their results with field measurements and observations, a fairly streamlined, systematic technique for developing initial material properties has been developed. Initially, the critical material properties (coal, gob, and rock mass) are determined using a combination of laboratory research, empirical formulas and experience. Then, in the calibration process, these initial material properties are systematically adjusted in subsequent runs of the model until the results correspond as closely as possible to field observations. This technique for determining material properties has many similarities to the procedure used by Karabin and Evanto (4).

First, to address the problem of determining the input coal behavior, the basic coal strengths are derived from the empirical pillar strength formulas which are solidly based on observed pillar behavior. Specifically, the peak strength of a model coal element is directly determined based on an in situ coal strength and its distance from the edge of the pillar (5) using the stress gradient implied by the Bieniawski pillar-strength formula (2). This peak strength is then implemented using an elastic, perfectly-plastic material model (6). For an initial estimate, an in situ coal strength of 6.2 MPa (900 psi) (7) and an elastic modulus of 2 GPa (300,000 psi) is typically used.

This general procedure for generating the initial coal properties for elements in LAMODEL fulfills a number of practical requirements. It provides LAMODEL pillars with peak strengths which closely follow the empirically proven Mark-Bieniawski pillar strength formula and stress profiles which closely follow the Bieniawski stress profile. As opposed to a simple elastic material model with no load limit, this procedure using elastic-plastic material allows the pillars to reach a maximum load carrying capacity and then realistically shed additional load to surrounding areas. Table 1 gives typical elastic-plastic material input numbers for 3 m (10 ft) coal elements in a 1.8 m (6 ft) seam with a 6.2 MPa (900 psi) in situ coal strength. (Note that the peak stress for the coal elements decreases from the core to the rib of the pillar, which give the pillar the proper stress profile.)

Table 1
Typical elastic-plastic coal and strain-hardening gob parameters

COAL ELEMENTS		
ELEMENT	PEAK STRESS (MPa)	PEAK STRAIN
A (CORE)	85.9	0.04152
B	56.1	0.02712
C	38.3	0.01992
D (RIB)	11.4	0.00552
GOB ELEMENTS		
INITIAL MODULUS (MPa)	FINAL MODULUS (MPa)	FINAL STRESS (MPa)
6.2	138	27.6

Next, to address the gob loading and compaction behavior, a combination of laboratory research and modeling experience is used. In the laboratory, Pappas and Mark (8) found that an exponentially strain-hardening material with a tangent modulus that increases linearly with stress provided a reasonable representation of simulated gob material. This material model is implemented in LAMODEL (5), and is used for the gob modeling. The necessary input for this material is; initial modulus, final modulus, and final vertical stress. From experience, these three values are initially set at 6.2 MPa (900 psi), 110 MPa (16,000 psi) and 27.6 MPa (4,000 psi), respectively (see Table 1).

The third critical set of material inputs in LAMODEL is for the overburden and consists of a lamination thickness and an elastic modulus. In LAMODEL, the lamination thickness has a major influence on the stress and displacement distribution at the seam and throughout the overburden. Prior research (5) comparing LAMODEL results with empirical relationships and measured field data shows that for large-scale stress distributions (such as longwall abutments) lamination thicknesses ranging from 15 to 100 m (50 to 300 ft) provide the best match to field measurements. However, when small-scale stress distributions (such as interseam stresses) or overburden displacements (such as subsidence) are of primary concern, then lamination thicknesses ranging from 3 to 15 m (10 to 50 ft) provide the best match to field observations (4, 8). In this paper, a lamination thickness of 15 m (50 ft) was used for case study 1 and a thickness of 5 m (15 ft) was used for case study 2. In both case studies, an elastic modulus of 20 GPa (3,000,000 psi) was used for the overburden.

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STRESS MAPPING

In order to optimally use LAMODEL for accurate stress prediction at a given mine, the program should first be calibrated to the site-specific geo-mechanics based on previously observed stress conditions at that mine. One of the simplest and easiest methods to "quantify" the stress at a particular mine is to use "stress mapping". The pillar-centric stress mapping technique used in this paper to quantify the observed stress conditions is a slight modification of the stress mapping technique originally developed for mapping areas of high horizontal stress (10). For LAMODEL calibration in this paper, the primary interest is the stress in the pillars; therefore, the primary stress indicator is the pillar rib damage, although other stress related features such as roof cracks or floor heave are also noted during the stress mapping process since they can be useful indicators of stress reactions.

Stress mapping an area of the mine essentially consists of traveling the rooms and crosscuts in that area and carefully observing the conditions of the pillars, roof and floor. The observed conditions are assigned a numerical rating and marked on a map. For the rib damage stress mapping used in this paper, the following rating criteria were applied:

- 0 - Rib still intact with no sloughed coal, original rock dust still in place.
- 1 - Very slight pillar sloughage, some broken coal at base of rib.
- 2 - Slight pillar sloughage, broken coal covers one third of rib.
- 3 - Significant pillar sloughage, broken coal piled half way up rib.
- 4 - Severe pillar sloughage, broken coal piled almost to roof.
- 5 - Rib is composed of completely broken coal at the angle of repose, pillar maybe failed.

MODEL CALIBRATION

In the model calibration process, the initial material properties are systematically adjusted in subsequent runs of the model until the results correspond as closely as possible to field observations and/or empirical formulas. For the coal properties, the in situ coal strength is adjusted until the pillar stress/failure in the model matches the observed pillar behavior as represented by the stress mapping/rib rating. For the gob properties, the final modulus value is typically adjusted up or down in LAMODEL in order to increase or decrease the gob stress until the model gob stress matches empirical abutment angle formulas (2) and/or field measurements and observations. For the overburden properties, the lamination thickness is typically adjusted up to provide wider abutment stresses and smaller interseam stresses or down to provide narrower abutment stresses and greater interseam stresses as dictated by the observed stress mapping.

Once the model is reasonably calibrated and realistic pillar strengths and load distributions have been established, the mechanics-based overburden behavior in the LAMODEL program can be effectively used to accurately analyze the complicated stresses and displacements associated with future complex mining

scenarios. The above technique of combining empirical pillar strength and abutment load formulas with in mine stress mapping and the analytical mechanics of a displacement-discontinuity model capitalizes on the strengths of both the empirical and analytical approaches to pillar design. The empirical formulas and observational calibration base the model on realistic behavior while the analytical mechanics allow the model to accurately consider and analyze the effects of numerous geometric and geologic variables. Using this technique, a displacement-discontinuity model can be the most practical approach for stress analysis and pillar design in complex mining situations such as: multiple seams, random pillar layouts and/or variable topography.

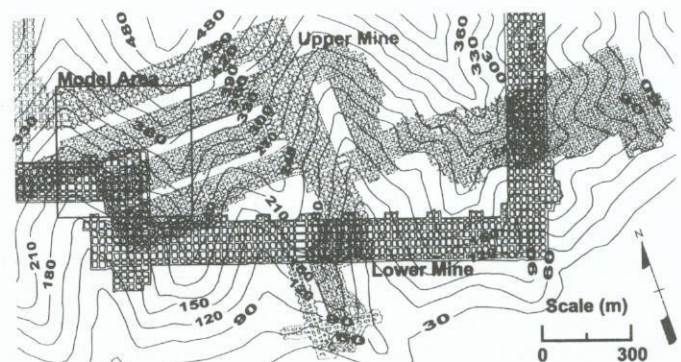


Figure 1. Map of mine for case study 1.

CASE STUDY 1

The first case study location is a multiple-seam, room-and-pillar, coal mining situation in Eastern Kentucky. At this location, the lower mine had been adversely affected by mining in the upper seam (see Figure 1). In particular, the lower mine experienced serious ground control problems when it mined under a barrier pillar between two upper seam gobs (model area in Figure 1). At this multiple-seam interaction site, in-mine stress mapping was used to quantify the severity of the multi-seam interactions. This stress mapping was also used to calibrate a LAMODEL simulation of the area. The results of this numerical simulation provided predicted stress levels to avoid in future multiple-seam or high-cover mining.

The geology at this location is fairly typical of the Southern Appalachian coal basin with various sedimentary layers of sandstones, siltstones, shales and numerous coal seams. The topography in the area is very rugged with various steep ridges and valleys that have a topographic relief of over 600 m (2000 ft) (see Figure 1). The overburden in the study area ranged from 150 to 450 m, (500 to 1500 ft) with an average around 300 m (1000 ft). Because of the highly variable topography at this mine, it was critical to include the topographic stress effects in LAMODEL in order to obtain accurate results.

The overlying, or upper, mine operates in the Upper Darby seam which typically averages about 2.0 m (6.0 ft) thick. The lower mine operates in the Kellioka seam which averages about

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1.5 m (4.5 ft) thick in the study areas. The interburden between the two seams averages around 14 m (45 ft) and consists of interbedded sandstones and shales. The nearest core logs to the study site indicate some 3.5 to 5 m (10 to 15 ft) of shale directly over the Kellioka seam. This is then overlain by 7.5 to 10.5 m (25 to 35 ft) of interbedded sandstones and shales, with shale primarily forming the floor of the Upper Darby seam. Both mines are room-and-pillar drift mines and utilize continuous miners for coal extraction. In some production sections, depending on local mining conditions, the mines remove the pillars on retreat for full extraction.

In the study area, the lower mine was forced to dogleg around an abandoned, flooded mine in the upper seam (not shown in Figure 1). This dogleg forced the lower mine to develop entries underneath a barrier pillar between two previously mined, upper seam gobs as shown in the detail of Figure 2. Mine management anticipated increased multiple-seam stresses in this area. In an effort to safely control these higher stress levels, the mine located the critical travel way and belt entries away from the influence of the barrier pillar and used a double row of supplemental cable bolts on 0.6 m (2 ft) centers throughout most of the travel entry under the upper seam mining. With the precautions taken, the mine was able to safely and efficiently mine the entries under the barrier pillar and surrounding gob. However, throughout the section, the stress effects of the overlying barrier and gob were abundantly visible, and on two occasions (in the northeast corner of the section), the mine was unable to complete crosscuts because of roof instability and poor pillar conditions.

Stress Mapping

In order to quantify the stress effects of the barrier pillar and gob zones on the lower seam, a detailed stress mapping of a large portion of over-mined area was performed. As previously described, the amount of rib sloughing was noted on a scale of 0 to 5, and any stress related features such as roof cracking, potting, cutting, or floor heave were also noted. The results of this stress mapping exercise are shown in Figure 3A. In this figure, the observed condition of the pillar ribs is shown in grey scale by degree of damage with the darker shades signifying increased sloughing (or stress). Also, the observed roof cracking, potting, cutting, and the floor heave is marked on the map.

There are a couple of useful observations that can be made from the detailed stress mapping shown in Figure 3A. First, the transfer of the abutment stresses from the overlying mine to the area under the barrier pillar and to the area at the ends of the pillared sections can clearly be inferred in the rib conditions of the

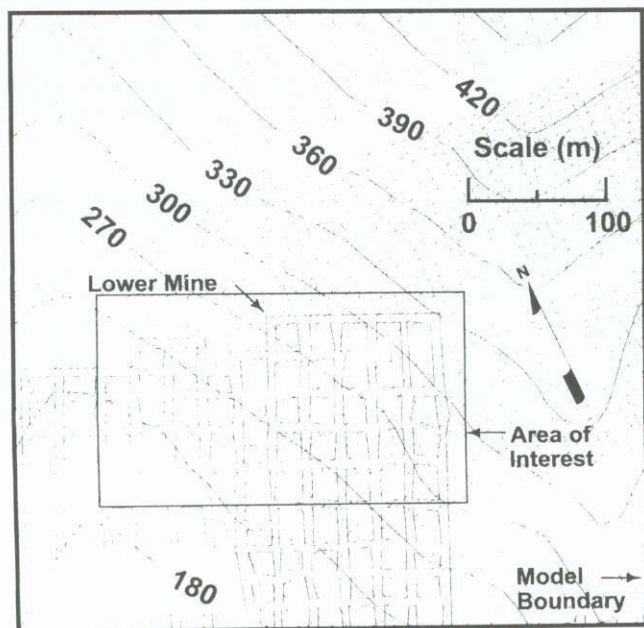


Figure 2. Enlargement of model area for case study 1.

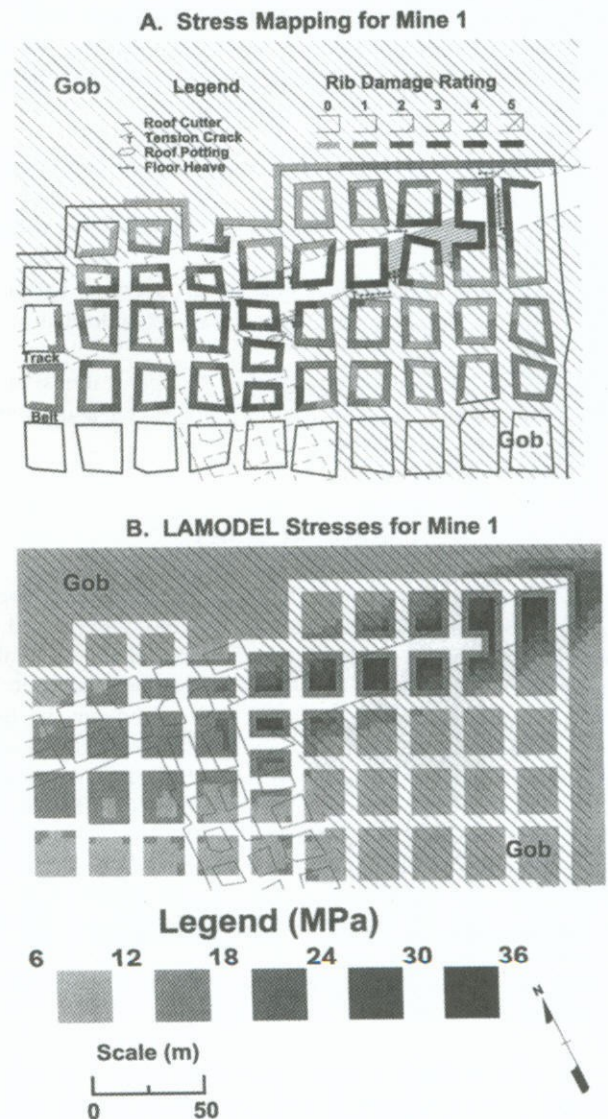


Figure 3. A comparison between in-mine stress mapping and LAMODEL calculated stresses.

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lower mine pillars. Also, as a corollary to the interseam transfer of the barrier pillar abutment stresses, the lower seam pillars under the gob areas in the upper seam show considerable stress relief. The next major observation concerns the location and orientation of the roof tension cracks and guttering. Clearly, the tension cracks in the roof of the northeast corner of the section are situated directly under the overlying barrier pillar and are oriented parallel to the axis of this pillar. Also, the observed compressional roof cutters are located at the edge of, or adjacent to, the overlying abutment zones and oriented parallel to these zones. This location and orientation of the tension and compression suggests that the lower mine roof is behaving like a beam which is bending into the relatively soft coal seam under the load of the barrier pillar in the upper seam. This beam scenario correctly accounts for the tension directly under the applied load and the compression adjacent to the applied load.

Model Design

For the LAMODEL simulation of this area, the seams were discretized with 3 m (10 ft) elements in a 150 by 150 grid with the model boundary as shown in Figure 2. Symmetrical seam boundary conditions were set on all four sides and no free-surface effects were included. The interburden was set at 14 m (45 ft) and the rock mass was simulated with a modulus of 20 GPa (3,000,000 psi) and 15 m (50 ft) thick laminations. An elastic, perfectly-plastic material was used for the coal in both seams, and the peak strength of the coal was determined from the Mark-Bieniawski pillar strength formula as in Appendix C of Heasley (1998). Table 2 gives the coal and gob input values used in LAMODEL for this particular case study.

Table 2

Coal and gob parameters for case study 1.

COAL ELEMENTS - UPPER MINE		
ELEMENT	PEAK STRESS (MPa)	PEAK STRAIN
A (CORE)	85.9	0.04152
B	56.1	0.02712
C	38.3	0.01992
D (RIB)	11.4	0.00552
COAL ELEMENTS - LOWER MINE		
ELEMENT	PEAK STRESS (MPa)	PEAK STRAIN
A (CORE)	113.2	0.05472
B	73.5	0.03552
C	53.6	0.02592
D (RIB)	13.9	0.00672
GOB ELEMENTS		
INITIAL MODULUS (MPa)	FINAL MODULUS (MPa)	FINAL STRESS (MPa)
6.2	110	27.6

Also, because of the high topographic relief at the site, the topography was discretized with 15 m (50 ft) elements for an area extending 300 m (1000 ft) beyond the limits of the displacement-discontinuity grids. The importance of including the topographic stress effects in the model is clearly evident in Figure 4 which

shows the topographic stress at the level of the lower mine. It is interesting to note in this figure the amount to which the topographic stress is "smoothed" with depth as compared to the original topography. Also, it is evident that the overburden stress changes some 3 MPa (450 psi) in traversing from the southwest to the northeast corner of the pillars in the study area. This difference in overburden stress could very well account for the increased mining difficulties at the northeast corner of the section.

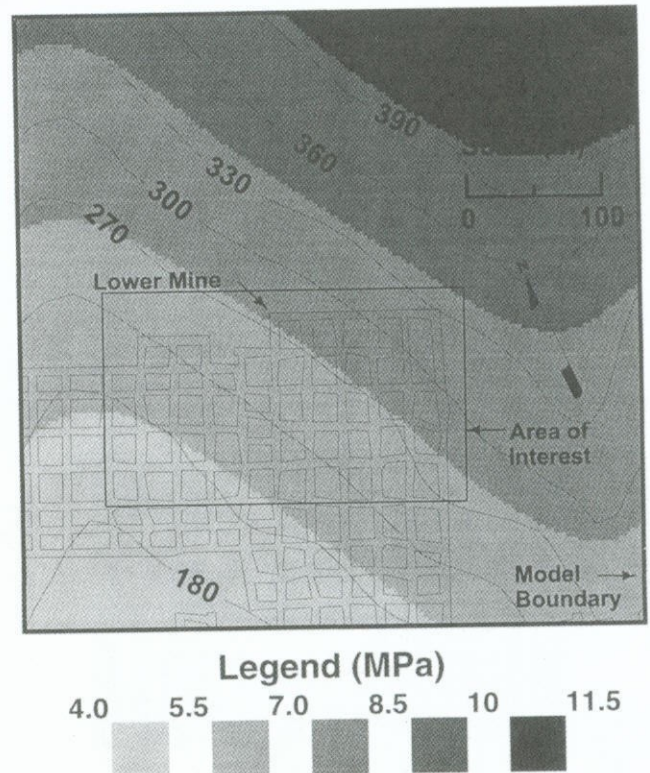


Figure 4. Calculated topographic stress for case study 1.

Model Calibration and Analysis

Very little work was required for calibrating the LAMODEL simulation to the observed stress mapping. In both seams, the original Mark-Bieniawski pillars strengths and the initial overburden modulus and lamination thickness provided a good fit to the observed pillar behavior (see Figure 3). The only parameter that was ultimately manipulated was the modulus of the gob material (see Table 2). This modulus was adjusted to provide a peak gob stress in the range of 40 to 60 percent of in situ stress, a reasonable range for a 90 m (300 ft) wide gob in 300 m (1000 ft) of cover (2). A number of variations in pillar strength, overburden modulus and lamination thickness were investigated, and the simulation results varied a little. However, the initial parameter values with the adjusted gob modulus provided a reasonably optimum fit to the observational stress mapping.

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The calculated pillar stresses from the final calibrated LAMODEL run are shown in Figure 3B. These modeled stresses correlate extremely well with the stress mapping in Figure 3A. The high stresses under the barrier pillar are clearly evident in the model results, and the area of stress relief under the gob is also shown. Even the intermediate stress levels under the overlying pillars and solid coal in the southwest corner of the model closely match the observed pillar stress mapping. A few more details of the modeled stress output are shown in Figure 5 where the isolated single-seam stress and just the interseam stress are displayed. In this figure, the effect of the overlying barrier pillar can be clearly seen. In

particular, the maximum single-seam stress on the pillars (see Figure 5A) of around 15 MPa (2200 psi) is seen to increase to over 36 MPa (5200 psi) with the addition of the barrier pillar stress (see Figures 5B and 5C). Also, it is interesting to note the increased abutment stress in the northeast corner of the section (see Figure 5C), presumably due to the increasing overburden and the increasing distance from the upper panel boundaries. A stress relief of some 7 MPa (1000 psi) under the gob areas is also shown in Figure 5C.

For the mine management, this stress modeling using LAMODEL in conjunction with good in-seam correlations with stress mapping provided valuable background information for future multiple-seam mine planning. In this case study, a calculated multi-seam stress concentration of around 15 MPa (2200 psi) with pillar stresses of 35 MPa (5200 psi) at this site caused sufficient roof instability to prohibit the mine from driving a couple of desired entries. Therefore, it appears that the 15 MPa stress concentration (35 MPa pillar stress) is close to an upper limit for successfully entry development at this mine. The mine can use this calculated limit in conjunction with future modeling in order to layout future room-and-pillar panels influenced by overlying workings.

CASE STUDY 2

The second study site is a longwall mine located in Greene County, Pa. and operating in the Sewickley seam. This mine is underlain by an abandoned room-and-pillar operation in the Pittsburgh seam. The primary problem at this site was the transfer of multiple-seam stress from the lower mine. Yielding of smaller pillars and the subsequent transfer of their load to larger pillars in the lower seam apparently caused increases in vertical stress in the upper seam which were noticed during development of the headgate entries (see Figure 6). Severe pillar spalling and poor roof conditions were experienced when mining the headgate over these large pillars in the lower seam (Figure 7). Mine management was concerned these underlying abutment pillar stresses would continue to be a problem further inby in the headgate and also in longwall panel because there were several areas in the lower seam where similar pillar conditions appeared to exist.

In the study area, the overburden above the Sewickley seam ranges from 150 to 280 m (500 to 910 ft) and consists predominately of interbedded shales and sandstones. The interburden between the Sewickley and Pittsburgh seams ranges from 27 to 30 m (90 to 100 ft) thick and consists of interbedded shales and limestones. The average mining heights of the Sewickley and Pittsburgh seams are 1.5 m (5 ft) and 1.8 m (6 ft), respectively. The immediate roof of the Sewickley seam is composed of a jointed dark sandy shale that ranges from 3 to 4.5 m (10 to 15 ft) thick overlain by a competent limey shale. The immediate floor of the Sewickley seam is composed of a 1.2 m (4 ft) thick dark, limey shale underlain by a competent limestone unit.

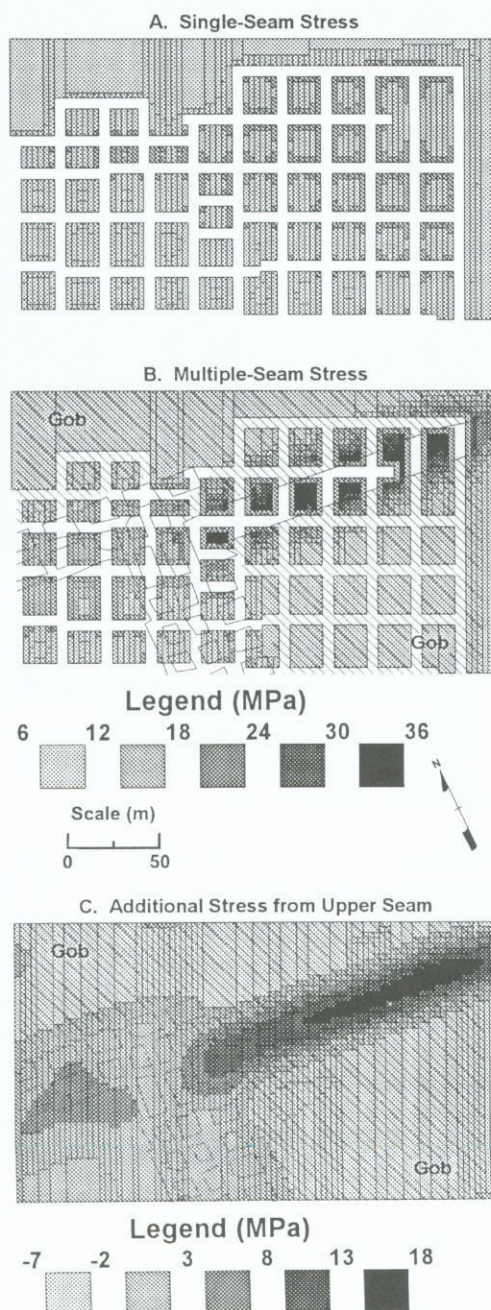


Figure 5. The LAMODEL stress output for case study 1.

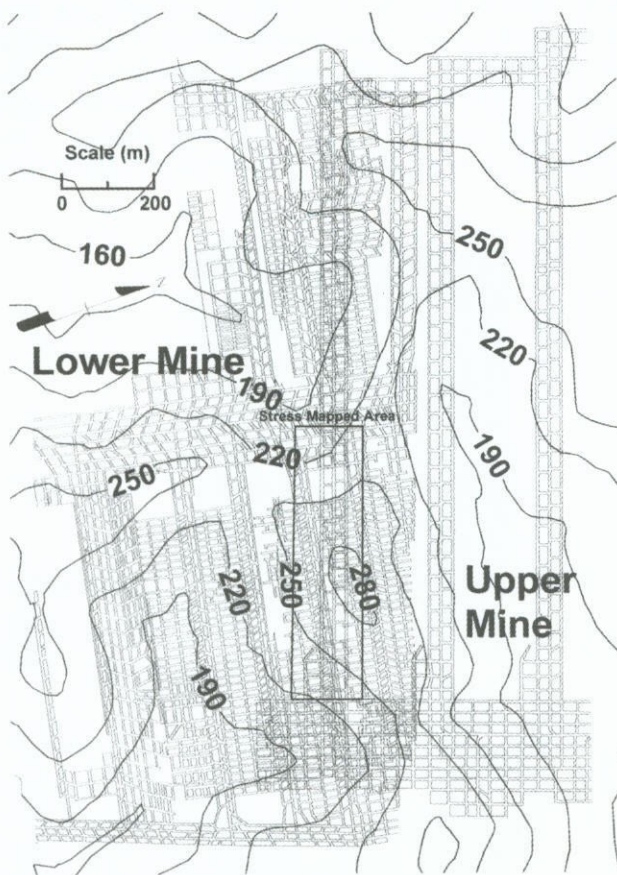


Figure 6. Map of mine for case study 2.

Stress Mapping

Figure 6 shows the overlay of the lower seam workings on the upper seam longwall panel and the area of the headgate where the stress mapping and model calibration were conducted. As described earlier, the process of calibration involved the use of stress mapping to assign a rating number from 0 to 5 based on the observed pillar rib conditions. The first 600 m (2000 ft) of the headgate entries, where problems first occurred, (see Figure 6), were traversed and assigned rating numbers based on the observed conditions. Figure 7A shows the rib damage rating assigned to each rib in this area of the headgate.

Model Design and Calibration

Once the stress mapping was complete, LAMODEL calibration was initiated. For calibration purposes, the area shown in Figure 6 was discretized with 3 m (10 ft) elements with a 90 by 200 grid. Symmetrical boundary conditions were set on all four sides and no free-surface effects were included. The interburden was set at 27 m (90 ft) and the rock mass was simulated with a modulus of 20 GPa (3,000,000 psi) and 5 m (15 ft) thick laminations. The overburden above the lower mine in this area ranged from 180 to 300 m (600 to 1000 ft). Due to this variable topography, the topographic stress effects were included in LAMODEL in order to obtain accurate overburden stress results.

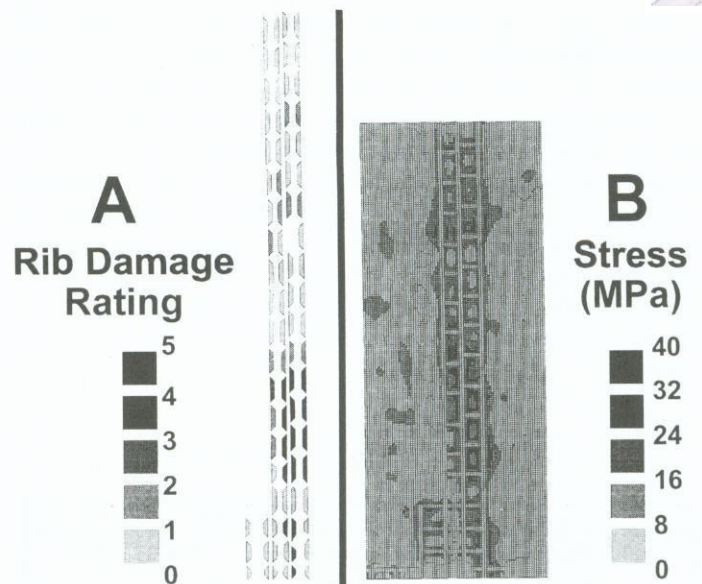


Figure 7. A comparison between in-mine stress mapping and LAMODEL calculated stresses.

Based on the observed stress mapping, model calibration was conducted under the assumption that the smaller pillars (< 10.5 m (35 ft) wide) in the lower mine had essentially yielded and transferred their load to nearby larger pillars. Therefore, in the first step of the calibration process, the coal strength in the lower mine model was adjusted until the pillars showed this observed behavior. Initially, using the elastic-plastic implementation of the Bieniawski formula, as previously explained, an in situ coal strength of 6.2 MPa (900 psi) was used to calculate peak stress and strain values for each coal element and the initial calibration model was run. In this initial model, the coal in the lower mine was too strong and did not show the desired yielding in the smaller pillars. Therefore, in order to obtain the desired small pillar yielding and subsequent stress transfer to the larger pillars, the in situ coal strength in the lower seam was gradually decreased to 4.2 MPa (600 psi).

With the in situ coal strength of 4.2 MPa (600 psi) in the lower seam and the original coal strength of 6.2 MPa (900 psi) in the upper seam, the model correlated very well with the rib damage rating from the stress mapping (see Figure 7). As seen in Figure 7, the rib damage rating is in grey scale on side A and the results from the model are in a comparative grey scale plot on side B. Clearly, the model pillars with high rib stress correlate well with the pillars with high damage ratings. Looking back at Figure 6, it can be observed that these high rib stresses occur over the large pillars located in the lower mine, in conjunction with overburden that exceeds 250 m (870 ft). The final coal and gob properties used in LAMODEL for the upper and the lower mine are given in Table 3.

Stress Prediction for Mine Planning

With material properties calibrated from observed stress conditions in the mine, additional LAMODEL analyses were created and run in order to predict areas of potential problems within the remaining headgate and the future longwall panel.

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Figure 8 shows two areas of the headgate and longwall panel that were modeled using optimized properties from the calibration process. These grey scale plots show the interseam stress, which is the additional stress on the upper mine due to the lower seam mining. In this figure, Zone 1 covers the upper (inby) part of the headgate panel and the first 365 m (1200 ft) of the longwall panel, and Zone 2 covers the lower part of the headgate (where the stress problems were first noticed) and the last (outby) 330 m (1100 ft) of the longwall panel. In these two zones, the lower mine pillar conditions and the overburden depths appeared similar; therefore, the poor pillar conditions encountered in Zone 2 were expected in Zone 1.

Table 3
Coal and gob parameters for case study 2.

COAL ELEMENTS - UPPER MINE		
ELEMENT	PEAK STRESS (MPa)	PEAK STRAIN
A (CORE)	102.3	0.04944
B	66.5	0.03216
C	48.7	0.02352
D (RIB)	12.9	0.00624
COAL ELEMENTS - LOWER MINE		
ELEMENT	PEAK STRESS (MPa)	PEAK STRAIN
A (CORE)	56.8	0.02747
B	36.9	0.01787
C	27.0	0.01307
D (RIB)	7.2	0.00347
GOB ELEMENTS		
INITIAL MODULUS (MPa)	FINAL MODULUS (MPa)	FINAL STRESS (MPa)
6.2	138	27.6

However, when comparing the interseam between these two zones as shown in Figure 8, it is obvious that the stress is considerably greater in Zone 2 than in Zone 1. Closer investigation reveals two primary reasons for this difference. First, the maximum depth over the gateroads and panel in Zone 2 is over 280 m (920 ft), whereas in Zone 1, the maximum depth is just over 250 m (870 ft). Second, when examining the model output for the lower mine, there appears to be less pillar yielding in Zone 1 than in Zone 2. Examining Figure 6, it can be seen that the smaller pillars in Zone 1 are disperse among larger pillars and have widths greater than 12 m (40 ft), in comparison to Zone 2, where there is a large area of pillars with widths less than 10.5 m (35 ft). The larger, more dispersed small pillars in Zone 1 suffer less pillar yielding, and therefore, cause less load transfer (or interseam stress) on the upper mine (see Figure 8). During headgate development in Zone 1, no pillar problems were encountered. Thus, the calibrated model successfully predicted the reduced stress conditions in the headgate of Zone 1.

The mine management was also concerned about the multiple-seam stresses adversely affecting the retreating longwall panel. In particular, a large, irregular shaped barrier pillar in the lower mine

is superimposed under the center line of the initial half of the longwall panel in Zone 1 (see Figure 8). However, the interseam stress calculated by the model from this barrier pillar only reaches about 3 MPa (450 psi). When the panel was mined, this slightly increased face stress and presented very little problem. Some slight spalling was present on the face during the extraction, but

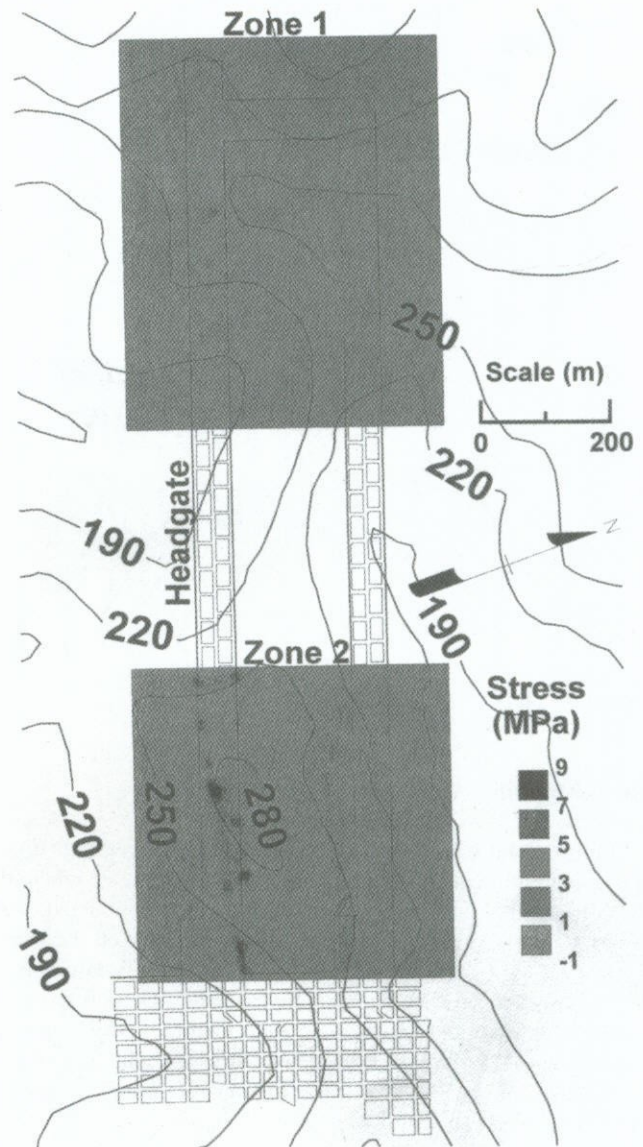


Figure 8. Interseam stress for Zone 1 and Zone 2.

overall face conditions were generally good and no severe ground control problems were evident.

However, in the lower part of the panel near the headgate location where poor ground conditions were first encountered (see Zone 2, Figure 8), an area of interseam stress up to 9 MPa (1300 psi) is evident in the panel. Because of the underlying barrier pillar, the mine anticipated difficult face conditions in this area. And indeed, when the longwall face reached this area, ground

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control problems which included severe face spalling and poor roof condition in the headgate entries were encountered. In fact, the stress interaction with the lower seam was severe enough to stopped the longwall face about 15 m (50 ft) short of the longwall recovery chute and make recovery of the supports difficult.

When comparing conditions in Zone 1 to Zone 2, there seems to be a very fine line on the occurrence of ground control problems in the upper seam depending on the overburden depth and the pillar size in the lower seam. Problems were more likely to occur when the depth of cover over the Sewickley seam exceeded 250 m (820 ft) and when large areas of narrow pillars (less than 10.5 m (35 ft) wide) in the lower seam, were located adjacent to a larger barrier pillar. These conditions caused yielding of the narrow pillars and the shedding of their load to the adjacent larger pillar. This concentrated abutment stress was then transferred to the upper mine resulting in poor ground conditions in areas of the headgate entry and longwall panel. Throughout this case study, the calibrated LAMODEL models, successfully predicted the high stress areas in advance of mining.

CONCLUSIONS

The primary purpose of the case studies present in this paper was to validate the new boundary-element program, LAMODEL, and investigation its utility for stress modeling in mine planning. Based on the comparisons between the stress mapping and the model results for the two case studies, it appears that the LAMODEL program can be calibrated to produce good correlations with the observed stresses. In addition, once realistic pillar strengths and load distributions were established by calibration, the mechanics-based overburden behavior in LAMODEL effectively analyzed the complicated stresses and displacements associated with the complex multiple-seam mining scenarios and successfully predicted upcoming high stress conditions in advance of mining for preventive action by mine management. In case study 1, a calculated multi-seam stress concentration of around 15 MPa (2200 psi) with pillar stresses of 35 MPa (5200 psi) appeared to be an upper limit for successful entry development at this mine. Similarly, in case study 2, a calculated multiple-seam stress concentration of 9 MPa (1300 psi) produced severe face spalling and poor roof conditions in the headgate entries, while a 3 MPa (450 psi) stress concentration was barely noticeable.

A secondary goal of this paper was to present a fairly streamlined, systematic methodology for developing initial material properties and then calibrating these properties to field observations. Initially, the critical material properties (coal, gob, and rock mass) are developed using a combination of laboratory research, empirical formulas and experience. Then, in the calibration process, a previously mined area is "stress mapped" by quantifying the observed pillar and strata behavior using a simple numerical rating system. And finally, the initial material properties are systematically adjusted in subsequent runs of the model until the results provide the best correlation between the predicted stresses and the observed underground stress rating. This methodology of combining empirical pillar strength and abutment

load formulas with in mine stress mapping and the analytical mechanics of a displacement-discontinuity model capitalizes on the strengths of both the empirical and analytical approaches to pillar design to provide a practical technique for mine planning in difficult situations.

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The Design of Room and Pillar Mining Systems in the UK

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ABSTRACT

State of the Art Room and Pillar mining techniques have been employed at two coal mines in the UK in the last two years. This paper describes the design of pillars and rockbolted support systems for Room and Pillar mining operations in these collieries.

Within the last year, rock mass classification systems have been used in the design process of a room and pillar mining layout at one of these collieries. In particular, the roofbolt support system was detailed using these techniques and the support system was linked with operational aspects such as depth of cut. The depth of cut was initially estimated using rock mass classification systems and then verified by a stand up trial which was monitored remotely. The stand up time has a direct influence on the operational aspects of the mine and this is described.

The design was verified by performance monitoring and the results are discussed. The importance of understanding the role of rockbolting is emphasised along with some examples of accurately determining in-situ bolt performance. The paper also recommends some modifications to rock mass classification systems when applied to underground coal mining operations.

INTRODUCTION

Longwall mining has traditionally been used in the UK as the preferred method of coal extraction for many years. In any mining operation, strains are induced above longwall workings and limits have to be placed on this strain at the interface of competent dry rock and water bearing rock, e.g. sea beds and subsurface aquifers in order to avoid excessive strain at these horizons. One method of limiting strain above coal mine workings is to change the mining method from longwall to room and pillar working. Room and pillar mining had been used in the latter years at Point of Ayr

Colliery which was mining beneath the River Dee estuary in North Wales and has been used for the first time at Wistow Mine in the Selby Coalfield in the UK in order to access a region of coal lying at close proximity to a sub surface aquifer.

Although room and pillar mining has been used at other locations in the UK there was a need to gain as much information as possible on the site at Wistow before operations commenced in order to produce an economically viable system and this will be described in more detail in this paper. Consequently, the Geomechanics Rock Mass Classification System (Bieniawski 1984) was used in order to produce a base line design that could be verified by in-situ tests. The initial pillar design was based on the work of Salamon (Madden 1991) and was later modified in view of the in-situ stress regime.

Accurate monitoring of the system performance was considered to be essential and was used to verify the design.

DESCRIPTION OF UK ROOM AND PILLAR WORKING

Multi-entry concept and place changing.

The concept of place changing is to have multiple "places" available to cut any one time. This allows cutting, loading and hauling to be independent of the roofbolting operations, thus maximises machine utilisation. Fig. 1 shows a typical layout. The system is based on cutting a standard advance in one "place" and then flitting the machine into an adjacent "place" to continue the next cut. Shuttle cars are employed to transport the cut mineral back to a common loading point located in the central heading. A roofbolting machine moves into the vacated "place" to install roof and rib support, two roofbolting machines are used simultaneously to minimise the time taken to set supports.

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The sequence of cutting will be designed to minimise machine move times and shuttle car haulage distances. The distance over which both shuttle cars travel in the same road will be minimised to avoid congestion and cable handling problems.

Once the roadway and pillar dimensions were finalised the number of entries were then agreed at five, with angled x-cuts staggered either side of the central conveyor road to limit the size of the central junctions. The number of entries used is a compromise between desired advance rate, roof control, ventilation and machinery limits.

The first area to be extracted by room and pillar mining at Wistow (PE1A's) has a maximum of 5 places active at any one time; one cutting, two bolting and two cut out ready for the bolters to flit into. The sequence of cutting is designed to minimise move times and distances for all vehicles especially the shuttle cars, and will tend to sweep right to left or left to right, concentrating workings. With the bolting cycle being much slower than the cutting cycle a number of places can be cut in advance of available bolters, allowing bolting to be continued between shifts or at weekends.

Deep cuts

The best method of maximising the coal cut each time the machine enters a place is to be cut out as much as possible and this is where the "deep cut" practice comes in.

With remote control miners and onboard ventilation units the combination allows deep cuts to be taken, in excess of 10m abroad and up to a maximum of approx. 7m in the UK. Move times between places can be lengthy and deep cut allows more coal to be mined before each move is encountered reducing travelling delays.

The length of time that unsupported ground is maintained should be minimised, the fact that stand up times allow, say 48 hours does not allow intense cutting at the expense of roofbolting. The placing of support should be as soon as practicable and normally within the same cutting shift.

Stand up times and their relevance to place changing

The mechanism of roof failure in the part of the mine at Wistow chosen for the room and pillar mining system was seen to be failure of discrete parting places up to 1.5m above the seam. These have relatively low shear strength surfaces when compared to the intact rock.

The stand up time is dependent upon the geotechnical parameter in the roof of the excavation. Time based deterioration of roof strata is one function of the mobilisation of low friction parting planes. Where mobilisation occurs the beds delaminate inducing tensile and shear forces which

can fail the beam through buckling.

Trials were carried out to identify safe cut out distances over a minimum period of 48 hours which is considered the maximum likely time delay before roofbolts are installed, e.g. machine breakdowns/weekend stand down.

Where the extended cut out distance is proved satisfactory the application of longer cuts is not always the optimum. Wistow has an exemption to operate a 5m cut out distance but has the information to use a 6m cut out. The drawback of 6m is that the ventilation system must always remain within 5m of the cut, which means extension of the vent ducting into the cut out area.

DESIGN OF A ROOM AND PILLAR MINING SYSTEM

Site details in the room and pillar area At Wistow Mine

The Barnsley Seam in the vicinity of the Room and Pillar area at Wistow lies at a depth of approx. 250m. The Barnsley Seam dips to the North East across the take at Wistow and the average seam thickness is 2.6m.

Cores of the overlying roof strata had been taken in the neighbouring H146's Gate and this log is shown in Fig. 2.

The seam was overlain by 1.95m of friable Mudstone containing plant debris. There was an Ironstone band at 1.48m. Above this was a moderate strength Siltstone with a passage top to another Mudstone sequence containing Ironstone bands. This in turn, is overlain by a Siltstone and Mudstone sequence.

Fracture logging of the H146's cores indicate a few sub-vertical fractures in the immediate roof with all others sub-horizontal. The fracture density was high with numerous bedding planes in the first 0.5m and occasionally above. The natural bedding planes are generally planar and smooth. All other fractures were induced by either the drilling process or from previous examination and transport.

Stress field

The take at Wistow is dominated by a series of geological faults which divide the mine into specific blocks. These geological features influence the in-situ stress regime.

Stress measurements had been made in the neighbouring H146's area by using the overcoring technique. The virgin in-situ conditions were given as follows:

Vertical Stress	8 MPa
Max. Horizontal Stress	10.1 MPa on azimuth of 326°

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Min. Horizontal Stress 6.5 MPa on azimuth of 56°

The direction of maximum horizontal stress follows the national trend of North North West to South South East (Azimuth approx. 335°). The horizontal stress anisotropy is 1.55 which was estimated to give some marked directional effects and this proved to be the case. Roadways driven in line with the maximum horizontal stress direction experienced good conditions, roadways driven at 90° to this direction were poor. It was estimated (Bickerton, 1998) that roadways driven at 50° to the maximum horizontal stress direction would experience conditions at least 70% of those driven in line. Underground observations subsequently confirmed this.

Overcoring stress measurements in the centre of a pillar in the Room and Pillar area showed that the vertical stress had increased by approximately 82% of its cover stress. This was in line with expectations.

Pillar sizes

The convention adopted in many parts of the world is to determine an effective pillar safety factor which will ensure stability. The safety factor of each individual pillar is determined from the strength of the pillar and the tributary area load applied due to the overburden weight. The load attracted to each pillar is a function of depth and the extraction ratio in the seam.

The strength of a pillar is a function of the following:

- i) width to height ratio of the pillar
- ii) pillar shape and regularity (in plan)
- iii) in-situ coal strength

A number of laboratory and empirical methods have been devised to determine pillar strength, the most well known being the statistical method of Salamon and Munro which has been applied consistently in South Africa, and other parts of the world, for many years.

Many tens of thousand of stable pillars have been successfully designed using the Salamon principles and later modifications which consider the increasing strength of larger pillars necessary at depths greater than about 100m. These principles were adopted for the design at Point of Ayr and Wistow.

Width to height ratio

Further work has been carried out on the Salamon method to extend its application to greater depths (Madden 1991). This has resulted in the so-called squat pillar method which takes account of the increase in the rate of strength gain as the width to height ratio of the pillar increases. Work in South Africa (and elsewhere) shows that the squat pillar

formula is applicable where the width to height ratio of the pillar is greater than about 4.

At the other end of the scale, evidence from other sources (Parker 1993) suggests that regular shaped pillars with width to height ratios in excess of about 8 are indestructible. A pillar becomes indestructible as a result of the triaxial (horizontal) confinement which is applied to the core of the pillar.

Coal strength

The strength of the coal varies from seam to seam and there are effectively two coal strengths to be considered.

- i) laboratory coal strength
- ii) in-situ coal strength

The in-situ coal strength must always be used in pillar design calculations.

Calculated pillar sizes

For the Wistow layout, it was initially recommended to utilise a 24m square pillar with 6m entries. However, after consideration of roadway roof support requirements it was decided to limit roadway width to 5m. Therefore using the "squat pillar" calculations with 24m pillar and 5m entries the FOS was 1.6. It was felt this allowed for an adequate factor of safety in the design.

After initial development work it was found that roadway roof conditions at 90° to the line of maximum horizontal stress were prone to overbreak to a stable top some 0.9m above the top of the seam. This made mining conditions more difficult. It was decided to re-orientate these roadways at 50° to the best direction, this in turn necessitated the pillar shape becoming a parallelogram.

Using the same "squat pillar" as prescribed by the ARMPS system (after Mark, et al., 1995) which allows for angled cross cuts, the pillar dimensions of 28m by 24m gave a factor of safety of 1.85. This is considered to be an adequate FOS and even more conservative than the original design.

Partial pillar extraction

A technique of Partial Pillar Extraction (PPE) was developed for Point of Ayr Colliery whereby two sides of a square pillar could be mined, on retreat after a Room and Pillar section had been mined to its planned extent. The basic system is shown in Fig. 3. Salamon and Oravec (1976) recommends that for PPE, the primary pillars should have a factor of safety of at least 1.8 and the ultimate factor of safety can be allowed to drop to 1.4. The system at Point of Ayr was designed in accordance with this recommendation and was worked successfully. The system was considered for

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Wistow but was not utilised on the first section as it was felt prudent to initially concentrate on the successful implementation of the new technology on a conventional layout.

Deep cut assessment

An assessment of the Stand Up Time is required in deep cut situations to comply with the Health and Safety Commissions "Guidance on the use of Rockbolts to Support Roadways in Coal Mines (1996)". This is usually accomplished by carrying out trials underground. However, in this new area it was considered that some guidance would be useful before underground operations commenced. Consequently, the Stand Up Time chart from the Geomechanics Rock Mass Classification System was adopted to provide some guidelines. In a mining situation, it has been considered that the roof span means either the span of the roadway, or the distance from the last support set to the unsupported head end. This concept has been used by the Authors for the design of mine supports in many parts of the world and gives rise to the concept of "equivalent span". This being the square root of the product of the roadway span and the distance of cut out, unsupported ground.

In the room and pillar area at Wistow, it was hoped to achieve a maximum of 6m cut out distance in a 5m wide roadway. The equivalent span was therefore 5.5m.

From core information such as that shown in Fig. 2, the basic rock mass rating (RMR) was estimated to be 55. Previous work (Daws, 1991) had shown that it is necessary to modify the basic RMR in coal mines where horizontal stress effects are prominent. Consequently, the basic RMR was modified using the chart as shown in Fig. 4. This gave the following results:

Roadways in line with max. horizontal stress
(Roadways) $RMR \times 0.9$

Roadways at 50° to max. horizontal stress (Cross Cuts) $RMR \times 0.75$

Hence modified RMR's of 49 and 42 respectively were used. Fig. 5 shows these values transposed onto the Stand Up Time chart. It was then estimated that stand up times would be as follows:

Roadways: Approx. 6 days
Cross Cuts: Approx. 2 days

Monitored trial

It was considered essential that the theoretical estimates of Stand Up Time be confirmed in practice and hence a monitored trial was established. Before this could be carried out, the initial circuit for PE1's at Wistow was driven with a maximum cut out distance (PPF or prop free front) of 2.9m.

To make progress towards optimising the system a greater cut out distance was desirable. Access to PE1A's and the proposed large barrier pillars gave the opportunity to develop a trial roadway with varying angles and cut out distances. After consultation with the HSE it was agreed to drive a roadway at 50° to the maximum horizontal stress line and with firstly a 4m cut out then 6m cut out trial length.

At the start of the trial roadway whilst operating the 2.9m PFF, a monitoring station was installed to give monitoring results for comparison with the results from the extended cuts. The monitoring station consisted of strain gauged roofbolts and roof sonic extensometer. These would provide monitoring at the 2.9m cut out and subsequent comparison to the 4m and 6m cut out distances.

4m Trial

A tell tale (TT) at the outbye edge of the trial was installed to confirm any roof overbreak did not override the existing roofbolted roadway. A roof sonic extensometer was installed 2m into the cut and read over the required 48 hour period. Limited roof dilation occurred and confirmed the 4m cut out was safe. This area was bolted up and cut forward a further 10m using the normal 2.9m PFF.

6m Trial

A 6m cut was then trialed, again a TT was installed at the outbye edge of the existing roofbolted roof to confirm any roof overbreak did not override the bolted roof. A sonic roof extensometer was installed 3m into the 6m cut out to show roof dilation.

The results from both the 4 and 6m cuts confirm the increased cut out distance from 2.9 to 4m or 6m could be carried out safely and confidently. At this early stage overbreak was a concern and was around 1.2m. Subsequently this was reduced to 0.3m consistently and the roof was cut to a well defined parting at this horizon. The trial was carried out to comply with the HSE guidance (1996).

From these trials, it was noted that a 5m cut out distance would stand for 3 to 4 days. Therefore, at an equivalent span of 5m, the RMR was back calculated to be 45. This applied to the Cross Cuts and compared well with the originally estimated value of 42.

Design of INITIAL bolting pattern

Previous experience at Wistow in neighbouring single entry facelines with a span of 5m utilised six 2.1m long roofbolts and two 1m long rib bolts in each side, per metre of roadway. These proved entirely satisfactory, but an estimate regarding the support requirements in the room and pillar area was considered to be necessary.

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Use was again made of the Geomechanics Rock Mass Classification System (GRMCS) in order to provide an estimate of anticipated rock loading on the support system.

From the GRMCS, Rock Load Height, ht , is given as follows

$$ht = \left(\frac{100 - RMR}{100} \right) \text{span} \quad (1)$$

Therefore, the maximum load to be taken by the support system, per metre of roadway drive, P , can be expressed as follows

$$P = FOS \times ht \times \text{Span} \times \text{Density of overlying rock} \quad (2)$$

Once this load is known, the rock mass confinement approach to rockbolt design (Daws 1991) can be used to estimate rockbolt spacing and length. The following expression gives the relationship as follows

$$PS^2 + qUS - qUL = 0 \quad (3)$$

Where

S	=	Bolt Spacing (on a grid basis) (m)
U	=	Yield load of rockbolts (tonnes)
q	=	$\tan^2 (45 + \Phi/2)$
Φ	=	Angle of friction of rock mass (from output to GRMCS)
L	=	Rockbolt length (m)

Using an RMR of 42 (minimum for cross cuts); a bolt length of 2.1m, using AT bolts with a minimum yield load of 22tonnes; an angle of friction of 25° (from GRMCS for an RMR of 42), then a spacing of $0.95 \times 1\text{m}$ was calculated giving a Factor of Safety of 1.8, in line with the pillar calculations.

In the event, the layout of the bolter machines meant that a spacing of $0.84 \times 1\text{m}$ was used. This was acceptable as it increased the Factor of Safety.

VERIFICATION OF DESIGN

In order to verify the design of the support system accurate instrumentation was used. This consisted of the following:

Strain Gauged Bolts Nine pairs of strain gauges were attached to rockbolts at intervals along their length. Axial and bending strain could then be measured and bolt load profiles and bending moments

estimated. These bolts were used in place of the normal roofbolts at the instrumented sites.

Sonic Extensometers Up to 20 permanent magnets were anchored into a borehole at intervals around a central guide tube. These extensometers were normally up to 7m in length and placed into the roof and ribs. Readings of inter anchor displacement were taken using a flexible probe and associated readout unit.

The initial circuit in the room and pillar area at Wistow was driven and steel supports were used in addition to the rockbolts whilst the verification process was taking place. Sonic extensometers were installed in relevant areas and strain gauged roofbolts used in the large 4 way junction (JB52's). These gave monitoring information on expected loading and roof dilation. During the drive of the initial air circuit the exemption was received from the HSE to allow primary support on roofbolts to be driven using hand held pneumatic machines at 2.9m PFF. A full sophisticated monitoring station was installed and this was the final instrumentation used in the design process to finalise the support requirements. The information confirmed the roofbolt density of 1.2 bolts/m^2 was satisfactory for roadways and 1.6 bolts/m^2 for junction areas.

The initial area was monitored with the normal dual height tell tales and multi horizon extensometers used in sole support roadways.

The use of this system introduced otherwise untried and untested technology and changes in the normal work practices at Wistow, these were :

1. Use of extended cut, not minimise cut out distance.
2. The use of wide long-term roadways with reduced bolt lengths and densities.

Results of monitoring

The roofbolting densities for a 5m wide opening are 1.2 bolts/m^2 , increasing to 1.6 bolts/m^2 for junction areas. These have proved to be satisfactory. There have been some areas where bolt densities have been increased and long tendon reinforcement has been installed but these are all related to geological features, which reduced the RMR. Typical monitoring shows low roofbolt loads and minimal roof movement.

The original pillar and support design implemented in late 1996 has proved to be satisfactory. The support design has not been altered and actual pillar stress measurement work has found the pillars to be of suitable design.

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Routine roof monitoring in the form of tell tales was very intense at the outset, with initial spacings at 2m, then 4m then 8m then ultimately to mid pillar and junction which is the present position.

Assessment of the monitoring results indicates the initial design is conservative. There is obvious spare capacity within the design, with reduction in pillar size along with an increase in roadway width presently being assessed.

The monitoring data over the last year shows stability and scope for improvement in roadway and pillar widths. The main concern for stability are the large 4 way junctions, in roadways 2 & 4. These present the largest spans and most complex stress regimes.

The layout allows for roadways in three positive directions. The main roadways are in line with the major horizontal stress, the initial PE1's R3 and R4 are at 90° and the cross cuts are at 50° to the best direction. This allows for stress effects to be judged and estimated. These are shown on Fig. 6 (Bickerton, 1998).

CONCLUSIONS

- i) The use of the Geomechanics Rock Mass Classification System allowed estimates of Stand Up Time and rockbolt patterns to be made in advance of developing a new area of the mine at Wistow.
- ii) The Stand Up Times and bolt patterns were subsequently verified by underground trials and accurate monitoring.
- iii) Rock Mass Rating (RMR) was back calculated from the actual trials and found to be in good agreement with the initial estimates taken from core loggings and modified using the factor to consider horizontal stress effects in coal mines.
- iv) These techniques enabled "what if" questions to be checked and considered before the underground operations began and were considered to be a useful addition to mine planning procedures.

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Any views expressed are entirely those of the authors.

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GRAHAM DAW ASSOCIATES						Geotechnical Borehole Log		
Site Wistow Colliery - 146's					Equipment	Cores 45mm diam.		
Distance 99m					Date Drilled 13/12/95			
Samples			Core Run RCD %	Discontinuities		Depth (m)	Lithology	
U.C.S. (N/A)	Soil Strength (C/F)	Strength in Laboratory Tests		Location	Scale (metres)			Log
					- 30° from horizontal	+ 30° from horizontal		
				8.00				Depths not guaranteed and no assessment of core loss made as drilling depth data supplied was not sufficiently accurate.
				8.01			8.01	Total Depth
				7.02				MUDSTONE, silty, grey, plant fragments and debris common, occasional diffuse ironstone bands to 1cm.
				6.00				
				5.98				
				5.00			5.31 5.06	SILTSTONE, fine, brown tinged grey, hard with ferruginous cement, plant debris common.
Comments								
Borehole vertically up in roof								
RCD measured during logging - figures are low due to drilling, transport and examination before this logging.								
A - smooth planar C - smooth undulating N - natural								
B - rough planar D - rough undulating L - longitudinal								
50000 Broken core								
Lost core								
Scale 1/25								
Page 2 of 2								

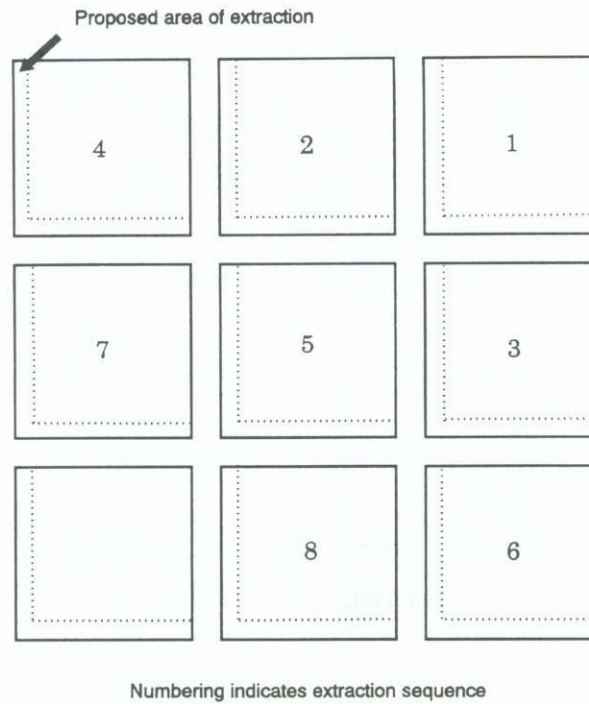


FIG 3 - CONCEPT OF PARTIAL PILLAR EXTRACTION

Multiplying Factor for RMR

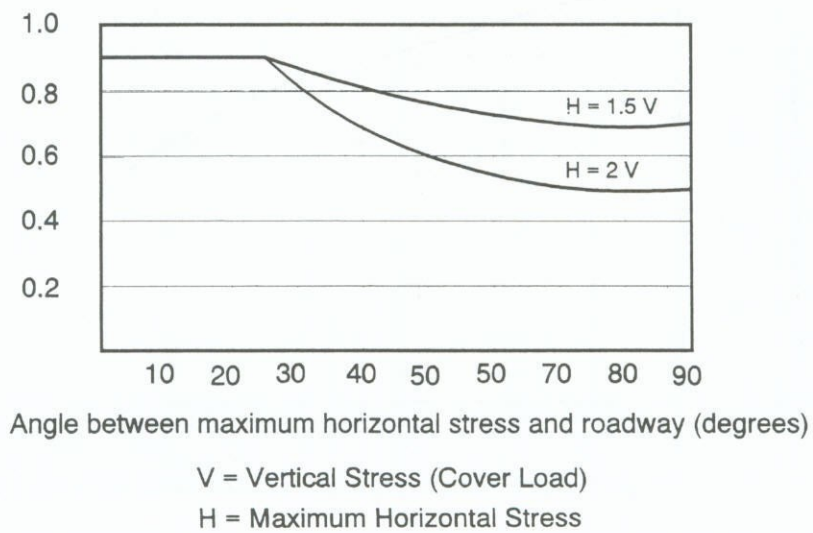


FIG. 4 - RELATIONSHIP BETWEEN ANGLE OF MAXIMUM HORIZONTAL STRESS TO ROADWAY DIRECTION AND MULTIPLYING FACTOR FOR RMR

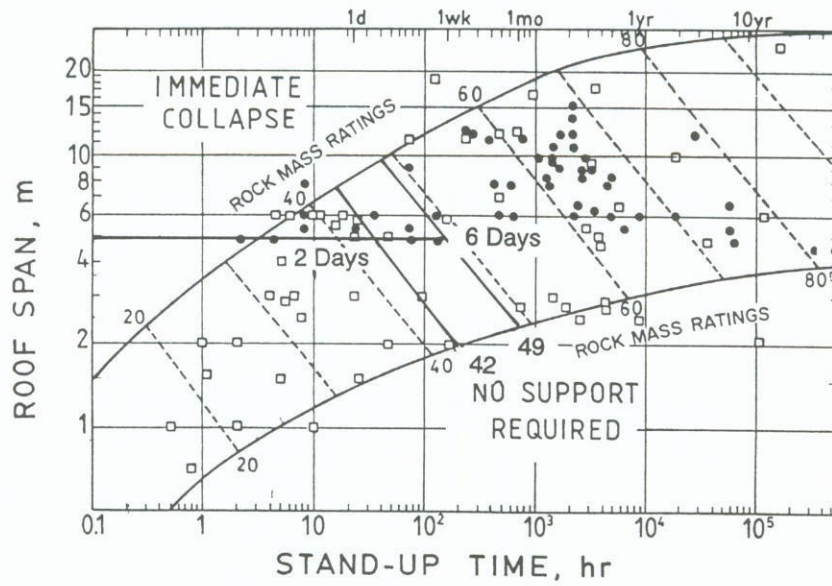


FIG. 5 - INITIAL STAND UP TIME ESTIMATES



Fig 6 - ROADWAY CONDITIONS IN RELATION TO ANGLE OF ROADWAY TO DIRECTION OF MAXIMUM HORIZONTAL STRESS (after Bickerton 1998)



Geotechnical Planning and Development of the BHP Minerals San Juan Pilot Mine, Waterflow, NM

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ABSTRACT

Due to ever-increasing stripping ratios associated with the No. 8 Seam at the San Juan Mine surface operation, BHP Minerals decided to implement longwall mining for the extraction of the substantial future reserve base. In preparation for longwall developments, BHP opted to first develop a small-scale pilot operation within the No.8 Seam on an adjacent lease to address geotechnical/geological concerns related to ground control, subsidence, caving, panel and entry design, and operational issues related to entry development and proper face equipment selection. Room-and-pillar mining methods are currently being utilized to develop a full-extraction panel for a wide range of geological, geotechnical, and geomechanical evaluations aimed at characterizing and assessing the mineability of the No.8 Seam. Variable roof and floor conditions, coupled with a multi-bench coal seam, require that a number of mining horizons, opening geometries, and roof and rib support fixtures be evaluated relative to expediting gate and panel developments. Issues related to development of timely caves in areas subject to overlying sand channel deposits will also be addressed through underground in-seam stress and deformation measurements, and interburden and surface subsidence monitoring. This information will ultimately be used to evaluate face support and gate and panel pillar/entry development requirements. The current depositional model for the property, developed from an aggressive drilling, highwall mapping, and lineament evaluation program, will also be refined during the Pilot Mine study through extensive in-mine mapping, additional roof and floor drilling and physical property testing, and determination of the in situ principal horizontal stress field.

This report describes the rationale supporting the development of the San Juan Pilot Mine study, geologic depositional modeling for the property, mining layout for the test panel, configuration of the measurement and monitoring plan for the operation, and the results to date. Although planning for this project began several years ago, portaling of the Pilot Mine commenced in March 1998, with test panel

retreat mining scheduled for later in the summer. This study currently represents the only underground pilot operation supporting future longwall developments in the U.S. coal industry, and represents a very prudent approach to protecting capital investments in seam settings of increasing geotechnical uncertainty.

INTRODUCTION

As part of a larger effort to provide a long-term, cost-competitive fuel supply to the San Juan Generating Station, BHP Mineral's San Juan Coal Company (SJCC) is conducting a limited underground trial mining operation (Pilot Mine) adjacent to existing surface operations at the company's San Juan Mine, near Waterflow, NM (Figure 1). The purpose of the Pilot Mine is to gather geologic, geotechnical, and mine hazard data relevant to developing mine design parameters, specifying ground control practices, and predicting surface subsidence for a larger scale underground operation. The information gained, and associated engineering analyses conducted will ultimately be used to address the following questions...

Is longwall mining feasible at the San Juan Mine Deep Lease and Deep Lease Extension properties?

What specific mine layout and operational approach is required to maximize mine safety, productivity, and reserve utilization?

To best address these questions, a four-phase program was developed involving (1) *site characterization*, including a review of property geologic work completed to date, planned and recommended drilling at the selected Pilot Mine site, physical property data acquired from the drilling program, an areal lineament study, and a review of horizontal stress impacts on mining; (2) *development of a data acquisition program*, including design/orientation recommendations for the Pilot Mine mains and panel developments, complete in-

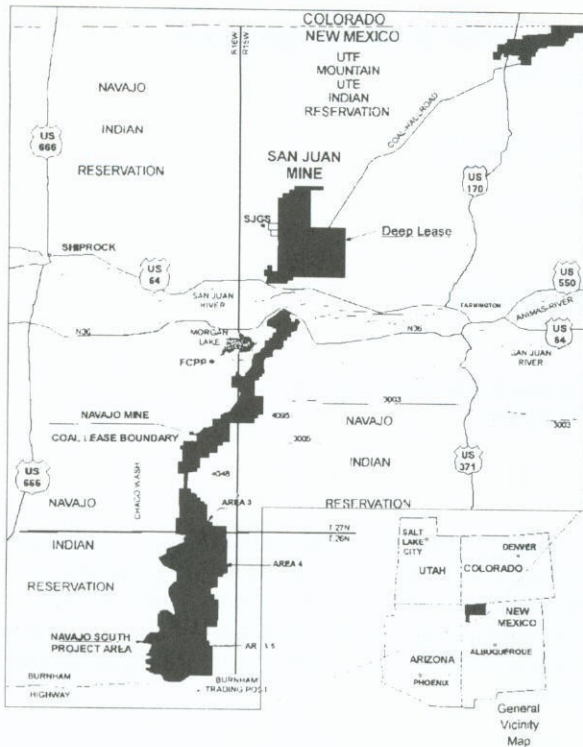


Figure 1. Location map for the San Juan Mine, Waterflow, NM.

mine testing program layout, and final development of the data acquisition system and controls; (3) **implementation of the data acquisition program**, including final scheduling of underground evaluations, daily data acquisition, and preliminary data refinement and assessment; and (4) **final data analyses and property mine design specification**, including complete analysis of the mine layout and support structures, specification of ground control practices, and prediction of surface subsidence.

At the time of this report, Phases 1 and 2 have largely been completed and the Pilot Mine developments are well underway. A comprehensive review of the site geology and historical mining in the region, preliminary characterization of the ground mass through a drilling and testing program, development of an interpretive lineament map for the property and surrounding area, and assessment of potential horizontal stress issues impacting the property have been completed. In addition, numerical modeling and empirical evaluations of the proposed panel layout have been conducted for a full-retreat mining scenario, with a complimentary instrumentation and monitoring program devised for adjacent retreat panels. The primary thrust of the effort to date has been to describe potential ground control problems related to development of the Pilot Mine, and to utilize this information as baseline data supporting design developments for future longwalling within the Deep Lease area.

DEPOSITIONAL MODELING

General Model Description

Korte (1995) has provided by far the most detailed accounting of site geology for the San Juan property to date, and continues to refine the site depositional model based on continued highwall mapping, regional structure analyses (lineament evaluations), and an on-going drilling program. An example of an initial depositional model given by Korte is shown in Figure 2. Depositional modeling is not only useful in identifying coal trends, but is a key to understanding the distribution of poor roof sequences, sand channeling, and fault structures across a property—potentially saving millions of dollars in lost reserves, equipment, and down-time, while increasing the safety of underground personnel.

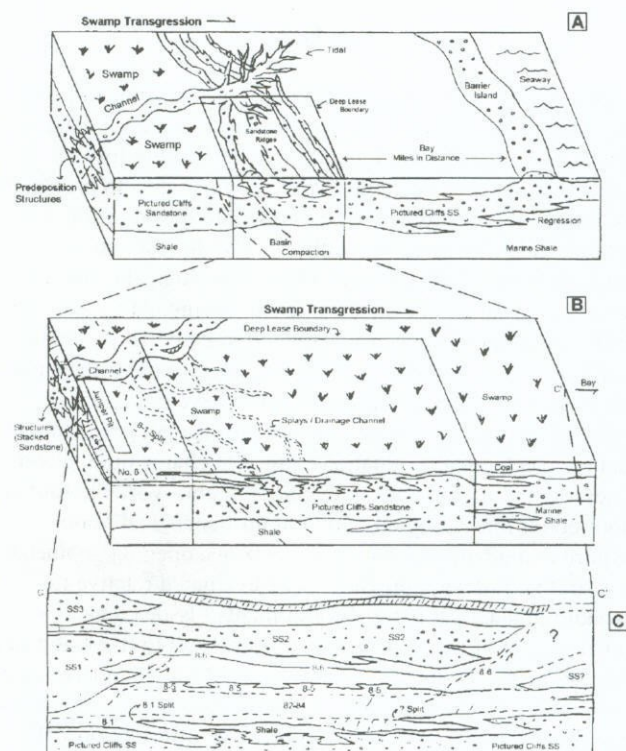


Figure 2. Possible sequence of No. 8 coal seam deposition (Korte, 1995).

Korte has identified six persistent coal benches comprising the No. 8 Seam, which is located within the Fruitland Formation in the San Juan Coal Basin of northwestern New Mexico. From the bottom up, these benches are listed as 8-1 through 8-6. The 8-5 bench can be further subdivided into three additional benches: the 8-5, 8-5a, and 8-5b. Each bench is defined by partings or, in some cases, a series of small partings. It has been assumed that due to the weak nature of the materials overlying the 8-5 bench (coaly shales/weak mudstones) that development mining will be relegated to the 8-1 through 8-4 benches, leaving the 8-5 as the immediate

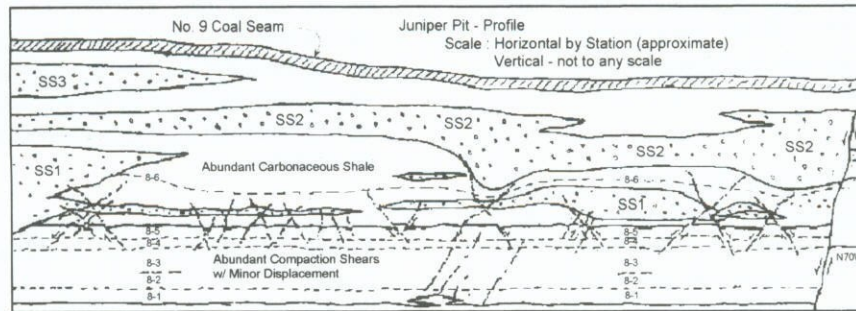


Figure 3. Example highwall stratigraphy within the current Juniper Pit (Korte, 1995).

roof. Historical mining in the area (small punch mines) indicates that a coal roof helps to effect stable mining conditions in the No. 8 Seam. It is anticipated that roof failure modes will involve delamination of weak, jointed beds along poor bedding planes and rider coal horizons, with delamination aided by swelling clays in the roof lithology, block failures associated with compactional shears and faults, and possibly horizontal stress-related shear failures. Excessive spans may also accompany rib yielding in high openings at depths in excess of 600-800 ft. To date, Pilot Mine developments have achieved stable roof conditions through the proper use of top coal coupled with both tensioned and untensioned support fixtures and adequate surface control.

Although the No. 8 Seam is considered largely undisturbed by large structural features, (no major faulting is expected across the property) three structural features will potentially impact entry development and possibly longwalling: minor faults (displacements less than 10 ft, most often less than 5 ft), compactional shears, and coal cleat. Small faults and compactional shears frequently occur in the presence of overlying sandstone channels and are projected from the coal seam to the base of these overlying strong units (Figure 3; Korte, 1995). Small graben features are not uncommon in some zones of the pit highwall, with associated shears and/or jointing extending for many feet either side of these larger features along the coal-roof interface. Most of these smaller joint and shear features terminate in the upper portion of the 8-5 bench; however, Korte notes that jointing is also present within the coal seam at some locations. Where these structures displace the seam, marginal roof conditions may be expected to occur. Predicting trends for these structures across the property will be a focal task for future site geologic mapping and modeling.

Coal cleating roughly parallels the inferred N40-50E principal horizontal stress direction, though localized rotation is expected to occur in the vicinity of major sand channels. It is not known what effect the cleat will have on rib stability; however, observations in the pit highwall indicate the face cleat is pronounced and may interact with partings between benches. Strong cleating coupled with high entries (greater than 9-10 ft), increasing mining depth, and longwall abutments may result in widening roof spans due to rib failures. Some form of rib

control may be required as mining progresses to cover depths in excess of 1,000 ft.

The immediate (1-2 ft) floor is described as being weak and susceptible to water damage. Because of the likelihood for floor damage to occur in travelways, it has been recommended to investigate the issue of leaving all or a portion of the 8-1 bench as the immediate floor. The quality of floor subject to travel and water will be investigated in greater detail during the Pilot Mine study. To date, floor damage has not been observed in the limited workings of the Pilot Mine.

Four major sandstone channel units have also been described by Korte, three residing just above the No. 8 Seam within the No. 8-No. 9 Seam interburden (SS1-SS3), and one immediately overlying the No. 9 Seam (SS4). Accurately predicting the course of these sandstones across the property as mining advances will be critical to predicting the presence of compactional structures associated with these channel deposits. Localized reorientation of coal and roof structures may also accompany these units as they migrate closer to the seam. Some of the worst mining conditions encountered in western U.S. underground coal mining are associated with sand channel systems—primarily related to differentially compacted strata within channel margins, compactional features immediately below the main sandstone body, water inflows from saturated channels, and poor caving and cantilevering roof over face shields and along gate roads adjacent to developing gobs. The generally weak nature of the sandstone units encountered thus far in the surface mining operation suggests that caving-related issues will not greatly impact the San Juan underground operations.

Korte's work will be carried forward into the Pilot Mine evaluation to determine the impact that coal and immediate roof structures have on entry and pillar stability during panel retreat mining. Much of the original work will likely be refined once ground response to mining is better characterized as development proceeds.

Geotechnical Issues Based on Highwall Mapping

NSA Engineering project personnel have observed and documented highwall conditions in conjunction with Korte's work over the past several years and agree that, from a mine

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design and geotechnical perspective, several topics are of key concern relative to the intact rock strength and rock mass structural condition of the proposed immediate roof strata. More specifically, the following items are of greatest immediate concern:

- **Intact rock strengths of proposed immediate roof strata.**

As determined from laboratory physical property tests, the intact rock strengths of the individual coal benches and immediate roof strata are moderately low compared to typical strength and stiffness data for western U.S. coal measure materials. The aggregate weighted intact strength of the proposed immediate roof strata (potential primary and secondary support horizons from the base of the 8-5 split upward) is further decreased by numerous carbonaceous shale, shaly coal, and ash bands, and 8-5 Seam splits that are generally too weak to test (i.e., standardized samples cannot be prepared). The materials are relatively soft (low stiffness), moderately weak (low unconfined compressive strength), and likely exhibit poor shear strength.

- **Lateral continuity of 8-5 to 8-6 interburden units.**

As described by Korte (1995) the 8-5 Seam splits (8-5, 8-5a, and 8-5b) and 8-5 to 8-6 Seam interburden can vary considerably in thickness and character. Of particular concern to roof support, assuming the 8-5 split comprises the bottom of the immediate bolting horizon, is the carbonaceous shale/boney coal units within the 8-5a and 8-5b units, and the variable-thickness partings separating these units from one another and from the 8-5 bench. Depending upon the presence of the partings between splits, the primary bolted horizon may contain all three coal units, or may require secondary support measures (e.g., cable bolting) to attain adequate anchorage in overlying 8-5/8-6 interburden materials. The overlying interburden between the 8-5 and 8-6 benches ranges in thickness from 2 ft. to 15 ft. and contains varying amounts of structural disturbance (compactional features). Overall, this immediate roof setting complicates the determination of primary support installation requirements, and whether some form of secondary support is required on a routine basis. To date, tensioned support fixtures have been successful in maintaining a quality roof, despite the weak character of the intervening strata.

- **Abundance of low-angle compactional shears in the immediate roof.**

Coupled with inherently weak, discontinuous strata units, low-angle compactional shears and faults in the immediate roof (first 10-20 ft) represent perhaps the most severe structural problem expected to impact roof stability over a potentially large portion of the underground property. These structures have been well documented by Korte (1995) as well as by NSA personnel during site visits. The most pronounced of these structures have been related to the differentially compacted sediments along overlying channel margins, though the spatial distribution, occurrence, and

persistence of these structures have yet to be fully modeled for the property. It is likely that proper entry orientation and secondary support selection, coupled with predictive modeling from the on-going depositional work, will satisfactorily mitigate problems associated with these structures.

- **Influence of channel sandstones.**

Channel sandstones are associated with poor ground conditions throughout western U.S. operations and will likely impact mine roof stability at the San Juan operation. Differentially compacted material along channel margins is commonly structured, slickensided, and poorly bonded to overlying strong strata units. The degree of immediate roof damage and disruption is a function of proximity to the seam of overlying channels, the thickness of individual channel units, and whether more than one channel is present (stacked both laterally and vertically). Within the Juniper Pit, sandstones have been responsible for the differential shear compactions witnessed throughout sections of the highwall (not pervasive everywhere), and have, on occasion (past highwall exams), scoured the immediate roof strata. Highwall maps compiled by Korte, Christensen, and Ludwig (BHP staff) clearly indicate the relative influence of sandstone channel features as they migrate over and across the No. 8 Seam.

- **Susceptibility of weak roof strata to water degradation and air slaking.**

During the drilling program reported on by Korte (1995), "...extreme core loss occurred due to expansion of the shale in the immediate roof and coal seam partings present in the 8-5 benches." There is little question that pervasive water (most likely introduced during roof bolting, should wet drilling be used) will result in further degradation of the roof material—possibly over very short time frames. Air and water susceptibility will be further evaluated during the full-scale geotechnical tests planned for the Pilot Mine; however, for the time being, the immediate roof materials (including the 8-5/8-6 interburden) should be considered extremely weak in the presence of water and susceptible to seasonal air slaking. Relatively dry conditions are expected within the mine, and the region is considered arid—potentially minimizing the influence of seasonal conditions on exposed roof strata. Leaving top coal will also aid in protecting against degradation of the immediate roof.

- **Influence of pronounced cleating on roof and rib stability.**

A well-developed face cleat is evident throughout the coal column and will significantly impact rib stability, and ultimately roof stability, during deeper cover mining (in excess of 600-800 ft). Physical property testing indicates a significant spread of intact coal strengths exists along the mineable coal column, suggesting that isolated bench failures may occur following development mining (deep settings) or with longwall advance, even at relatively shallow cover depths (approximately 400-500 ft). High openings may exacerbate rib deterioration and may result in

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widening entry spans, not to mention potentially hazardous rib fall conditions. The presence of variable-strength and water-susceptible partings within the coal column will also greatly influence the structural stability of the developed ribline. Weak partings will act as low-friction shear zones, and may worsen rib deterioration by decreasing the confining stresses generated through the intact portion of the rock mass. Strong partings may increase the confinement within any given bench, allowing "undermining" of the bench by the gradual failure of weaker benches. As illustrated in Figure 4, shear and joint structures do occur in the upper portions of the seam (as well as within the main benches of the seam (Korte, 1995)), which will also adversely impact strongly cleated sections. Rib deterioration will likely be an issue for entry developments at the San Juan operation and may become a roof support issue as well during the high-abutment-load-generating operations associated with longwalling. Rib control measures will be investigated during the Pilot Mine study. To date, rib control has not been a problem in the Pilot Mine, with proper entry alignment potentially mitigating many of the problems once mining proceeds under deeper cover.

Highwall mapping has provided the best possible insight into potential roof and rib problems to be experienced during underground mining. It has also been an invaluable source of information relating structural anomalies to depositional environment, thereby allowing educated projection of conditions over at least a portion of the property (as provided to date by Korte and Christensen). The immediate indications are that roof stability will be a function of intersecting shear and fault compactional structures, the strength and rigidity of substantial lithologic units in the primary and secondary bolted interval, the presence of weak partings or release surfaces (bedding-plane faults, coal riders, etc.) in the secondary support interval, integrity throughout the ribside coal column, and the ability of the 8-5 bench to provide sufficient structural strength to develop a supportable beam with the overlying weak units.



Figure 4. Typical coal column indicating 8-2 through 8-5 benches (possible top section of 8-1 bench), and a portion of the overlying 8-5/8-6 interburden strata.

The Juniper Pit highwall will continue to serve as a source of detailed information throughout development of the Pilot Mine especially as roof and rib failure modes become more apparent.

Immediate Roof, Seam, and Floor Lithologies

In the summer of 1997, the San Juan Coal Company performed a five-hole exploratory drilling program along the proposed east-west mains driveway bordering the northern edge of the Pilot Mine lease area. Each hole was rotary drilled to within approximately 40 ft of the top of the mineable 8 Seam benches, with NX core drilling then proceeding through the roof, seam, and immediate floor column to acquire detailed information regarding the seam bounding stratigraphy, and to further provide samples for standardized physical property testing. The core was carefully logged, photographed, and packaged and then shipped to Advanced Terra Testing, Lakewood, CO, for test sample selection and preparation by NSA project staff. In addition to the logging, RQD values (Rock Quality Designation) for each major lithologic unit were calculated prior to shipping.

Figure 5 provides a comparative look at the relative lithologic units encountered in each hole (limited to 30 ft above the 8-5 bench, the 8-1 through 8-5 benches, and 10+ ft of floor strata), based on the generalized columns used in presenting RQD values. In general, the following characterizes the mine setting across the Pilot Mine property:

Immediate Roof (Secondary Bolting Interval—8 to 20 ft):

Each of the corehole logs, except 9705, indicates the presence of a substantial siltstone or sandstone unit within the potential anchorage horizon of an integral secondary support system (e.g., cable bolting). The physical properties (strength and stiffness) of these "strong" units, as previously noted, are somewhat low for these types of materials compared to other western U.S. properties. As a result, structural deficiencies will largely dictate the contribution of these units to effecting quality secondary support anchorage horizons. Although such deficiencies were not readily identified in the core logs, experience with compactional shear failures in the vicinity of sandstone units along the highwall suggests that these features may exist in the Pilot Mine as well. The lowest RQD values presented in Figure 5 are found immediately beneath the sandstone units in the upper roof; however, it is not known if these zones represent compactional shears.

Immediate Roof (Primary Bolting Interval—0 to 8 ft):

The immediate bolted interval for primary support fixtures is assumed to include the entire 8-5 bench, which averages close to 1.5 ft thick across the coreholes (not shown in Figure 5). The immediate roof above the 8-5 is primarily carbonaceous shale, and the 8-6 bench is a moderate-to-poor quality primary support horizon. Attaining adequate anchorage for active supports (tensioned fixtures) may be problematic for point-anchor systems, but satisfactory for full-column resin installations (as seen to date). Overall,

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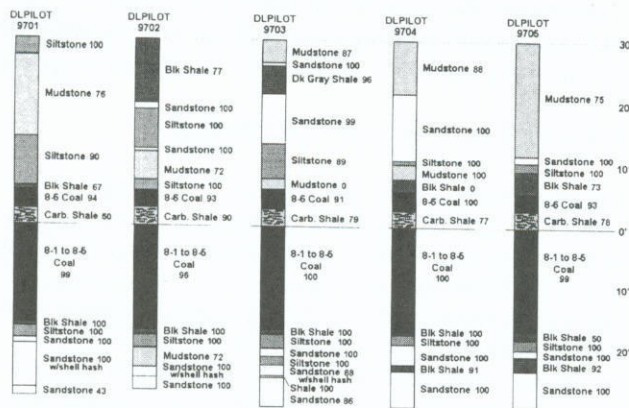


Figure 5. Generalized stratigraphic columns across the Pilot Mine developments for the immediate roof, seam, and floor strata.

the primary bolted interval is characterized by a moderate strength coal bench forming the immediate roofline, overlain by steadily weakening materials that are potentially susceptible to water and air degradation. Thus far in mining, the 8-5 bench has demonstrated a suitable capacity to support the overlying weaker strata.

Minable Coal Seam:

The 8-1 through 8-5 benches maintain relatively uniform thickness across the drilled portion of the Pilot Mine. High RQD's are also evident for these units, suggesting that the cleat may not greatly contribute to rib problems within the Pilot Mine area (which has been the case during initial Pilot Mine development).

Immediate Floor (0 to 10 ft):

The immediate floor strata can be characterized as a thin, moderate strength black shale underlain by siltstone and sandstone strata of variable, but substantial thickness (2-10 ft). Although the strongest materials still possess relatively low strengths and stiffnesses, water susceptibility appears low, suggesting moderate-to-good floor conditions may be experienced over a majority of the property. The weaker portions of the lower 8-1 bench and the black shale may contribute to minor floor problems; however, good floor conditions may be expected within the Pilot Mine developments.

RQD is commonly used in conjunction with more detailed rock mass classification systems that have been empirically related to ground stability and support selection and performance. Such methods include the Rock Mass Rating system developed by the South African Council of Scientific and Industrial Research (CSIR), as proposed by Bieniawski, and the Norwegian Geotechnical Institute "Q" system (Afrouz, 1992). Both of these methods rely on detailed information concerning roof structures—primarily jointing—and will be further developed and evaluated during the full-scale Pilot Mine geotechnical evaluation. These methods have been developed primarily for use in the tunneling industry, but do

give insight into maximum unsupported spans and stand-times for unsupported ground, as well as minimum long-term support requirements. In addition to these assessments, the Coal Mine Roof Rating system (Molinda, 1994) will also be employed to characterize the mine roof encountered in the Pilot Mine.

GEOTECHNICAL ISSUES

Phase 4 of this project will utilize full-scale, in-mine performance data acquired from the Pilot Mine test sites, coupled with the geologic and geotechnical data compiled to date, to develop final designs for longwall layouts, entry systems, roof control, face supports, gate, mains, and barrier pillars, etc. After reviewing the available geotechnical database, the following types of information will be required from the Pilot Mine operation to better address longwall layout and design requirements (addressed in more detail in Phase 2):

Geologic Characterization

- underground stability assessment of identified surface lineaments
- fault, compactional shear zone, and joint mapping (detail lines)
- cleat orientation mapping
- immediate roof and floor strata characterization (coring and fence diagrams)
- prediction of sand channel and fault/shear zone trends across the property
- correlation of roof-rib-floor instabilities to highwall structures

Mining Orientation

- in situ stress measurements in roof and floor strata
- correlation to in-mine geologic mapping
- correlation to roof failure mapping (preferential failure directions)
- correlation to observation hole offsets and bedding shear directions

Caving Characteristics

- determination of abutment angles and initial cave height
- influence of overlying sandstone units
- abutment load timing, magnitude, and distribution
- potential for caving over shields and/or ahead of the face
- potential to arch across face to gate pillars

Mining Horizon

- selection of appropriate roof and floor mining horizons
- operational capability to mine specified entry dimensions (marker beds)
- influence of rock mass structural characteristics on mining horizon selection
- entry versus panel horizons
- influence of coal partings on selection of horizon (rib control, floor horizon selection)



Roof Control

- characterization of roof failure modes
- maximum unsupported span per roof horizon
- stand-times for given roof sequences (support timing and extended cuts)
- determination of minimum support horizons required (top coal requirement)
- primary and secondary support selection performance
- optimal entry dimensions
- active versus passive support options/requirements
- influence of horizontal stress
- special requirements for headgate and tailgate, set-up and recovery rooms, overcasts, etc.
- integral support installation methods
- seasonal/weathering considerations
- intersection development sequences and required support
- standing support selection and applications
- fault/shear zone support

Pillar Design

- potential mains, gate, bleeder, and barrier pillar configurations
- application of abutment versus yield pillar concepts
- yield zone development in pillars versus entry height
- characteristics of pillar failure (weak horizons)
- pillar w/h ratio versus roof/floor stability
- pillar loading profiles in tributary and gob abutment loading conditions

Floor Stability

- trafficability of immediate floor strata beneath the 8-1 bench
- trafficability of the 8-1 bench
- floor failure modes
- pillar punching potential
- water damage potential of selected floor horizons
- influence of horizontal stress
- preferential orientation of floor failures

Rib Control

- rib failure modes (units prone to failure)
- rib failure versus mining depth and location wrt retreating face-line
- rib support performance
- rib stability versus entry height
- pillar corner stability at intersections
- influence of roof and/or floor strata on rib stability

Face Support Selection

- caving height and abutment angle
- caving characteristics (typical versus sandstone channels)
- floor bearing capacity (wet and dry)
- face heights to be considered
- fault, shear, and joint structure orientations
- identification of hanging roof scenarios
- estimated distance to first cave

Set-Up and Recovery Room Design

minimum operational dimensions required

- initial mining horizon required (top coal roof)
- ground deformation and failure modes anticipated
- support methods required (primary, secondary, and standing)
- stand-time duration
- influence of front face abutment

Surface Subsidence

- timing of initial and maximum subsidence
- maximum subsidence achieved versus mining depth and overburden constituents
- angle of draw and maximum surface strains
- relationship to subsurface caving characteristics
- subsidence profiles in changing topography
- subsidence profiles expected over gate pillars and solid coal abutments

Miscellaneous

- water inflow sources/significance
- gas development versus mining advance
- productivities achieved/achievable
- spontaneous combustion experience

The primary goals of the Pilot Mine geotechnical program development under Phases 2 and 3 are to (1) enhance the geologic model of the immediate roof-floor-coal strata sequences, (2) identify potential ground failure modes, and (3) develop the basic rock mechanics parameters necessary to provide specific recommendations regarding future mine layout, ground control practices, and surface movements (relative to existing structures). The data reviewed to date will serve as a strong foundation from which to build upon during later project phases, and will help to direct the Pilot Mine geotechnical program along the lines of information collection and analysis listed herein.

PILOT MINE TEST PROGRAM

Phases 2 and 3 of the geotechnical program involve the development and implementation of the Pilot Mine instrumentation and monitoring program. The ultimate goal of this work is to provide quantitative design information relative to longwall developments within the Deep Lease area of the property. The challenge of this effort lies in the fact that the Pilot Mine will be developed utilizing full-extraction room-and-pillar mining, as opposed to the longwall method. Acquiring representative pillar and entry load and deformation data immediate to an advancing gob requires careful planning of the instrumentation network to adequately characterize front and side abutment loads, potential tailgate roof conditions, pillar behavior, face support performance, caving characteristics, etc.

Figure 6 illustrates a portion of the instrumentation installation planned within the first panel developments. Six

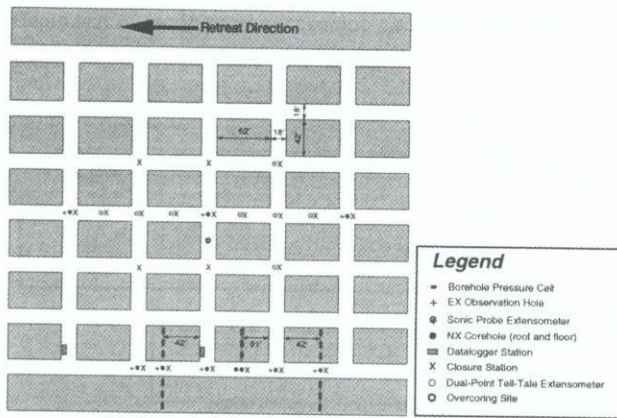


Figure 6. Proposed Instrumentation plan for first and second panel mining.

pillars will be retreated, leaving a single bleeder row (instrumented pillar line) for first panel mining. Second panel retreat (not shown) will develop the intervening barrier and pull the first panel bleeder pillars. The mid-panel instrumentation is intended to evaluate roof stability and entry deformation ahead of the retreating pillar line and to roughly determine the front face abutment profile for four pillar line extraction sequences. This information will also be coupled to an overlying surface subsidence network for longitudinal and transverse panel profiles, as well as to time domain reflectometry (TDR) installations along the centerline of the panel. The mid-panel instrumentation consists of multi-point sonic probe extensometers, dual-point telltale extensometers, closure point stations, open observation holes, and a stress-relief overcoring site. Multi-point extensometers are utilized to acquire detailed roof movement information relative to support type, bolted interval, and timing of retreat. The dual-point extensometers are used to supplement the more expensive multi-point extensometer installations, providing simple, rapid measurement of gross roof movements. The closure point installations simply allow discernment of floor movements relative to roof deformations, and the open observation holes provide for visual examination of bed separations as well as lateral movements between main roof strata. Stress-relief overcoring is used to determine the secondary horizontal stress components impacting roof stability—providing absolute stress magnitudes in the horizontal plane, relative stress ratios between maximum and minimum stress components, and absolute orientations for each component.

The bleeder instrumentation shown has been planned to help characterize the front face and side abutments during first panel mining, the second panel front face abutment, and resulting entry deformations during second panel mining (simulating tailgate conditions). In addition to a detailed examination of roof movements utilizing the aforementioned instrumentation, USBM-style borehole pressure cells (BPC's) will also be employed to provide relative pillar and panel load data. Although this gage does not measure absolute stresses, relative load distribution, magnitude, and timing are all well characterized during mining. This information is key to understanding load distributions about the periphery of the

developing gob, and directly impacts gate road pillar, end-panel barrier pillar, and gate road roof support design.

In addition to the test site configuration development, which includes not only the instrumentation effort described above but also a surface subsidence monitoring plan, a cave monitoring plan, and an end-panel/first cave characterization plan, NSA Engineering also performed a preliminary numerical modeling assessment of the proposed panel layout to identify the optimum locale for the study sites. Figure 7 briefly illustrates the results of the NSA-modified Mulsim/NL boundary element numerical modeling application to the evaluation of pillar loads following development and during panel retreat along the entire length of the panel. The model accounts for actual pillar geometry and overburden topography, and gives an early indication of anticipated high stress areas within the panel—areas where monitoring will focus to provide conservative estimates of ground response to eventual longwalling. The NSA version of Mulsim/NL provides a unique three-dimensional display of the yield condition for model elements, with color-coded elements representative of the degree of yielding at equilibrium, and the height of the element bar indicating relative stress level. Depiction of the modeling results in this manner provides the mine operator with a rapid means for evaluating potential problem areas prior to mining.

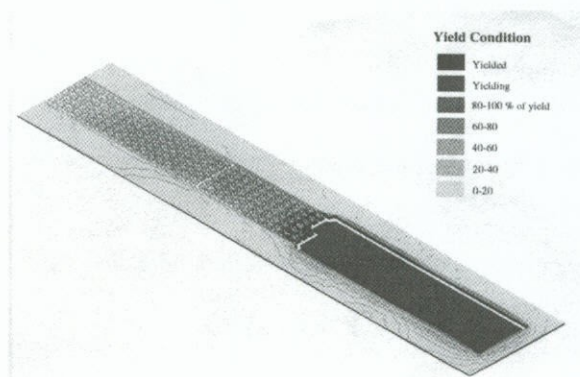


Figure 7. Mulsim/NL boundary-element modeling assessment of stress conditions for the first panel developments, indicating yield condition immediate to the retreat line.

SUMMARY

BHP Minerals has prudently undertaken the development of a pilot mine at the current San Juan Mine operation, near Waterflow, NM, to determine the full-scale feasibility of implementing longwall mining within the future Deep Lease region of the property. To date, an aggressive drilling program, coupled with highwall mapping within the current surface operation, Landsat image lineament mapping, and geotechnical testing of recovered core samples, has developed the basis for

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the area depositional environment. Forthcoming geotechnical monitoring within the developing Pilot Mine will help to characterize the response of the geologic environment to high-extraction mining. The information acquired during the Pilot Mine evaluation will then be utilized to formalize the elements of a successful longwall operation—providing information relative to proper panel layout and sequencing, required primary and secondary roof support, face support selection, gate, bleeder, and barrier pillar dimensions, and entry configurations, to name a few. Without a substantial prior history of underground mining within the San Juan coal basin, and considering the questions surrounding the capability of the immediate roof to remain stable without extraordinary support measures, the development of the pilot operation is well justified.

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Coal Pillar Life Prediction in the Vaal Basin, South Africa

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ABSTRACT

This investigation was done to develop a method to predict the life of coal pillars. It has long since been known that coal pillars deteriorate over time, but there was no method to quantify the deterioration.

It was found that the rate of pillar deterioration was essentially a function of two parameters, namely mining height and time. The higher the mining height, the more rapidly the pillars deteriorated. The rate of deterioration is not constant, but decreases exponentially with time.

The pillar life should be viewed in conjunction with the safety factor. While the relationship between safety factor and pillar life is ambiguous, the trend is nonetheless for higher safety factor pillars to have longer lives. The safety factor can be used to quantify a failure probability. The pillar life prediction indicates **when** the pillars which are predicted to fail, can be expected to do so.

INTRODUCTION

Recent unexpected subsidence in South Africa led to the realisation that coal pillars will not necessarily be stable forever. Failure of very old coal pillars poses two problems: Firstly, it may place undermined structures at risk and secondly it places restrictions on the expansion of infrastructure over previously mine areas.

In the past, the safety factor was used as the only norm to predict whether or not pillars will be stable. It was assumed that pillars with high safety factors do not pose any meaningful risk of instability. However, as Figure 1 indicates, even those pillars may fail, and, perhaps more disturbingly, there is no correlation between safety factors and pillar lives.

The work was initially done in order to predict the long term stability of coal pillars in the Vaal Basin, and intended for later extension to other coal fields. The reason for choosing the Vaal Basin as the initial study area was that there are more failures in that region than in the others and because the coal strength has been defined specifically for that region (Van der Merwe, 1993).

It has been argued that the safety factor, being empirically derived, implicitly includes time effects because the data base included very old stable pillars. However, figure 1 indicates very clearly that the safety factor cannot be used to predict pillar lives quantitatively.

This study was based on the recorded cases of pillars that have failed. Therefore, it is valid for that very small group of pillars out of the whole population that are likely to fail, quantifying their time of failure. It thus follows that it predicts the *minimum* life of pillars. Combined with the safety factor, it states the following about a pillar mining system: "The probability of these pillars failing is x per million, and then it may fail only after y years have passed."

Another potential approach to the undermining of structures would be to design for absolute stability. According to Pyrak-Nolte (1988), the fracture process in rock starts at 40% of the ultimate strength, and therefore a layout that is designed to contain the load to less than that will be stable, as the failure process will never start. However, the inherent strength of the coal material is highly variable and not known for individual pillars. The k-constant of 7.2 MPa used in the Salamon and Munro (1967) safety factor formula (or 4.5 MPa for the Vaal Basin) is an average, empirically derived figure for all the pillars in South Africa.

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Safety Factors and Pillar Lives at Failure

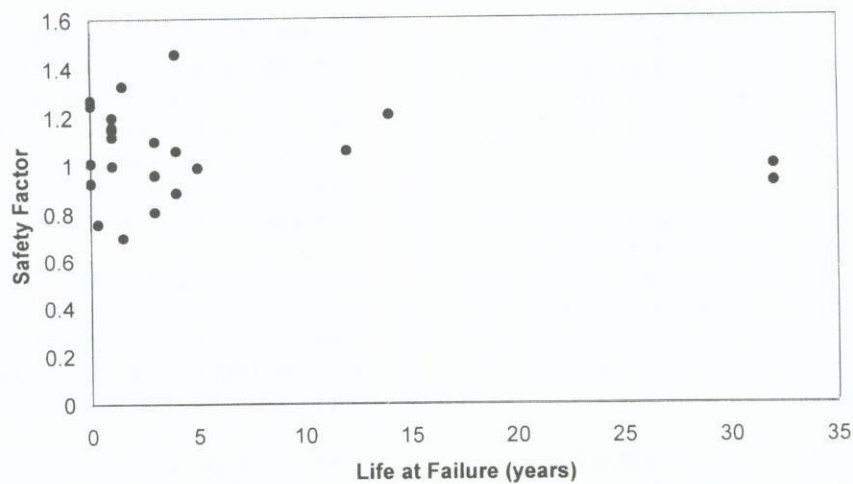


Figure 1. Pillar lives and safety factors. The data in the figure was obtained from Madden and Hardman (1992).

MODE OF PILLAR FAILURE IN THE VAAL BASIN

Visual inspection of pillars at Sigma Colliery indicated that the mode of pillar failure was progressive scaling of pillar sides, like the failure modes proposed by Wilson (1972) and Barron and Pen (1992). This is illustrated in Figure 2.

Previously virtually all the Vaal Basin failures were attributed by others to roof falls. In the author's personal experience of several years dealing with the coal stability in that area, this is simply not the case. It is true that roof falls occur more frequently in the Vaal Basin than in other coal fields and that some of the falls may progress into the roof for several metres, but the fact remains that



Figure 2. Illustration of progressive pillar scaling.

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the roof falls are statistically scattered and unlikely to affect a sufficient number of pillars in any one area to induce pillar failure.

By contrast, all pillars in that area are prone to more severe scaling than elsewhere. At least two sites were directly observed where pillars were in the process of failing. The observed failure mode was severe scaling, while the roof remained more or less intact.

Once sufficient scaling had taken place, logic dictates that at some or other stage the span between pillars should be great enough to induce roof falls, which could accelerate the failure process. However, this has to be true for the pillar systems in other areas and seams as well. There is no reason to attribute this argument exclusively to the Vaal Basin.

The failure process of a pillar in a semi-contained environment underground is complex. As the pillar scales progressively, its load bearing area decreases, progressively increasing the pillar load. Eventually the bearing strength of the floor or the roof could be exceeded and the pillar may punch into the roof or floor. The punching action may often generate lateral tensile forces in the pillar, tearing it apart.

However, both the fallen roof material and the pillar sidewall debris, if not removed, will afford at least some confinement and hence strength to the pillar. At this stage there are simply too many unknowns to allow reliable numerical modelling of pillar failure to be done.

Under these conditions, one has to make simplifying assumptions. In this case, it was assumed that the mode of failure is progressive scaling. The effect of the simplifications is softened by the empirical nature of the investigation: whatever the assumptions, the end results have to agree with reality or else the method is invalid.

OUTLINE OF METHOD OF INVESTIGATION

The safety factors of pillars in the Vaal Basin were calculated with a "k" constant in the Salamon and Munro formula equal to 4.5 MPa. It was then seen that at a safety factor of 0.3 all pillars had failed, or that at a safety factor of 0.3 the failure probability was 100%. On the basis of the evidence it was concluded that at a safety factor of 0.3, a pillar has to fail.

The minimum safety factor used to calculate the scaling distance is to some extent arbitrary. However, it is of little consequence. Choosing any other minimum would have resulted in both a different scaling distance and rate of scaling, without an effect on the time taken to fail. The lowest possible safety factor was preferred, because if any other limit is chosen, some pillars in the data base will have a negative scaling distance, i.e. they would

have had to expand to reach the required width. Using the lowest possible safety factor simplifies the mathematics without affecting the final outcome.

For each of the recorded cases of pillar failure, the difference was calculated between the pillar width as mined and the width to result in a safety factor of 0.3. Then, for each of the 27 recorded cases, a rate of scaling was calculated by dividing the scaling distance by the recorded life of the pillars. Note that this is an average rate over the full life of the pillars.

The next step was to seek correlations between the scaling rates and other characteristics which could be used for predictive purposes. The characteristics included depth, pillar stress, safety factor, width, width to height ratio, mining height, life of pillar, etc.

There were good correlations with only two of the characteristics: mining height and pillar life, as shown in Table 1. The higher the pillar, the greater the scaling rate and the older the pillar at the time of failure, the lower the rate. The scaling rate decelerates exponentially with time.

Table 1. Correlation coefficients between scaling rates and selected other pillar characteristics

Characteristic	Correlation Coefficient	Log-log Correlation Coefficient
Mining height	0.844	0.743
Depth	0.471	0.545
Load	0.389	0.441
Safety factor	-0.369	-0.521
Pillar width	0.311	0.378
Width to height	-0.289	-0.174
Pillar life	0.522	-0.929

The latter conclusion is especially interesting. One line of reasoning suggests that the more the pillar scales, the higher its load and consequently the rate should increase. What probably happens is that as the pillar continues to fracture, the confinement builds up, increasing its strength and thus retarding the process.

It is also conceivable that atmospheric weathering affects pillar strength over time. The inner portions of the pillar, being protected from the atmosphere, would then weather at a lower rate.

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Equations were found describing the best fits for mining height and time respectively to the scaling rate data. The two equations were then combined into a single one and the relative contributions of each, and the final constants, were refined to obtain the best retrospective predictions of the lives of the recorded cases. The correlation coefficient between recorded and predicted lives was 0.787 for the final equation.

d_v	=	Vaal Basin scaling distance
w	=	pillar width
h	=	mining height
H	=	mining depth
C	=	pillar centres
SF	=	Vaal basin safety factor

Then, the rate of scaling, r_v , is:

$$r_v = \frac{d_v}{L_v} \quad (6)$$

Where L_v is the observed Vaal Basin pillar life.

The best fit equations for the best two correlations (i.e. mining height and pillar life) were:

$$r_h = 0.008h^4 + 0.44 \quad (7)$$

and

$$r_i = 16.453 \exp(-1.7L_v) + 0.878 \quad (8)$$

where r_h is the rate with respect to mining height and r_i is the rate with respect to pillar life.

Figures 3 and 4 show predicted versus actual scaling rates for r_h and r_i , respectively.

The best expression for the combined rate was found to be:

$$r_{vp} = (r_h^5 r_i)^{0.1667} \quad (9)$$

It then follows that the predicted life, L_p , is found by:

$$L_p = \frac{d_v}{r_{vp}} \quad (10)$$

Where r_{vp} is the predicted scaling rate for Vaal Basin pillars. Using equation (1) iteratively, pillar lives, L_p , can be predicted. The best fit between predicted and actual lives is found by adjusting L_p as follows:

$$L_{pv} = 0.025(L_p + 2)^{2.5} + 2.178 \quad (11)$$

RESULTS OF THE ANALYSIS

The initial pillar strength according to Van der Merwe (1993) is:

$$\text{Pillar Strength} = 4.5 \frac{w^{0.46}}{h^{0.66}} \quad (1)$$

The Tributary Area Loading is:

$$\text{Pillar Load} = 0.025H \frac{C^2}{w^2} \quad (2)$$

Consequently the Safety Factor of pillars in the Vaal Basin is:

$$SF_v = \frac{180w^{2.46}}{h^{0.66}HC^2} \quad (3)$$

After scaling for a distance d_v , the pillar width is $(w-d_v)$ and equation (3) becomes:

$$SF = \frac{180(w - d_v)^{2.46}}{h^{0.66}HC^2} \quad (4)$$

Equating SF in equation (4) to 0.3 and solving for d_v yields the following expression for d_v :

$$d_v = W - [0.0742h^{0.268}H^{0.407}C^{0.813}] \quad (5)$$

where:

Scaling Rate vs Mining Height

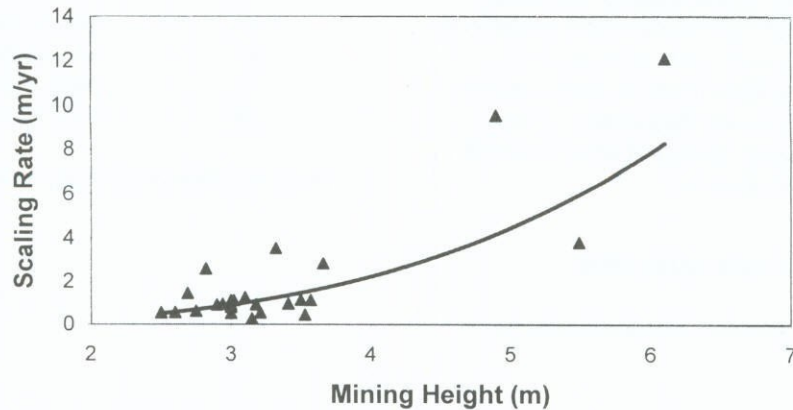


Figure 3. The relationship between mining height and scaling rate.

Scaling Rate vs Time

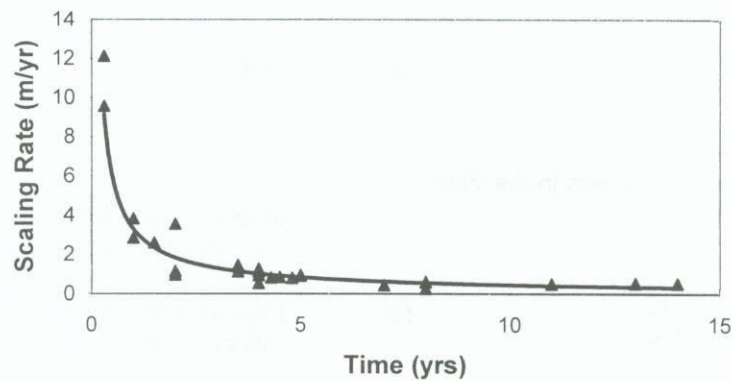


Figure 4. The relationship between time and scaling rate.

The correlation coefficient between L_{pv} and observed pillar lives in the Vaal Basin was 0.787.

Figure 5 shows the error between actual and predicted lives expressed as a ratio of the observed lives, as a function of observed pillar life. Note that the prediction improves markedly for the older pillars. For this reason, the prediction method is deemed satisfactory, as the primary objective is to predict the life of high safety factor pillars with long lives.

THE LINK BETWEEN FAILURE PROBABILITY AND PREDICTED LIFE

In order to be useful in this sense, the safety factor concept's expansion to failure probability needs more prominence. The link between safety factor and failure probability is not theoretical, it is based on real observation. In order to make the calculation easy, an attempt was made to describe the curve mathematically. The curve does not follow any of the standard theoretical paths perfectly and is described more accurately by two equations:

$$F_{p*} = 0.5[1 - \tanh(4.91SF - 4.91)] \text{ for } SF \geq 1.0 \quad (12)$$



Error in Predicted Life - Vaal Basin

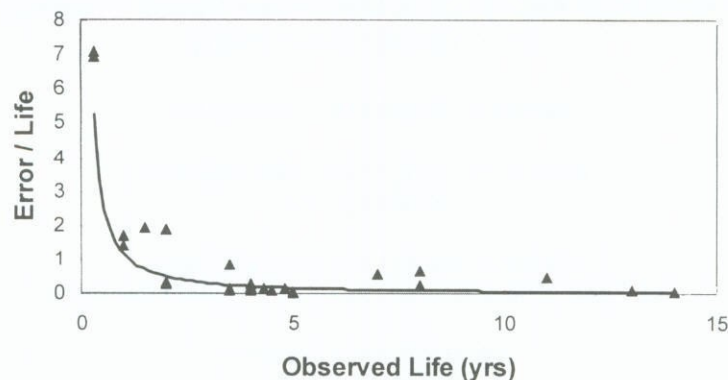


Figure 5. The relative life prediction error, expressed as the absolute difference between observed and predicted lives as a ratio of the observed lives.

and

$$F_{p-} = 0.515[1 - \tanh(5.32SF - 5.02)] \quad \text{for } SF < 1.0 \quad (13)$$

CONCLUSIONS

The method presented here cannot yet be used to predict the life of individual pillared mining systems. However, coupled with the failure probability, it provides an additional consideration for the purposes of evaluating the long term subsidence risk associated with bord and pillar mining methods. As it stands, the method is only valid for the Vaal Basin area. However, to the extent that the coal in other areas will behave fundamentally similar, the basic method can be adjusted relatively simple to refine the method for new areas.

ACKNOWLEDGMENTS

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An Analytical Approach to Determine Stress Distribution in Longwall Chain Pillars

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ABSTRACT

This paper introduces an analytical approach for the determination of the vertical stress in the chain pillar system between two adjacent longwall panels. The development of this approach is based on the principles of fracture mechanics and superposition. Some empirical formulae have also been incorporated into the development of the mathematical model. The mine entries and longwall gobs are treated as Mode I cracks in an infinite plate while the resistance of gob material and the characteristics of the pillar yield zone are considered as the distributed forces on the surface of the cracks. An user-friendly computer program has been developed to facilitate the calculation involved in this task. This approach is as accurate as the numerical analysis methods but requires much less expertise and computer hardware and software to apply.

INTRODUCTION

The magnitude and distribution of the vertical stress in the chain pillar system between longwall panels is an important piece of information for the design of a longwall mine. Determination of the vertical stress normally requires the use of numerical analysis methods (e.g., finite element and finite difference methods). Application of these methods normally require a lengthy data collection and preparation process, relatively expensive computer hardware and software, and specially trained and experienced personnel to use the software and to interpret the computer output. These requirements often prevent most mining companies, especially those small operations, from performing any detailed in-house studies during their mine design task. In addition, the accuracy of the numerical simulations is heavily dependent on the setup of the model, material properties, and the artificial treatments of the longwall gob materials and the caved and fractured strata.

An analytical approach has been developed for the determination of the stress in the chain pillar system of a longwall operation. A number of stress distribution functions for

cracks in fracture mechanics and the principle of superposition have been applied and extended to model the mine openings such as entries, mine gobs. The yield characteristics of the pillars is modeled by incorporating two empirical pillar strength formulae into the development.

Based on the proposed mathematical models, a spreadsheet program has been developed to facilitate the application of the analytical approach. The program is capable of handling three-entry and four-entry longwall systems, both popular in US and Australian longwall mines. For each type of chain pillar system, three mining stages are simulated, that is, (1) entry development only, (2) mining of first adjacent longwall panel, and (3) mining of the second adjacent panel. The program only requires input of some general information about the panel. The program can be operated on any PC with Microsoft Windows and make the stress analysis in coal mine readily accessible by any coal mining companies.

MATHEMATICAL MODEL

In the application of fracture mechanic approach (FMA) in dealing with underground mining problems, a mine opening can be visualized as a thin crack in an infinite rock mass because of its relatively small size in comparison to the overburden strata. Therefore, the mechanics of cracks can be employed to analyze the stresses surrounding the mine opening. The fracture mechanic approach has been successfully applied to determine strata movements caused by underground mining activities (Hackett, 1959; Berry, 1960 and 1963). FMA has also been applied to determine the stress distribution in pillars in coal mines (Kramer, 1996; Hoch et al, 1998). The FMA results are sufficiently accurate when it is compared to the results from the finite difference method. Kramer (1996) also found that the changes of material properties do not affect the stress distribution in the coal pillar significantly based on his sensitivity studies using a series of numerical simulations. Therefore, the FMA provides a much simpler alternative to the numerical analysis methods (i.e., finite element method, finite difference method and boundary element method) in stress

analysis in underground coal mines, especially for those users with limited resources. This section presents some basic approaches in the development of the mathematical model.

Westergaard Function

Two basic functions in fracture mechanics served the foundation of this work. The first one is the *Westergaard function* (Westergaard, 1939). A particular form of the Westergaard function ($\phi = 0$ and $y = 0$) states that the vertical stress distribution adjacent to a thin elliptical crack in an infinite plate (Fig. 1), a mode I crack problem, can be expressed by Eq. 1. It should be noted that the origin of the coordinate system is located at the center of the crack. Since a mine opening can be adequately considered as a thin crack due to its relatively small size, equation 1 can be used to determine the distribution of vertical stress in the coal seam adjacent to a mine opening.

$$\sigma_y(x) = \frac{\sigma_i x}{\sqrt{x^2 - a^2}} \quad |x| \geq a \quad (1)$$

where: $\sigma_y(x)$ - vertical stress
 a - half width of the crack
 σ_i - in-situ vertical stress, $\sigma_i = h\gamma$
 x - distance from the center of the crack

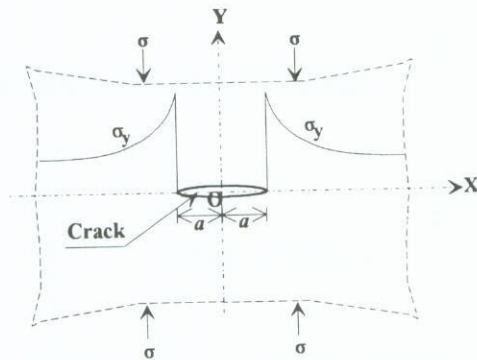


Fig. 1 Stress Distribution Adjacent to a Crack of Width $2a$ Subjected to a Uniform In-situ Stress Field

Green Function

In order to model the influence of the resistance forces of longwall gob material and pillar yield zone on the stress distribution in the chain pillar system, the *Green function* (Paris and Sih, 1965) is used as another basic function. This function allows the determination of the stress intensity factors, important in fracture mechanics (Broek, 1982), at the tips of the crack caused by a wedge force P that pushing against the crack surface (Fig. 2). The stress intensity factors at the tip points A and B of a crack are:

$$K_A = \frac{P}{\sqrt{\pi a}} \sqrt{\frac{a+x}{a-x}} \quad (2)$$

$$K_B = \frac{P}{\sqrt{\pi a}} \sqrt{\frac{a-x}{a+x}} \quad (3)$$

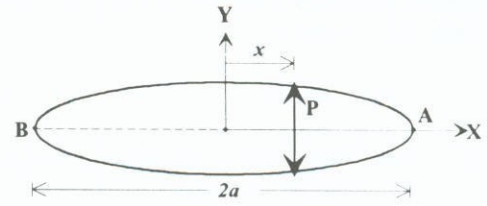


Fig. 2 Crack with wedge force P located at x from its center

where: P - point force per unit plate thickness
 x - location of P from crack center

Modification to the Stress Distribution

The presence of a point force P in a crack will affect the stress distribution surrounding the crack as well as the stress intensity factors at the tips of the crack. In the absence of the point force P , the stress intensity factor at tip point A is:

$$K_{2a} = \sigma_i \sqrt{\pi a} \quad (4)$$

With the point force P in place, the total stress intensity factor at point A, K'_A , becomes:

$$K'_A = K_{2a} - K_A \quad (5)$$

The reduction of the stress intensity factor at point A as shown in Eq. 2 implies that the point force P has the same effect on the stress distribution as a crack of reduced width. The effective width for the stress on tip A side is $2a'$ now and a'_A can be determined as:

$$a'_A = \frac{(K'_A)^2}{\pi \sigma_i^2} \quad (6)$$

The Westergaard function for the vertical stress on tip A side due to the presence of the point force P becomes:

$$\sigma'_y(x) = \frac{\sigma_i x}{\sqrt{x^2 - a'^2_A}} \quad (7)$$

This procedure can be readily applied to represent the effect of post, cribs and other types of artificial supports erected in the entry on the stress distribution in the pillars.

Modeling of Longwall Gob Material

The caved rock in the longwall gob is capable of providing certain amount of support to the overlying strata. Therefore, the preceding procedure can be applied to determine the effect of gob material on the stress distribution in the pillar. The supporting force of the gob material is normally proportional to the degree of compact of the caved rock. Therefore, the supporting force is the least at the head or tail gates of a mined longwall panel and increases toward the center of the gob. Based on the published results of numerous

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fields monitoring, it is reason to assume that the supporting force is linearly distributed as shown in Fig. 3 and the distribution function can be written as:

$$P(x) = P_0 + bx \quad (8)$$

where: P_0 - supporting force at the origin of coordinate system which is defined as

$$P_0 = \frac{\lambda a}{\tan \alpha} \quad (8a)$$

λ - in-situ vertical stress gradient. Normally = 1.1 psi/ ft of overburden strata

α - break angle, normally $\alpha = 21^\circ$ (Fig. 3)

a - half width of the partially supported overburden strata over one half of the longwall gob (Fig. 3)

b - slope of the supporting force function

$$b = -\frac{P_0}{a} \quad (8b)$$

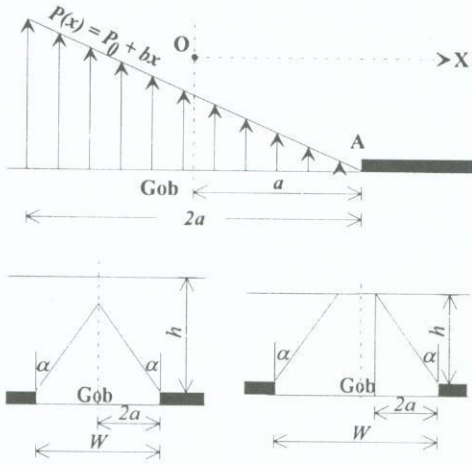


Fig. 3 Supporting Force Distribution in Longwall Gob

By incorporating the supporting force distribution function into Green Function (Eq. 2), the stress intensity factor due to the distributed supporting force of the gob material at the edge of the gate pillar A can be determined as:

$$K_{gob} = \frac{1}{\sqrt{\pi a}} \int_{-a}^a (P_0 + bx) \sqrt{\frac{a+x}{a-x}} dx \quad (9)$$

$$\text{or} \quad K_{gob} = \sqrt{\pi a} \left(\frac{ab}{2} + P_0 \right) \quad (9a)$$

Using the similar approach stated in Eqs. 5 to 7, the distribution of the vertical stress in the pillar with the presence of the longwall gob can be determined.

Elastic Solutions for Different Mining Stages

Without considering the yielding characteristics of the coal pillar, the elastic solution of the vertical stress at a point

in a pillar can be determined by superimposing the vertical stresses caused by different mine openings (i.e., entries and longwall gobs). The distinguished three mining stages in terms of the stress changes that a longwall chain pillar system normally will experience are: (1) completion of the entry development, (2) completion of mining in the first adjacent panel, and (3) completion of mining in the second adjacent panel as shown in Fig. 4. In the first mining stage (entry development stage), the vertical stress at a given point in the pillar can be determined as:

$$\sigma_y = (\sigma_y)_{E1} + (\sigma_y)_{E2} + (\sigma_y)_{E3} \quad (10)$$

In Eq. 10, $(\sigma_y)_{E1}$, $(\sigma_y)_{E2}$ and $(\sigma_y)_{E3}$ are the vertical stresses caused by the presence entries E1, E2 and E3 of a three-entry chain pillar system alone, respectively. Each of the three stresses can be determined using the original Westergaard function (Eq. 1). It should be noted that three local coordinate systems, C1, C2 and C3 (Fig. 4), are employed in the processes of determining stresses $(\sigma_y)_{E1}$, $(\sigma_y)_{E2}$ and $(\sigma_y)_{E3}$, respectively.

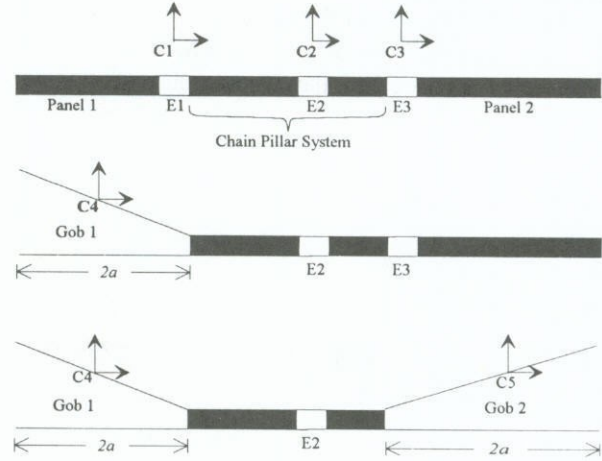


Fig. 4 Three Mining Stages Adjacent to a 3-Entry Chain Pillar System
Top: Development Stage
Middle: Mining of Panel #1
Bottom: Mining of Panel #2

After the first adjacent longwall panel has been mined, one more term, $(\sigma_y)_{G1}$, is added to the right side of Eq. 10. The stress caused by the first gob, $(\sigma_y)_{G1}$, can be determined using Eqs. 4, 5, 6, 7 and 9a. At this mining stage, the vertical stress at a given point in coal pillar is expressed by Eq. 11 while the origin of the local coordinate system for determining $(\sigma_y)_{G1}$ is set at the location of C4 in Fig. 4.

$$\sigma_y = (\sigma_y)_{E1} + (\sigma_y)_{E2} + (\sigma_y)_{E3} + (\sigma_y)_{G1} \quad (11)$$

Similarly, the vertical stress after mining of the second adjacent longwall panel can be expressed as:

$$\sigma_y = (\sigma_y)_{E1} + (\sigma_y)_{E2} + (\sigma_y)_{E3} + (\sigma_y)_{G1} + (\sigma_y)_{G2} \quad (12)$$

The elastic solutions for a four-entry chain pillar system is very similar to those in Eqs. 10, 11 and 12. An item, $(\sigma_y)_{E4}$, should be added to these formulae to account for the stress caused by the development of the fourth entry.

Yield Zones and Its Effect

The original Westergaard function introduces a singularity at the edge of the pillar where $x = a$. This is where the stress distribution approaches infinity under elastic condition. The pillar edge must yield and the excessive stress has to be redistributed to eliminate the singularity. In this process, a yield zone is formed near the edge of the coal pillar. The yield zone retains a residual strength that offers confinement to the core of the pillar. The distribution of the residual strength can be estimated by an empirical formula proposed by Karabin and Evanto (1994).

$$S_R(x) = [0.1385 \cdot \ln(x) + 0.413] \cdot S_p(x) \quad (13)$$

where: $S_R(x)$ - residual stress level, psi
 x - distance from pillar free face, ft
 $S_p(x)$ - peak strength that can be determined by the following empirical Mark-Iannacchione formula (1992)

$$S_p(x) = S_1 \cdot (0.78 + 1.74 \frac{x}{H}) \quad (14)$$

where: S_1 - in-situ coal strength, psi
 H - height of the pillar, ft

In order to determine the depth of the yield zone near the edge of a coal pillar, the residual strength in the yield zone, $S_R(x)$, should be equal to the vertical stress distribution in the non-yield zone, σ_y , at the joint point A between these two zones (Fig. 5). The same procedure for determining the effect of gob material can be employed to determine the distribution of σ_y . In this process, the width of the yield zone, d , should be added to the half width of the original mine opening, a . Therefore, the stress intensity factor at point A of the widened mine opening without considering the residual strength in the yield zone becomes:

$$K = \sigma_1 \sqrt{\pi(a+d)} \quad (15)$$

The stress intensity factor caused by the residual strength of the yield zone can be determined by Eq. 16.

$$K_A = \int_{-\frac{d}{2}}^{\frac{d}{2}} \left[\frac{S_R(x)}{\sqrt{\frac{\pi d}{2}}} \sqrt{\frac{\frac{d}{2}+r}{\frac{d}{2}-r}} \right] dr \quad (16)$$

where d is the depth of the yield zone in the coal pillar which is to be determined. There are two independent variables used in Eq. 16. The first one, x , starts at the pillar edge while the second one, r , originates from the middle point of the yield zone as shown in Fig. 5. There exists a close form solution for Eq. 16 though it is a bit too complicated for the inclusion in this paper. Then the total stress intensity factor at point A is K

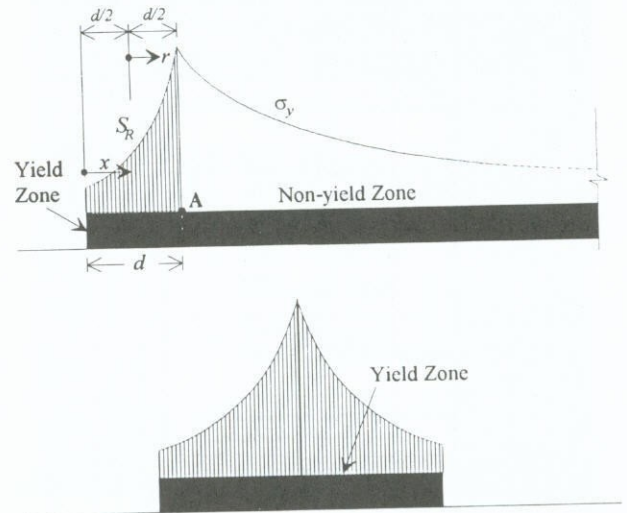


Fig. 5 Determination of the Depth of Yield Zone
 Top: Partially Yield Pillar
 Bottom: Fully Yield Pillar

- K_A and the distribution of the vertical stress in the non-yield zone, σ_y , can be determined for a given d . By setting up the nonlinear equation $\sigma_y(d) = S_R(d)$ and a initial guess value of d , the depth of the yield zone can be solved through an iterative solution process.

If the total width of the yield zones on both side of the pillar is equal to or greater than the actual width of the pillar, this pillar is a fully yield pillar (Fig. 5). In this situation, the two entries adjacent to the yield pillar and the yield pillar itself are combined into one single opening with the resistance of the pillar being treated in the similar way as the supporting face of the gob material. The distribution of the supporting force provided by the yield pillar is symmetrical to the center of the yield pillar with each half being expressed by Eq. 13.

COMPUTER PROGRAM

Based on the FMA mathematical model, a spreadsheet program has been developed. The current version of the program works on personal computers with the software Quattro Pro for Windows installed. Another version for MS Excel could be available fairly soon. The program has a menu driven structure as shown in Fig. 6 so that the maneuver in the program is very easy. In the main menu, you are given options for three-entry or four-entry longwall chain pillar system. Then the chosen subprogram will be automatically loaded and its menu shown.

In the input session as shown in Fig. 7, the only required input data for the program are the overburden depth, panel width, entry width, pillar width, the spacing of cross-cuts and the coal in-situ strength. Then the program automatically implements all the computations involved such as the determination of various stress intensity factors, depth of yield

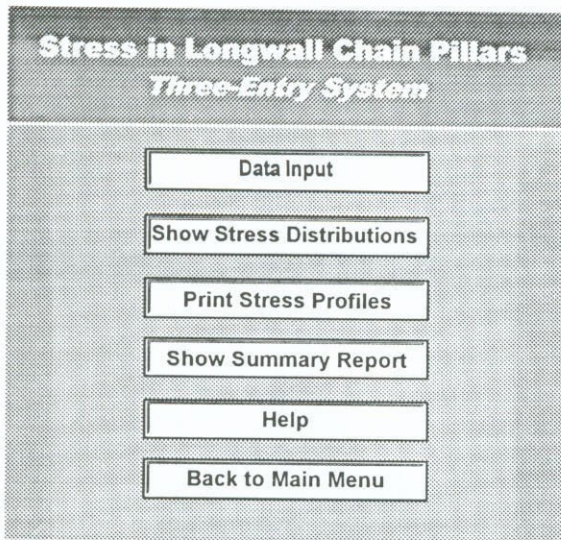


Fig. 6 Sub-Menu for the Three-Entry Chain Pillar System

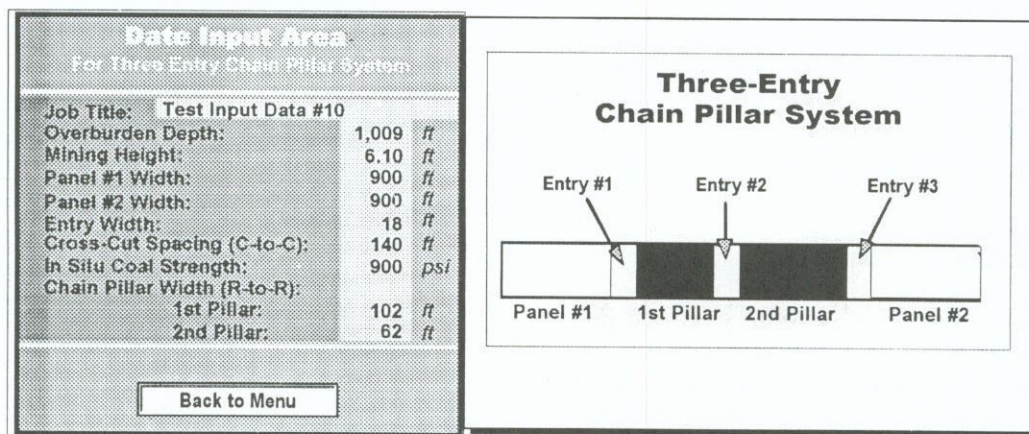


Fig. 7 Data Input Screen for Three-Entry Chain Pillar System

zones, superposition of the stress functions, etc. The resulting stress distribution profiles for the three mining stages for each of the pillars and then for all the pillars will be presented subsequently. A summary table will also be generated. In the summary table, the original input data, the depths of the yield zones on both sides of each pillar, the average vertical stress and confined strength as well as the safety factor for each of pillars in the chain pillar system are listed.

MODEL VALIDATION

FMA Versus Numerical Modeling

The basis of the FMA, the Westergaard equation, assumes that a crack of zero thickness is contained within an infinite, homogeneous plate and subjected to a uniform biaxial stress field. These conditions match, to some degree, the conditions of a mine opening enclosed in massive strata. There are differences though. A mine opening has an actual height (thickness). The coal has different structural properties than

the surrounding rock mass. A coal mine's environment is under the influence of a graduated, non-uniform, biaxial stress field controlled by gravity. It is necessary to consider all these factors when justifying the use of the Westergaard equation to predict the stress near a mine opening.

Comparisons, using numerical models with varying levels of non-homogeneity, validate the ability of the FMA to predict stress distributions in underground mine structures. Included in this section are graphical excerpts from previous investigations (Kramer, 1996). These charts demonstrate the capacity of the FMA to predict stress distributions in rock with dissimilar structural properties and in structures containing multiple mine openings. It should be noted the characteristics of pillar yield zones are not considered in the simulations presented in the following three figures (Figs. 8 to 10).

Figure 8 shows the comparison between the FMA and FLAC (a popular software for solving geomechanical problems with finite difference method) simulation results with homogeneous properties. A very good agreement can be

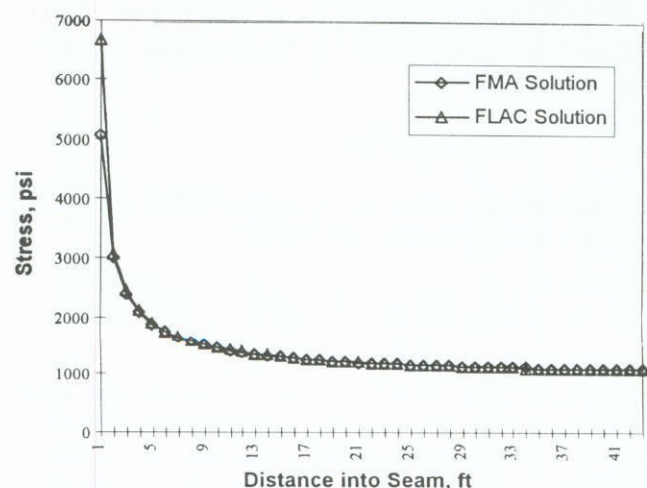


Fig. 8 Distribution of Vertical Stress in a Coal Seam Next to a Mine Opening - Comparison Between FMA and FLAC Solutions ($\sigma_i = 1,000$ psi and $a = 50$ ft)

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observed between these two approaches in the determined stress profiles.

Figure 9 plots the stress profiles through the center plane of a coal pillar from a series of FLAC simulations with a large range of the Young's moduli (Note: Unit Mpsi = 10^6 psi). It demonstrates that although the coal and surrounding strata have different mechanical properties the stress distributions in the pillar are not significantly different. This further indicates that the mechanical properties do not play a significant role in the pillar stress distribution as long as they are within a reasonable range, which also explains why the mechanical properties are not considered in Westergaard function.

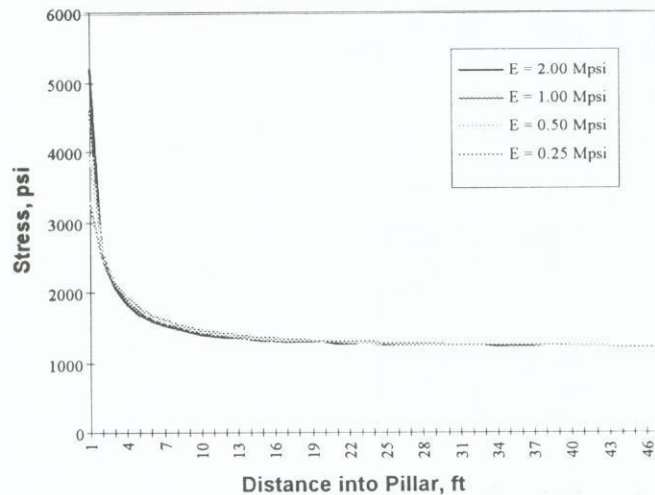


Fig. 9 Vertical Stress Profiles in a Coal Seam Simulated by FLAC with Varying Young's Moduli for the Coal

Figure 10 demonstrates the applicability of the principle of superposition in FMA. A mine structure with multiple openings has been simulated using both FMA and FLAC. The resulting stress distribution profiles are virtually the same.

An Application Example

Figure 11 shows the resulting stress profiles in a three-entry chain pillar system for the input data shown in Fig. 7 using the developed computer program. The overburden depth in this case is 1,009 ft while the panel width is 900 ft. The summary table for this case is presented in Fig. 12. The FMA simulation shows that both pillars are only partially yield in all three mining stages. The depth of the yield zones varies from about 9.5 ft to 15.5 ft. The average pillar stresses in the two pillars in the development stage is only slightly larger than the in-situ vertical stress (about 1,110 psi) and the overall safety factor of the chain pillar system at that time is about 1.4. Mining of the first adjacent panel nearly doubles the peak stress in pillar No. 1 on the first panel side. The overall safety factor of the chain pillar system decreases to 1.24. The completion of the second longwall panel mainly affected the stress distribution in pillar No. 2. The overall safety factor is further reduced to 1.16.

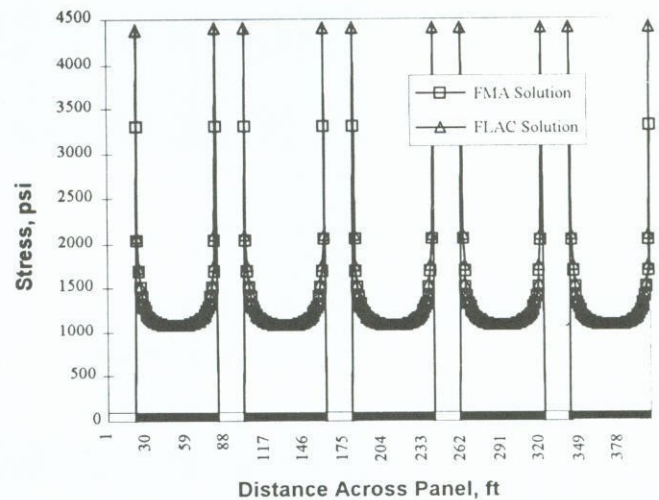


Fig. 10 Distribution of Vertical Stress in Coal Pillars for a Multiple-Entry Panel - Comparison Between FMA and FLAC Solutions

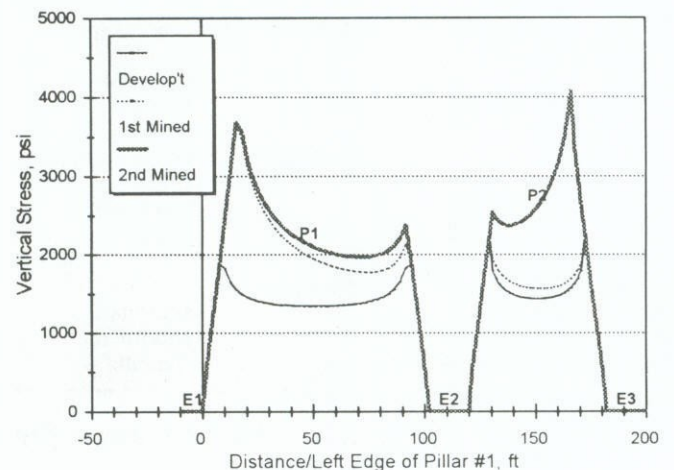


Fig. 11 Vertical Stress Profiles in the Chain Pillar System Using the FMA Approach at Three Mining Stages for the Input Data in Fig. 7

CONCLUSIONS

An fracture mechanics approach for determining the stress distribution in longwall chain pillar systems has been developed. This approach produces results as accurate as those by numerical analysis methods with no requirement on detailed mechanical properties of the materials involved. Furthermore, the developed FMA model is capable of considering the characteristics of the yield zones in the coal pillars, which is often ignored in ordinary numerical stress analyses. The computer program based on the FMA model is easy to use.

Summary Report				
Test Data Set #10				
Input Information				
Overburden Depth:	1009.0	ft		
Mining Height:	6.10	ft		
1st Panel Width:	900.0	ft		
2nd Panel Width:	900.0	ft		
Entry Width:	18.0	ft		
Cross-Cut Spacing :	140.0	ft		
In Situ Coal Strength:	900.0	psi		
Chain Pillar Width (R-to-R):				
1st Row:	102.0	ft		
2nd Row:	62.0	ft		
Output Information				
In-Situ Stress:	1109.9	psi		
Depth of Yield Zone				
1st Pillar Gob Side:	15.47	ft		
2nd Pillar Gob Side:	15.47	ft		
Beside Middle Entry:	9.53	ft		
Average Vertical Stress (psi)	Develop't	1st Mined	2nd Mined	
1st Pillar:	1355.3	1969.1	2092.5	
2nd Pillar:	1419.6	1505.4	2241.7	
Overall:	1379.6	1793.8	2148.9	
Average Confined Strength (psi)	Develop't	1st Mined	2nd Mined	
1st Pillar:	1906.0	2361.6	2453.3	
2nd Pillar:	1953.8	2017.5	2564.1	
Overall:	1924.1	2231.5	2495.2	
Safety Factor	Develop't	1st Mined	2nd Mined	
1st Pillar:	1.41	1.20	1.17	
2nd Pillar:	1.38	1.34	1.14	
Overall:	1.39	1.24	1.16	

Fig. 12 Summary Report Generated by the Program

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In Situ Strength Testing of Rocks with the Borehole Penetrometer

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ABSTRACT

One of the major difficulties in the proper characterization of rocks surrounding excavations is the lack of data. The existing method for strength determination requires core drilling for sample collection, which, when performed from the surface is costly, and when performed underground, in addition to its cost, interferes with the mine operations. Handling samples tested in the laboratory is troublesome and also influences the obtained results. Over several years, the author has been interested in developing a method of in situ strength determination that could rapidly generate strength data at low cost without a negative effect on mining activities. The recently developed borehole penetrometer is an improved version of the initial design (3 inches in diameter), which was used mainly for strength determination of coal pillars. The new penetrometer has been designed to work in 1.5 inch diameter holes. Such holes can be drilled at low cost with either hand-held drilling machines or roof bolters, eliminating limitations of the previous system and opening the possibility of broad application. Strength testing of roof rock around longwall pillars is an example of an obvious benefit when designing secondary roof support. In the paper are given the principles of work of the apparatus and examples of the obtained strength data.

INTRODUCTION

In recent decades a rapid progress in data processing and information technologies can be observed. This has been paralleled with the steady improvement of computational methods, taking advantage of progress in computer software and hardware. At the same time, funding for research conducted in mines has been steadily decreasing, resulting in a smaller amount of available data, which describes the actual properties of rocks and thus characterizes present mining conditions. Modern, highly productive mining techniques have been developed in the industry. They employ systems that

have high capital costs, and due to their high output, should be run without interruption.

It is apparent that the development in "tools of the trade" has become uneven. The generation of basic data lags behind the good analytical methods in engineering and mining systems employed in production. In strata control, decisions concerning roof support are made more by experience than by actual engineering design because of the lack of data. The exploration holes are drilled mainly for proving reserves, and then cores can also be geologically described, and geotechnical tests can be conducted. However, the assumed conditions based on interpolation between the core data may not be sufficient in the geologic environment, which varies over distances smaller than the distance between the holes. For instance, presently designed longwall panels are of large dimensions, and thus the probability of changing geologic and hence, mining conditions over distances well in excess of ten thousand feet is quite high. In spite of this, it is a common practice that the support system is chosen for one type of condition and then is applied around the panel. Consequently, a successful support system needs to satisfy the worst conditions, while outside of the weak area it may become overly conservative and lead to unnecessary expenditures.

Stability in longwall gate entries, where loading conditions change when the development loads to high abutment loads and then decrease when active loading travels away with the face line, is probably the most challenging situation in underground mining. Ensuring stability through effective support design requires good knowledge of geologic composition of the immediate and main roof, and strength parameters of the rock strata. A mining situation with variable geologic conditions also requires good characterization of the rock mass parameters. Another mining situation where stability becomes a problem is encountered if there is an interaction of close-by seams. Barrier and other remnant pillars are often encountered in an earlier mined-out seam, creating zones of increased load above and below them. High

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horizontal stress conditions may also negatively influence stability of excavations. In all these situations where the stress concentration around excavations are high, a good knowledge of geotechnic parameters of rock is of importance.

Recognizing the limitations in obtaining geotechnical data by conventional methods, the author has worked on development of an in situ method for determination of rock and coal strength in drill holes. These efforts have resulted in a highly successful method called penetrometric determination of coal strength. Horizontal 3 inch holes were drilled into the coal rib at different heights, to obtain good representation of the seam strength. An indenter, hydraulically driven, loads the wall of the hole until failure of the material under the indenter occurs. The same penetrometer has been used in testing roof and floor strength in coal, limestone, and metal mines. Even though this method was a great advancement in comparison to the conventional testing of rock in terms of data generation, it still had limitations relating to the drilling of 3 inch holes. In limestone mining, drilling of such a diameter does not create difficulties, because the production drills can drill holes of this diameter. However, in a coal mine, such drilling requires application of a special drill that complicates the whole testing procedure and raises its cost. Therefore, it was essential to reduce the penetrometer size so that it could be used in the 1.5 inch holes. Such holes can be drilled with the production bolting machines and then can be used as test holes for borescoping observations and for the in situ penetrometer testing. The reduction of the penetrometer size, while maintaining its required load capacity on an indenter, has been quite a challenging design exercise.

DESIGN PRINCIPLES AND OPERATION OF THE PENETROMETER

The main part of the penetrometer is the indenter, which actually loads a rock until its failure. There are three indenter sizes. The largest has an area of 1 cm sq., used for strength determination in coal and other materials within a similar strength. The next size down is 0.5 cm² and is appropriate for rocks with an intermediate strength such as shales, siltstones, and weaker sandstones. The smallest indenter has an area of 19.63 mm² (5 mm diameter) and is used for strong rocks such as sandstone, limestone, and igneous rocks. The different sizes of indenters are necessary to generate a unit load required for material failure, within the available maximum hydraulic pressure of the penetrometer. From experience with the previous larger model, it was found that reaching 10,000 psi pressure with the hand pump requires hard physical labor. Therefore, in the new model, the maximum pressure has been set at half of the previous one, equal to 5000 psi. Consequently, an area of the piston had to be increased from 1 to 2 inches square. This, together with the space constraints for the small diameter penetrometer's body of 1.25 inch, required application of the elongated, oval hydraulic jack, which propels the indenter. Such a jack has been made by using precision

machining. The cylinder of the jack is made within the cylindrical body of the penetrometer (Figure 1).

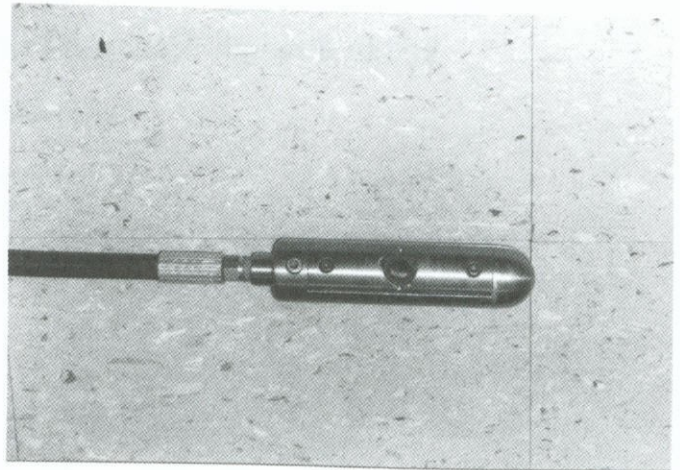


Figure 1. Penetrometer head

Other parts of a unit include a hydraulic hose 25 feet long, a hydraulic pump, and a clamp assembly that helps in retraction of the indenter after the test. Hydraulic parts have several points where the air can be bled from the system.

The penetrometer also has a provision for obtaining recorded data concerning pressure and indenter penetration, by using appropriate indicators and a data acquisition system. The small dimensions of the penetrometer ruled out the use of an LVDT for indicating movement of the indenter inside the body of the unit, as was done in the previous, larger model. Therefore, this measurement is made outside the test hole. For that purpose, a double action hydraulic cylinder has been incorporated in the hydraulic system. The piston in the cylinder can be manually moved (through the lever-clamp) back to help displace the oil from the hydraulic jack back to the pump after the test, by creating a negative pressure. This makes it easier for the penetrometer piston to move down the cylinder to its initial position. The cylinder to a degree damps the hydraulic pulse from the system pressurized by a piston-type, hand pump. The position of the indenter is monitored by recording the movement of the metal plate attached to the piston of a double action cylinder with the LVDT, which is mounted on a base plate (Figure 2). The movement of the plate is calibrated against the movement of the indenter. The hydraulic pressure during the load increases and, at failure, is recorded by the pressure transducer mounted on a T-connector of a hydraulic line, which pressurizes the jack of the penetrometer.

The testing can be performed in two modes. The first one is conducted when only strength is determined by observation of the maximum pressure at the hydraulic gage at the point of rock (coal) failure. In brittle materials such as coal and in harder rocks such as siltstone, sandstone, and limestone, the failure is associated with an audible sound. This is a quick

and simple determination of strength. It takes about a minute to complete one test. The other mode is performed when the data acquisition system is hooked on to record both the position of the indenter, which is a measure of the deformation of rock, and a corresponding pressure from the pressure transducer. The corresponding, recorded data can be paired by the

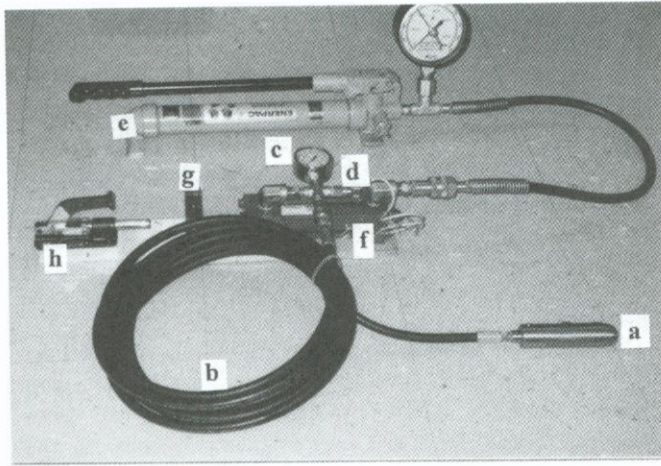


Figure 2. View of equipment ready for penetrometer testing (strength only-without data acquisition system for recording pressure and penetration). a - penetrometer head, b - hydraulic hose, c - pressure gage, d - pressure transducer, e - hydraulic pump, f - double action cylinder, g - moving plate for LVDT measurements, h - manual lever bracket for pushing back piston in cylinder

computer software, Pronto, allowing for viewing and drawing of graphs; stress - deformation. A deformation modulus can be calculated from such graphs, and the nature of a rock material strain-stress characteristic can be studied.

If it is desired to determine the yielding characteristics of the tested material, a longer term loading condition can be created, and appropriate data gathered. The penetrometer can work either as a relaxometer (recording pressure drops as a result of the yielding of rock), or as a yield meter where the penetration of an indenter is measured under constant pressure. The precision of these measurements might be affected by the leakage of hydraulic valves; therefore, a good check of the system (bleeding air from the system) reliability would be necessary before such application.

The testing procedure is as follows. At the test site a decision concerning the direction and position of drill holes has to be made. Then the holes are drilled. A mine bolting machine with a swivelling drilling head is most useful, because it can drill in any direction. However, when a conventional (vertical drilling) machine is available, it can drill only roof holes. Holes in another direction, such as into the rib or into the mine floor, have to be drilled by a portable drilling machine. Holes should be straight and, therefore, use of a stabilizer on a drill steel is recommended. When drilling is

complete, the hole should be flushed with water to evacuate dust and rock particles. Horizontal holes should be drilled at a slight incline so that water can run out. It is recommended to proceed with testing after the hole is dry, especially in shaly rocks. It is also recommended that the testing hole be borescoped before the penetrometric test. It should be performed before the rock dries out, because when wet, rock color is more visible, helping in strata determination. Borescoping allows location of the cracks and fractures in rock. It is important to know their location not only for strata control considerations, but also to avoid setting the penetrometer test point in close vicinity to or over the crack. The result of such a test will not determine an intact rock strength, and additionally, the indenter may get stuck in a crack, making it impossible to retrieve.

Tests can start at the end or at the beginning of a hole. In coal pillars, the direction of testing is vertical (with the indenter up and down). In the mine roof and floor, the

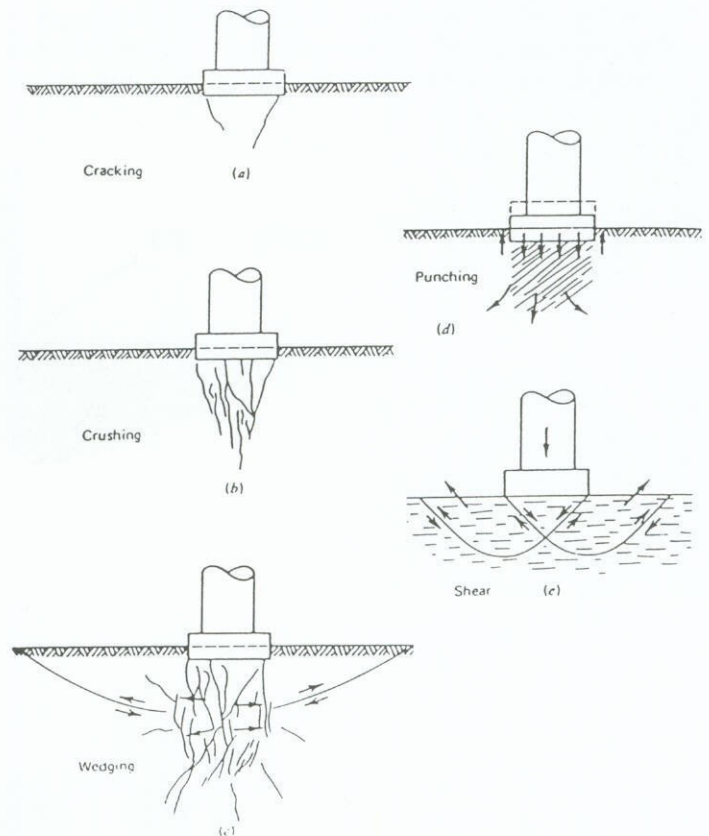


Figure 3. Failure modes of penetration on rock (Goodman, 1972)

direction of the indenter does not matter. A test can be repeated at a given position (depth) of the penetrometer in the hole by rotating it either 180 or 120 degrees. By doing this, a mean strength value in a given rock band can be determined.

Fractures in coal called cleat are quite frequent and often impossible to avoid. As was previously mentioned, when the indenter is loading over a fracture (cleat) or close to one, the results can be affected. Such values would be substantially lower than other results. Similarly, when the indenter hits some type of a hard geode or concretion, the strength result can be much higher and should be rejected. The statistical rules should be applied in defining such results called "outliers." For example, in sets of about 20 points, which are typically obtained in horizontal holes (in one stratum of rock/coal material), the outliers to be rejected are the results which are 17% and more, above or below the mean value from a given set of results.

INDENTER INTERACTION WITH ROCK

Figure 3 shows the modes of penetration into brittle, nonporous rock (2) and (3). Assuming the rock mass is relatively free of fractures, loading takes place in a precise form, depending on the shape and deformability of an indenter. After attaining a load that initiates a crack, further loading extends the cracks (Figure 3a), and at still higher loads, they

coalesce and interfere. Eventually, the cracks separate into slivers and wedges that buckle and crush under additional increments of the load (Figure 3b). Due to dilatancy, the bulb of cracked and crushed rock under the loaded area expands outward, eventually generating a radial network of cracks, one of which may finally propagate to the free surface (Figure 3c).

In practice, rock masses undergo additional permanent deformation due to the closing of the fractures, cracks, and pores. In coal fields with cemented sedimentary rocks, a mode of failure termed punching is shown in Figure 3d, as in soft shales and mudstones. Since the penetrometer indenter is relatively small compared to the rock mass being tested, the existence of the cracks and fracture around the indenter can be ignored. Therefore, the most common mode of failure in this testing is shear failure (Figure 3e). It is worth observing that shearing is a principal mode of failure around the mine excavations.

According to the bearing capacity theory, if a load applied by an indenter increases on the surface of the testing material, displacement takes place. The relation between a pressure of unit load and displacement depends on the properties of rock. Figure 4 shows that there are three pressure

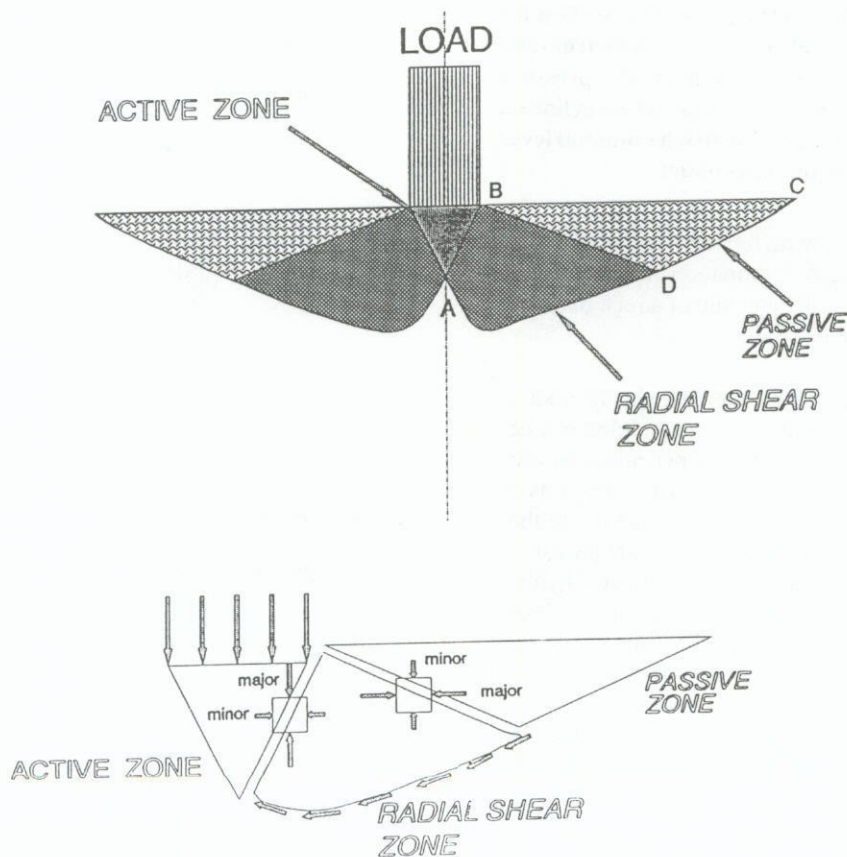


Figure 4. Stress analysis under penetrometer loading (Zhang, 1995)



zones at rock failure when a load is applied on the rock as described below. The rock is assumed to be in contact along the entire penetrated sides of the indenter, and no frictional forces exist at the indenter rock interface.

Active Pressure Zone

This is a cone shaped zone located directly underneath the loading indenter. The material in this zone will be uniformly yielding when the major principal stress exceeds its compressive strength. In practice, the evidence of rock failure in this zone is hard to identify during testing. For brittle rocks and coal, the following two pressure zones play more important roles in penetrometer testing.

Radial Shear Zone

This zone is located between the active and passive zones, and the angle between the two sides which is adjacent to the two zones is 90 degrees. Cohesion exists along the bottom line, which is shaped by the function of the log spiral. When the load applied from the active zone is large enough to overcome the cohesion, the major principal stress will be created in the passive zone.

Passive Pressure Zone

In this zone, the major principal stress is in a horizontal direction. While the stress increased to a certain level, the indenter rapidly penetrated the rock, making a small crater around the indenter tip. The state of failure is known as "general shear failure," and the failure control parameters are the cohesion and angle of internal friction of a tested material. The sudden increase in displacement and the decrease in the applied pressure may be identified at that critical point. The needle of a hydraulic gage jumps, and a distinct failure sound can be heard in coal and brittle rock, tested both in the laboratory and in the field. All those symptoms have been utilized for determining the rock failure load point.

Influence of Stress Concentration on Penetrometer Readings

Laboratory experiments have been carried out, where the one cubic foot sample with the hole drilled in it for the penetrometer testing was loaded with flat jacks in a triaxial reaction chamber. The resulting stress concentration on the perimeter of the hole did not have a noticeable influence on the penetrometer reading. The possible explanation might be that the indenter loading is much larger than the stresses and, therefore, the results are not affected.

INDENTER PRESSURE AND THE HYDRAULIC PRESSURE

The penetrometer indenter pressure P_i can be obtained

from a hydraulic pressure P_p , which is recorded by a data acquisition unit through a pressure transducer, or reading of the dial gage. The relation of those two parameters is given as:

$$P_i = kP_p$$

and thus $k = P_i/P_p$. The area of a penetrometer's piston is 2.15 in² (13.87094 cm²), while the indenters have three sizes, $I_1 = 0.155$ in² (1 cm²), $I_2 = 0.0775$ in² (0.5 cm²), and $I_3 = 0.03043$ in² (0.19635 cm²). Consequently, the k factors will be for the three sizes of indenters, 13.87, 27.74, and 70.65, respectively. With the maximum pressure of 5000 psi in the hydraulic system, the stress on the indenter tip will then reach for $I_1 = 69,350$ psi, $I_2 = 138,700$ psi, and $I_3 = 353,250$ psi.

Laboratory experiments have been carried out comparing results of the former (3 inch diameter) penetrometer testing with the present model of the apparatus. The samples of rock cores encapsulated in one cubic foot concrete blocks were exposed for testing by a diamond core drilling of a 3 inch hole. The tests were performed with the larger penetrometer as a part of a previous investigation. These same concrete blocks were used for tests with the new apparatus. An adapter has been made to hold the small penetrometer in a 3 inch diameter hole. The tests were run between the previous test locations with a larger unit along the hole drilled through the concrete block. The results obtained by both penetrometers came very close. This was expected because the size of the indenters are identical. The factor between the pressure recorded with the larger and smaller unit is equal to $1/2.15 = 0.465$, due to the larger area of the cylinder's cross section of the hydraulic jack in the smaller unit.

SUMMARY AND RECOMMENDATIONS

The penetrometric test is in essence a shear test of intact material, because the rock material tested is in the triaxial loading condition. The strength values depend on yield limit in compression, angle of internal friction, and cohesion of a tested material. All of these parameters also control the stability of excavations, and are more directly related to mining conditions than the uniaxial compressive strength, which is a broadly used parameter in engineering classifications of rock and other engineering calculations. In an actual mining situation, the relative comparison of strength of the rock strata surrounding excavations is of great importance, because it gives a base for particular decisions concerning stability and support, e.g., length of roof bolts. Such relative comparison can be obtained by application of the penetrometer with a fraction of the cost per test when compared with the conventional testing on samples in the laboratory. The low cost of testing can allow for more extensive data generation, giving engineers better defined conditions and, therefore, increasing reliability of the design. The correlation factors between the penetrometric test failure pressure and compressive strength of a given material has been determined (4). For coal, the factor

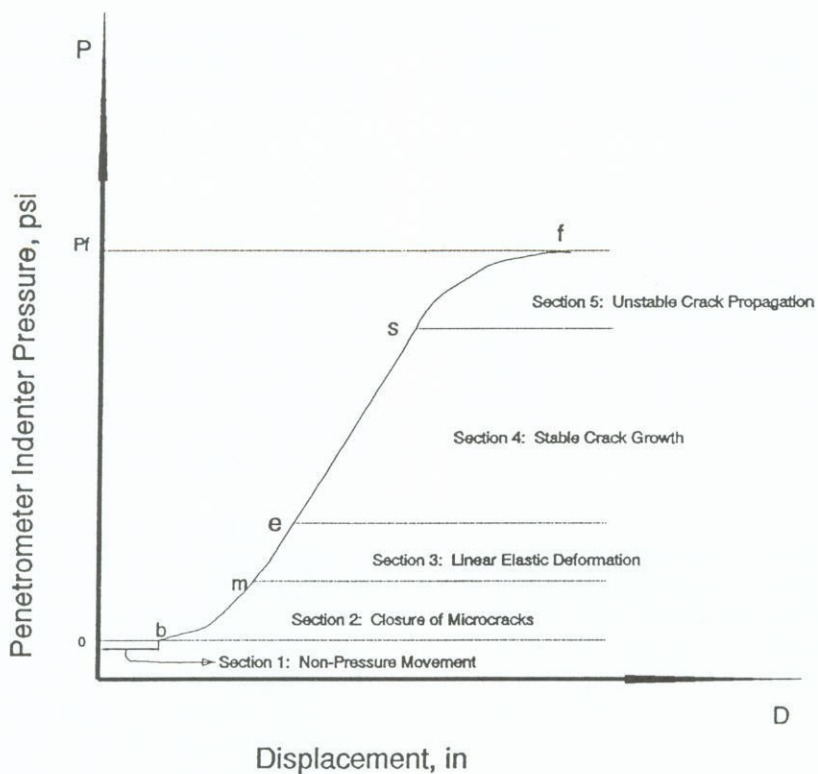


Figure 5. Typical curve from penetrometer testing

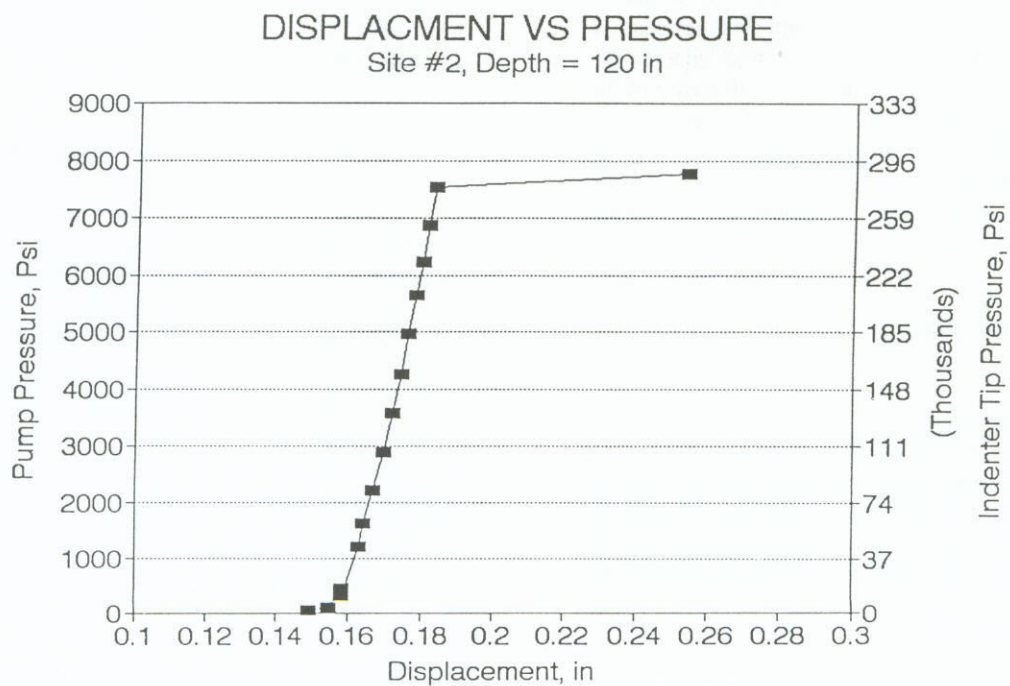


Figure 6. Diagram of displacement versus pump pressure for hard rock (quartzite)



was 1.25 for the 3 inch apparatus, and for the small unit it was $1.25/0.465 = 2.688$. It is worth mentioning that the loading in both the penetrometer test (holes drilled into the coal rib) and the laboratory test is perpendicular to bedding. The data base for such tests is substantial, and the correlation factor mentioned above is well defined.

When testing with the penetrometer in holes drilled perpendicular to bedding, which is the case in coal mines where holes are drilled in the mine roof and floor, the indenter loads rock parallel to bedding.

The correlation factors for the rock material between the penetrometric tests parallel to bedding and the uniaxial compressive strength on core samples that have been tested in the laboratory perpendicular to bedding, have a larger scatter of results. It is perceived that such a correlation will not be as good as for the above named tests, where both were performed perpendicular to bedding. However, with the increase of the penetrometric strength determination data base, particular correlations will certainly be improved. It is emphasized here that the penetrometric test stands on its own merit and does not need to be compared to the uniaxial compressive strength test.

The complete apparatus with the pump can be carried in a shell type suitcase (Samsonite wagon) that has been found to be practical in mining situations. Its portability makes possible testing over larger areas, e.g., along longwall gate entries, where roof conditions can be determined around the panel.

By monitoring with a data acquisition system the penetration of the indenter and the corresponding hydraulic pressure (load), deformation modulae can be evaluated for the particular strata, (Figures 5 and 6). Such data can be utilized for numerical modeling.

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Performance and Safety Considerations of Hydraulic Support Systems

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ABSTRACT

Hydraulic cylinders are used in several roof support systems, such as longwall shields and mobile roof supports, to provide critical ground control. Many state-of-the-art hydraulic support systems operate at pressures as high as 7,000 psi to provide the necessary support capacities. Pressure in the upper stages of multi-stage cylinders can be intensified by a factor of 2 or 3 providing pressures of 20,000 psi or greater. A proper understanding of the operation of these high pressure systems is necessary to safely maintain and repair these support systems. Likewise, the performance characteristics need to be fully understood to evaluate the impact of design parameters on the capability of the support to maintain effective ground control. This paper examines the basic operating principles of state-of-the-art hydraulic cylinders and discusses relative issues pertaining to: (1) setting loads, (2) support stiffness, (3) yielding behavior, (4) errors in assessing support loading, and (5) hydraulic failure mechanisms and how to detect them.

INTRODUCTION

Hydraulic support systems are used frequently in underground mining to provide critical ground control. Some examples of their applications include: (1) shields which provide ground support along the working face in longwall mining, (2) mobile roof supports which protect miners from caving strata during retreat mining pillar extraction operations, and (3) temporary support systems such as those used on bolting machines which are used to support the mine roof while more permanent support can be installed.

Hydraulic roof support systems have two major advantages over other types of supports. First, they have the flexibility to extend and retract, thereby allowing the same support to be installed in a wide-range of mining heights. Second, hydraulic supports can be pressurized to provide active roof support. By actively setting the support against the mine roof and floor, hydraulic support systems close any gaps within the immediate

roof structure, creating a laminated roof beam that is much stronger and more stable. In contrast, passive roof supports such as wood cribs and timber posts, develop load carrying capacity only through the convergence of the mine roof and floor. The downward movement of the roof weakens and eventually causes failure of the roof beam. In addition, an active hydraulic support creates a stress concentration at the support that acts to break off a roof fall, whereas there is a much greater probability of a roof fall riding past the support into the working area with a passive support system.

On the surface, it appears that the performance of hydraulic supports is straight forward, and that little consideration needs to be given to the design, selection, and operation of hydraulic roof support systems. Upon closer examination, it can be seen that the performance of hydraulic support systems is dependent upon several factors that can influence ground control capabilities and safe operation of these popular support systems. The purpose of this paper is to examine the characteristics of hydraulic roof support systems that impact ground control and operator safety.

HYDRAULIC CYLINDER DESIGN AND BASIC OPERATING PRINCIPLES

The support capacity of all hydraulic roof supports is provided by one or more hydraulic cylinders. The remaining structure in these supports is simply to transfer load to the hydraulic cylinders to provide the necessary work to support the weight of the mine roof and control the movement of the mine roof and floor.

The basic operation of a hydraulic support can be described as follows (see figure 1). A hydraulic power supply pumps fluid into a cylinder(s) on the roof support structure. The fluid acts against a piston, causing the piston and attached steel rod to displace outward. This displacement will continue with very little hydraulic pressure until the support is set against the mine roof and floor. The hydraulic pressure will then increase rapidly until the full pump pressure is reached. After the pump supply is

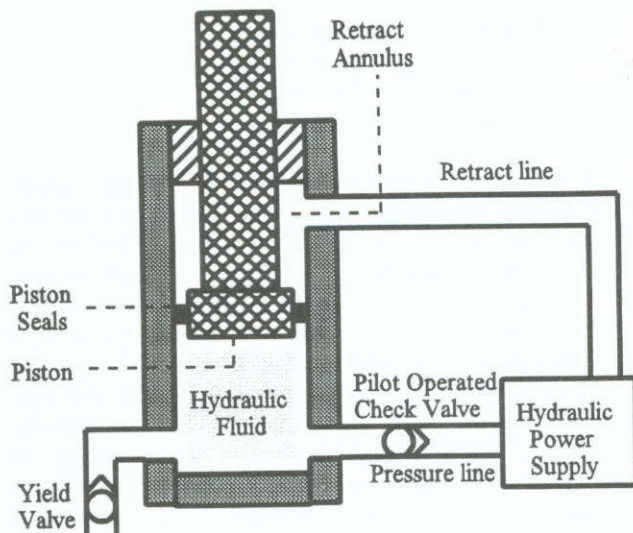


Figure 1.-Basic components of a hydraulic roof support.

turned off or isolated from the support, this pressurized hydraulic fluid is trapped inside the cylinder by a pilot-operated check valve. The pressure of the hydraulic fluid inside the cylinder is then intensified in proportion to any increase in roof loading. A yield valve limits the maximum pressure inside the cylinder to prevent excessive loading that would damage the cylinder. Most state-of-the-art hydraulic cylinders used in modern mine roof support structures are double acting cylinders, meaning that they are hydraulically powered to both extend and retract (see figure 1). This requirement stems primarily from longwall shield supports, where powered lowering of the shields is required to minimize the advance cycle time. Powered lowering of the support is achieved by pumping fluid inside the retract annulus (see figure 1), while at the same time applying pilot pressure to open the check valve to allow fluid from inside the main cylinder cavity that supports the weight of the mine roof to escape back to the hydraulic power supply tank.

The previous example described the basic operation of a single-stage cylinder, where extension and retraction is provided by one stage. Many supports, including longwall shields and mobile roof supports, utilize multi-stage hydraulic cylinders. Figure 2 depicts a two-stage hydraulic cylinder used on a state-of-the-art longwall shield, and figure 3 depicts a three-stage hydraulic cylinder utilized on a state-of-the-art mobile roof support. For discussion, the stages are referenced starting with the largest diameter as stage 1, and each successively smaller diameter as stage 2, stage 3, etc., or in reference to their location with designations of bottom (stage 1), middle when applicable, and top stage. Incorporating more stages into a leg cylinder design generally allows for a lower collapsed height, combined with an equal or greater maximum working height to provide operation in a wider range of mining heights than fewer stages would permit.

The basic operating principles of these multi-stage designs are the same as the single stage hydraulic cylinder previously described, but there are some noteworthy differences relative to how the individual stages perform.

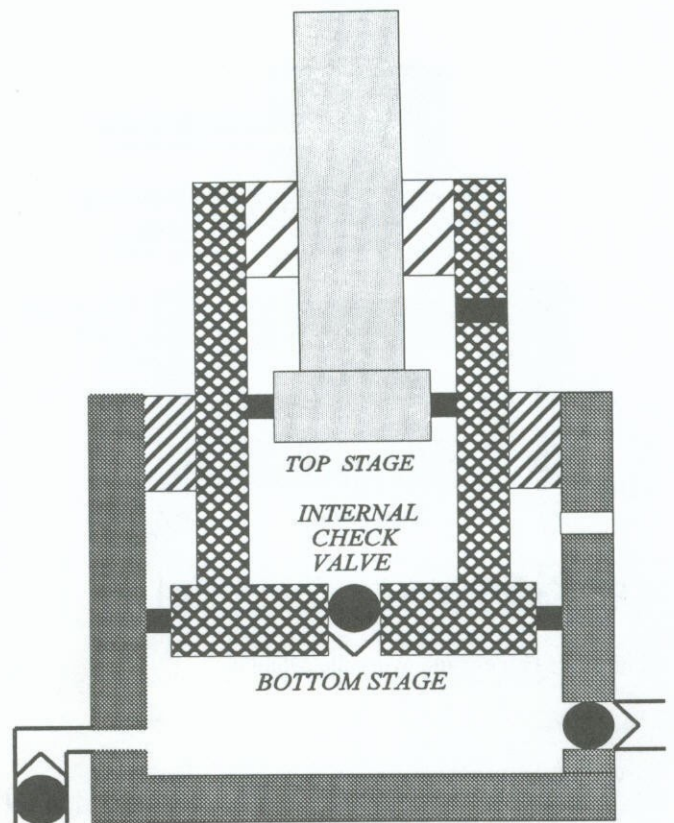


Figure 2.-Two-stage hydraulic cylinder.

1. In multi-stage cylinders used on most mining support systems, the largest (diameter) stage extends and retracts first, followed in succession by the stage with next smallest diameter. For example, in a three-stage leg cylinder, the bottom stage would extend first, followed by the middle stage, and then the top stage. Each stage will extend until it is fully stroked before the next stage will begin to extend.
2. A check valve is incorporated into the piston(s) of all but the top stage. For example, in a two-stage leg cylinder, a check valve would be installed in the bottom stage piston. This check valve functions to allow hydraulic fluid to flow from the bottom stage to the top stage to cause the top stage to extend during the setting operation whenever the bottom stage is fully extended. It also isolates the bottom stage from the top stage to allow the hydraulic fluid to be intensified in the top stage once the support is actively set against the mine roof and floor. This is necessary to allow the top stage to carry the same load as the bottom stage. A three-stage leg cylinder would incorporate check valves in both the bottom stage and middle stage pistons as illustrated in figure 3.

How the staging functions to provide the necessary support capacity is essential to understanding the operation of multi-stage leg cylinders. For proper operation of the support cylinder to occur, force equilibrium must be satisfied for each stage, meaning each stage must carry the same load. This requires different hydraulic pressures in each stage, since the area of each stage is different. Mathematically, this requirement is expressed by equation 1.

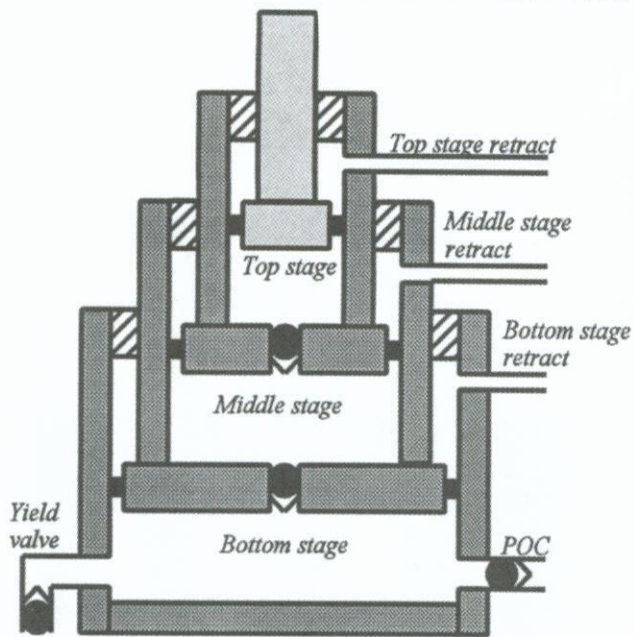


Figure 3.-Three-stage hydraulic cylinder.

$$A_1 * P_1 = A_2 * P_2 = A_n * P_n \quad (1)$$

where A_1 = area of bottom stage, sq. in;
 P_1 = pressure in bottom stage, psi;
 A_2 = area of second stage, sq. in;
 P_2 = pressure in second stage, psi;
 A_n = area of n^{th} stage, sq. in;
 P_n = pressure in n^{th} stage, psi.

Using a two-stage cylinder as an example, if the top stage area (A_2) is one half (50 pct) that of the bottom stage area (A_1), the pressure in the top stage (P_2) will be twice that of the hydraulic fluid in the bottom stage (P_1).

$$P_2 = \frac{A_1}{A_2} * P_1 \quad (2)$$

For a three-stage leg cylinder,

$$P_2 = \frac{A_1}{A_2} * P_1 \quad (3)$$

$$P_3 = \frac{A_2}{A_3} * P_2 \quad (4)$$

where P_1 = pressure in bottom stage, psi;
 P_2 = pressure in middle stage, psi;

P_3 = pressure in top stage, psi;
 A_1 = area of bottom stage, sq. in;
 A_2 = area of middle stage, sq. in;
 A_3 = area of top stage, sq. in.

The extension of specific stages on a hydraulic support system, such as a longwall shield or mobile roof support, at a particular point in time (cycles of operation) depends on the history of the operating heights of the support. When the support is initially raised from a fully collapsed position, the bottom stage will extend first. If the mining height is greater than the stroke of the bottom stage, the second stage will extend until roof contact is made, or in the case of a three-stage cylinder, the third stage will extend after the second stage is fully stroked to achieve the roof contact. On subsequent cycles, the upper stage(s) will remain at the initial extension(s) until: (a) the mining height is increased beyond that of the initial cycle, or (b) the support is sufficiently lowered such that the bottom stage is fully collapsed. Assuming the support is not lowered to the point where the bottom stage is fully collapsed, the upper stage will extend beyond the initial extension only when the mining height is increased. In essence, the bottom stage will be fully stroked when: (a) the support is initially raised from a collapsed position, and (b) on the mining cycle that establishes a **new maximum operating height**, whenever the support height is higher than it has been on all previous mining cycles. On all other cycles, the bottom stage extension will reflect changes in mining height relative to the initial mining height and the maximum operating height at which the support was utilized.

A two-stage hydraulic cylinder, as shown in figure 2, is used as an example to illustrate these concepts. As shown in figure 4, the initial setting of the support causes the bottom stage to be fully extended (24-in stroke) and a 12-in (partial) extension of the top stage. If the roof-to-floor mining height is reduced on second operating cycle by 3 in, the bottom stage extension now will be 21 in while the top stage extension will remain at 12 in. Now if the mining height is increased by 6 in from that of the second cycle, the bottom stage will again be fully extended (24-in stroke) and the second stage extension will increase to 15 in on the third cycle. This exercise can continue as shown in figure 4. As shown in this exercise, the top stage extension on any cycle will equal the initial extension plus the incremental increase in mining

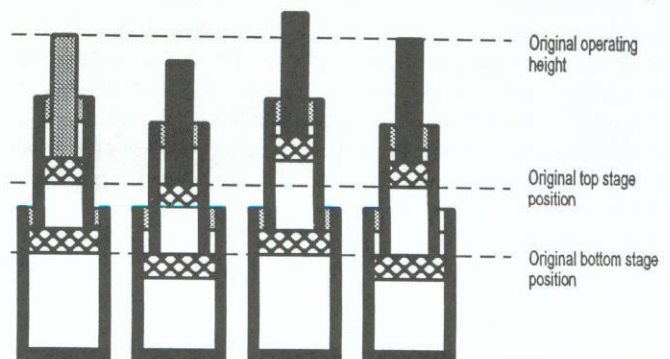


Figure 4.-Stage extensions produced by changes in operating height.

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height beyond the initial mining height. The top stage extension will never be less than the initial extension unless the support is lowered to the point where the bottom stage is fully retracted. The bottom stage extension will fluctuate with changes in mining height on each cycle, and will be fully stroked whenever a new maximum operating height is attained.

ACTIVE (SETTING) LOADS

Hydraulic support systems can be actively set against the mine roof and floor by pressurizing the hydraulic cylinders. The force that is exerted against the mine roof and floor is dependent upon the size of the cylinder and the hydraulic (pump) pressure. In multi-stage cylinders, the setting force can be determined by measuring the pressure in any stage and multiplying that pressure by the (piston) area of that stage. It is important to remember that the pressure in all but the bottom stage is intensified during the setting operation, and only the bottom stage will be at pump pressure once the support is set (assuming none of the stages are at fully stroked). In most multi-stage hydraulic cylinders, such as those used on longwall shields and mobile roof supports, the hydraulic pressure is measured only in the bottom stage, hence the pump pressure times the bottom stage area should be used to calculate the setting force. The total setting force for the support is simply the sum of each individual hydraulic cylinder forces.

If one or more stages is fully stroked, the setting force will be diminished. The stage that is fully extended no longer transfers all of its force to the mine roof and floor, instead part of it is consumed by tensioning the cylinder casing as the piston is forced against the mechanical stops that limit its travel. And, the pressure in the stage above the fully extended stage will not be intensified, but instead will be at pump pressure. The setting force will always equal the pump pressure times the area of the largest diameter stage which is not fully extended. Hence, in a two-stage cylinder, the setting force will equal the pump pressure times the second stage area whenever the bottom stage is fully extended. In a three-stage cylinder, the setting force will equal the pump pressure times the second stage area whenever the bottom stage is fully extended, and the setting force will equal the pump pressure times the third stage area whenever both the bottom and middle stages are both fully extended.

The reductions in setting force due to full extension of stages can be very large. Table 1 below documents the actual reductions in setting force for a state-of-the-art, 700-ton, two-stage, longwall shield and a 600-ton, three-stage, mobile roof support.

Table 1. Reduction in setting force due to stage extensions.

STAGE CONDITION	ACHIEVED SETTING FORCE, PCT OF FULL SETTING FORCE	
	700-ton Longwall Shield	600-ton Mobile Roof Support
No stage fully extended	100	100
Bottom stage fully extended	54	55
Bottom and middle stage fully extended	N/A	30

Recalling the operation of the hydraulic cylinder previously described, the setting force will be diminished whenever the bottom stage is fully extended, which occurs on the operational cycle where a new maximum operating height is attained. The setting force will be restored to its full capability whenever the support is operated at a height less than the highest operating height on any previous operating cycle.

SUPPORT STIFFNESS

Stiffness is a measure of how quickly the support develops its load carrying capacity in response to roof-to-floor convergence. The stiffness of hydraulic support systems is examined in terms of the parameters that affect the support stiffness and the impact of support stiffness on ground control.

Discussion of General Stiffness Principles

The stiffness of a hydraulic cylinder can be determined theoretically using the following principles. Full-scale testing of longwall shields and mobile roof supports in NIOSH's Mine Roof Simulator load frame verify that these principles are valid.

The various stages of a multiple stage leg cylinder act in series to provide an equivalent stiffness as described by equation 5.

$$\frac{1}{K_{eq}} = \frac{1}{K_1} + \frac{1}{K_2} + \dots + \frac{1}{K_n} \quad (5)$$

where K_{eq} = equivalent stiffness of the leg cylinder
 K_1 = stiffness of stage 1
 K_2 = stiffness of stage 2
 K_n = stiffness of stage n

For a two-stage and three-stage leg cylinder this equation can be transformed as follows:

$$K_{Two\ stage} = \frac{K_1 * K_2}{K_1 + K_2} \quad (6)$$



$$K_{\text{Three stage}} = \frac{K_1 * K_2 * K_3}{K_1 * K_2 + K_1 * K_3 + K_2 * K_3} \quad (7)$$

The stiffness of the individual stages can be determined using the bulk modulus of the oil as shown in equation 8. The manufacturer of the oil can provide accurate data on the bulk modulus. A value of 320,000 psi is recommended in the absence of manufacturer data. This value can also be used for water-oil emulsions such as those used in longwall shields.

$$K_n = \frac{A * \beta}{L} \quad (8)$$

where K_n = stiffness of stage n, lbs/in
 A = area of stage n, in²
 β = bulk modulus of the oil, psi
 L = stage extension, in

Since the cylinder structure (i.e., casing) is not perfectly rigid, the stiffness of the cylinder structure should also be considered in the calculation of the overall leg cylinder stiffness. The cylinder casing can also be considered to act in series with the hydraulic stiffness to provide a total stiffness as shown in equation 9. The stiffness of the cylinder casing will vary depending on the design of this steel structure. A study of mobile roof support hydraulic cylinders revealed cylinder structural stiffness of 2,300 kips/in for a two-stage cylinder and 3,200 kips/in for a three-stage cylinder (kip = 1,000 lbs).

$$K_{\text{System}} = \frac{K_{\text{Hydraulic}} * K_{\text{Cylinder}}}{K_{\text{Hydraulic}} + K_{\text{Cylinder}}} \quad (9)$$

where K_{system} = Overall leg cylinder stiffness, lbs/in
 $K_{\text{hydraulic}}$ = Hydraulic (oil component) stiffness, lbs/in
 K_{cylinder} = Cylinder structural stiffness, lbs/in

Key Points in Determining Support Stiffness

A close examination of these stiffness principles reveals several key points regarding the determination of the stiffness of hydraulic support systems.

1. The stiffness of a hydraulic roof support is not significantly affected by the setting pressure.
2. The stiffness of a hydraulic roof support is a nonlinear function of operating height, decreasing exponentially as the height increases.
3. The diameter of the hydraulic cylinder primarily controls the stiffness of the hydraulic support with increased stiffness as the diameter of the cylinder increases. However, an important point to remember in multi-stage cylinders is that the smallest (diameter) stage will have the least stiffness and therefore largely control the overall stiffness of the support. This is why a three-stage hydraulic cylinder will always be less stiff than a two-stage hydraulic cylinder with the same size (diameter) bottom stage.

4. The stiffness of a multi-stage hydraulic cylinder is also dependent on the extension of the respective stages. The stiffness of each stage decreases exponentially as the stage extension increases. The top stage will produce the most influence on the support stiffness. The stiffness of the support increases dramatically when the top stage is not extended at all or only very slightly extended. In this condition, the support can be as much as 10 times stiffer than when the all stages are extended greater than 75 pct of their stroke. A minimal extension on the top stage of 3 in reduced the stiffness of a mobile roof support two-stage leg cylinder by 45 pct compared to no extension of the top stage.
5. In supports using multi-stage hydraulic cylinders, the same overall support stiffness can occur at two different operating heights, depending on the combination of stage extensions. In other words, 5 in of bottom stage extension and 15 in of top stage extension will not produce the same stiffness as 15 in of bottom stage extension and 5 in of top stage extension. Figure 5 depicts a stiffness envelope for a 150-ton two-stage hydraulic cylinder used on a 600-ton mobile roof support. For this particular cylinder, a stiffness of 400 kips/in occurs at operating heights of 64 and 75 in. Likewise, at a 64 in operating height, the stiffness can vary from 400 to 620 kips. A theoretical stiffness envelope can be determined by calculating the individual stage stiffness (eq. 8) and the system stiffness (eq. 9) at different stage extensions.

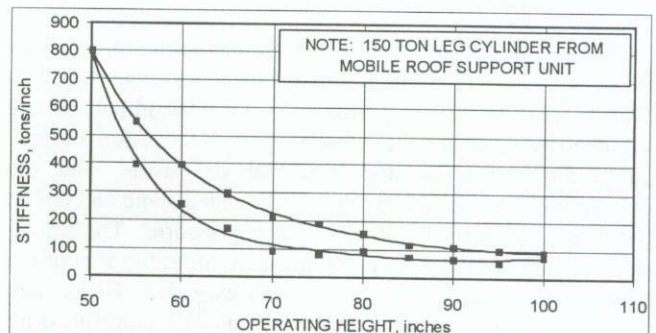


Figure 5.-Stiffness envelope for a two-stage 150-ton hydraulic mobile roof support cylinder.

6. The total displacement in a hydraulic cylinder will equal that of the roof-to-floor convergence, but this displacement will be distributed differently among the stages in proportion to the stiffness of the individual stage. The stiffest stage will move (displace) less than the softest stage. This means that most of the convergence will occur in the top stage of multi-stage hydraulic cylinders.

Impact of Stiffness Issues on Ground Control

A stiffer support will develop more load carrying capacity for the same roof-to-floor convergence than will a softer support (fig. 6a). Conversely, a softer support will allow greater roof movement to occur, assuming that the supporting force is controlling the roof movement (fig. 6b).

The following recommendations are made regarding design considerations for hydraulic roof support systems in regards to the stiffness of the support.

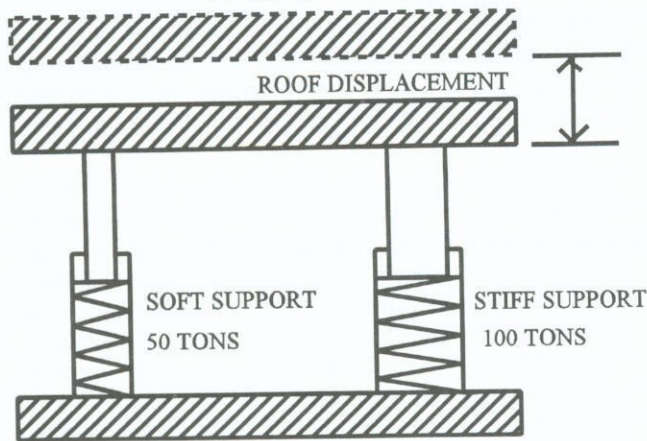


Figure 6a.-Stiffer support develops more loading due to roof and floor movement.

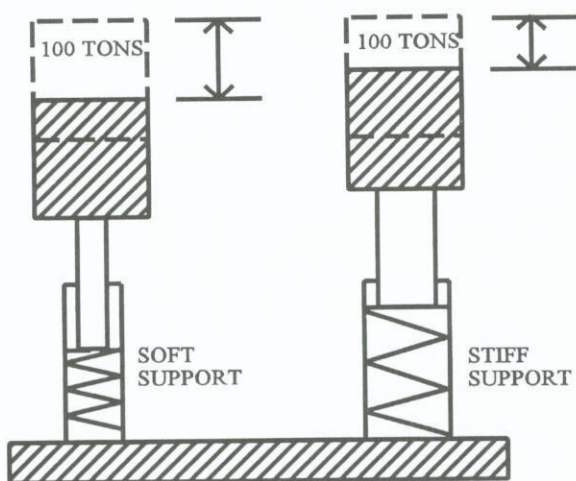


Figure 6b.-Softer support will allow more roof movement to occur.

1. Two-stage systems should be used whenever possible, since two-stage systems are stiffer than three-stage systems for the same (bottom stage) cylinder size and yield pressures. If three-stage cylinders are required to obtain the desired operating range, consideration should be given to making the top stage diameter sufficiently large to provide the required stiffness, even if this requires a larger diameter bottom stage.
2. Since hydraulic support systems are considerably less stiff at high operating heights, they are less efficient at controlling roof movement when used at high operating heights. Therefore, when designing a support, the collapsed height should be no lower than is necessary. This practice will minimize the operating range and provide the least reductions in support stiffness as the operating height increases.

Since the stiffness of the support changes as function of the leg staging, the interpretation of loading on hydraulic support systems may require knowledge of the leg staging, particularly if the support forces are not controlling the main roof movements which is generally the case in longwall mining. In essence, the

developed support loading due to roof/floor movement will be greater when the support stiffness is greater. This issue also complicates the utilization of load rate monitoring devices, such as those being developed for mobile roof supports to be used to assess impending strata caving. The rate of loading is likely to be dependent on the stiffness of the support, hence changes in the support configuration due to leg staging and operating heights will cause the loading rates to change for the same roof behavior (movement).

YIELDING BEHAVIOR

Yield valves are normally connected to the bottom stage of multi-stage hydraulic cylinders to provide overload protection. These valves open whenever the pressure exceeds the design threshold and allows the pressurized fluid to escape from the bottom stage of the cylinder. This loss of fluid causes the pressure to drop in the bottom stage until the valve reseats. The reseating pressure is typically about 90 pct of the yield pressure, hence the support capacity will drop by approximately 10 pct when the yield valve opens. As the pressure drops in the bottom stage, a force imbalance occurs between the top and bottom stage (in a two-stage hydraulic cylinder), which causes the bottom stage to lower until force equilibrium between the two stages is attained. The lowering of the bottom stage is associated with lowering of the mine roof, hence excessive yielding is not desirable from a ground control perspective. Successive yielding events will eventually cause the bottom stage to be fully collapsed, at which time the top stage will begin to collapse. Additional yielding can continue until the top stage is fully collapsed, at which time the support is considered to be iron-bound.

How much the bottom stage lowers due to yielding depends on three factors: (1) the area of the bottom stage, (2) the extension of the bottom stage, and (3) the yield pressure and reseating pressure of the yield valve. The reduction in support height due to yielding which equates to roof lowering will be more for large extensions of the bottom stage than for small extensions of the bottom stage. The magnitude of bottom stage lowering due to yielding can be calculated using equations 10 and 11. Using a 700-ton longwall shield with a bottom stage diameter of 300 mm and a yield pressure of 6,389 psi as an example, the bottom stage of each hydraulic cylinder will drop approximately 0.028 in during yielding if the bottom stage was extended 14 in when yielding began. A reduction in support loading of 70 tons would be caused by the yielding. As the support yields, it also becomes stiffer due to the reduction in operating height (stage extension). Hence, less roof movement will be required to produce additional yielding.

$$\Delta V = \frac{\Delta P * V}{\beta} \quad (10)$$

where ΔV = change in volume of fluid in bottom stage of cylinder, in³
 ΔP = change in hydraulic pressure in bottom stage of cylinder, psi

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ΔP = Yield pressure - reseating pressure, psi
 V = Volume of fluid in bottom stage when yielding, in³
 V = Area of bottom stage times extension of bottom stage, in³
 β = bulk modulus of oil, psi (i.e., 320,000 psi)

$$\Delta L = \frac{\Delta V}{A} \quad (11)$$

where ΔL = reduction in bottom stage extension due to yielding, in

ΔV = reduction in volume of fluid in bottom stage due to yielding, in³

A = Area of bottom stage, in²

ERRORS IN ASSESSING SUPPORT LOADING

Support loading is determined from measurement of the pressure in the bottom stage of the hydraulic cylinder. Since the pressure increases in direct proportion to the increase in support loading, this provides an accurate assessment of the roof loading through the full loading cycle.

However, when the bottom stage is completely extended to its full stroke, a portion of the increase in roof loading after the support is set against the mine roof and floor will not be detected by changes in hydraulic pressure in the bottom stage (see figure 7). The reason for this period of undetected roof loading is that when the bottom stage is fully extended, the bottom stage piston is being held against the mechanical stops with a force exerted by the pump pressure at the time the support was set. The pressure in the bottom stage will only increase when this

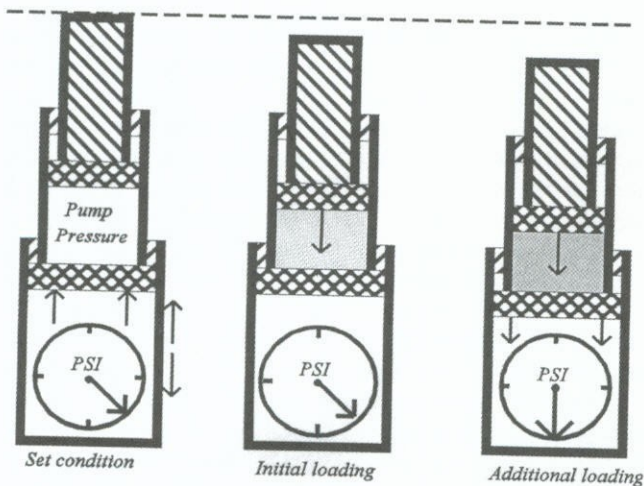


Figure 7.- Initial roof loading after the support is set will be undetected by changes in bottom stage pressure whenever the bottom stage is fully extended.

piston is moved off the stops and begins to compress the hydraulic fluid in the bottom stage. In order for this to happen, the pressure in the upper stage, which is at pump pressure when the support is set, must increase to cause a force in the upper

stage that exceeds the setting force in the bottom stage. The additional roof loading that is required to produce this additional pressure in the upper stage is the roof loading that is undetected by the gages measuring hydraulic pressure in the bottom stage.

The mechanics of the pressure development in the various support stages can be described by examination of equilibrium requirements for each individual stage, realizing that each stage must carry the same load (1). The undetected roof loading for a two- and three-stage hydraulic cylinder with the bottom stage fully extended can be calculated using equation 12. Equation 13 must be used for a three-stage cylinder where both the bottom and middle stages are fully extended. An examination of these principles indicates that the period and magnitude of undetected roof loading will increase as the setting pressure increases. The undetected roof loading occurs immediately after the support is set. The remaining shield capacity (rated capacity minus setting load and undetected load) will be measured by the pressure gages.

$$URL_{Two-stage / Three stage} = \frac{A_2 * P_2 - A_1 * P_1}{2000} \quad (12)$$

$$URL_{Three-stage} = \frac{A_3 * P_3 - A_1 * P_1}{2000} \quad (13)$$

where URL = undetected roof load, tons

A_1 = area of first (bottom) stage, in²

A_2 = area of second stage, in²

A_3 = area of third stage, in²

$P_1 = P_2 = P_3$ = setting (pump) pressure, psi

Figure 8 depicts undetected roof loading for the following state-of-the-art hydraulic support systems: (a) 700-ton (two-stage) longwall shield, (b) 600-ton (two-stage) mobile roof support, and (c) 600-ton (three-stage) mobile roof support set at full pump pressure. It is seen from this figure that the unrecorded

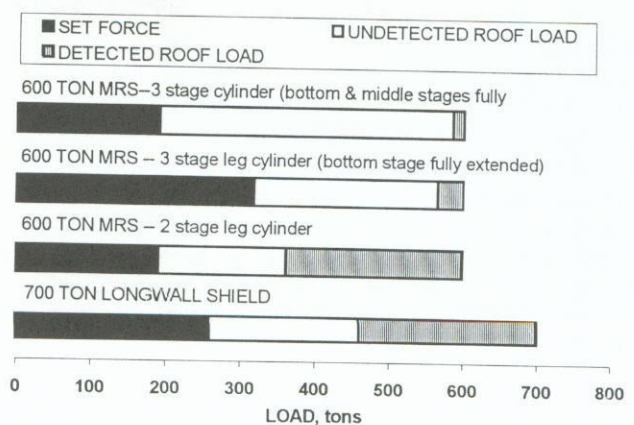


Figure 8.-Magnitude of undetected roof loading for various support systems.

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roof loading is quite large. For example, approximately 200 tons of undetected roof loading, which represents 46 pct of the remaining shield capacity, will occur on the 700-ton longwall after it set against the mine roof and floor. The undetected roof loading for mobile roof support using three-stage hydraulic cylinders can consume nearly the entire loading cycle (see figure 8).

HYDRAULIC FAILURE MECHANISMS AND HOW TO DETECT THEM

Hydraulic roof support systems depend on hydraulic fluid being maintained under pressure to provide the necessary supporting capability. Any condition that results in pressure loss reduces the capacity of these hydraulic roof supports. In many cases, the pressure loss will be internal such that the fluid loss is not noticeable. Sources of pressure loss and ways to detect them are described below.

SOURCES OF PRESSURE LOSS AND IMPACT ON SUPPORT CAPACITY

Pressure losses in hydraulic roof support systems are typically due to leakage of hydraulic fluid. The major components of a hydraulic support system were identified in figure 1 as follows: (1) bottom stage piston seals, (2) upper stage(s) piston seals, (3) pilot operated check valve, (4) yield valve, and (5) staging check valve(s). Each of these components must function properly to prevent leakage of hydraulic fluid that will result in pressure losses. Each of these component leaks result in the exponential decay of pressure with time to **zero**, meaning the support will have **no** load carrying capability after a certain amount of time (assuming the leakage rates are greater than the pressure increases caused by any additional roof loading).

The change in volume required to produce pressure losses can be theoretically determined by equation 14. Assuming a drop of fluid measures $1/8 \times 1/8 \times 1/8$ in and thereby consumes 0.002 in^3 , table 2 shows the time in minutes necessary to produce pressure losses of 1,000, 2,000, 3,000, and 4,000 psi in the bottom stage of a longwall shield cylinder for a leakage rate of 10, 100, and 1,000 drops per minute assuming no additional roof movement or loading occurs. In a recent test of a longwall shield with a bad staging check valve and worn seals, 2,300 psi of setting pressure dropped to zero psi in approximately 4 minutes.

$$\Delta P = \frac{\beta * \Delta V}{V} \quad (14)$$

where ΔP = change in pressure, psi
 β = bulk modulus of oil (i.e., 320,000), psi
 ΔV = change in volume, in^3
 V = original volume, in^3

Table 2. Time required to produce leg cylinder pressure losses

Average Leakage Rate	Time in minutes required to produce designated pressure losses			
	-1,000 psi	-2,000 psi	-3,000 psi	-4,000 psi
10 drops/min	255.5	510.9	766.4	1021.9
100 drops/min	25.6	51.0	76.6	102.2
1,000 drops/min	2.6	5.1	7.6	10.2

1 drop = $.002 \text{ in}^3 = 1/8 \times 1/8 \times 1/8$ in cube

10 drops = $.02 \text{ in}^3 \sim 1/4 \times 1/4 \times 1/4$ in cube

100 drops = $.2 \text{ in}^3 \sim 1/2 \times 1/2 \times 1/2$ in cube

1000 drops = $2 \text{ in}^3 \sim 1-1/4 \times 1-1/4 \times 1-1/4$ in cube

Note: Data in table 2 assumes a cylinder (bottom stage) area of 110 in^2 with a 15 in extension and an initial pressure of 4,000 psi. Table 2 describes pressure losses relative to setting pressure of 4,000 psi in terms of *average* leakage rates. The leakage rate is governed by the pressure differential where the leak is occurring. Hence, the rate of leakage will decrease in time in an exponential fashion as the leakage causes a reduction in pressure differential.

The cooling of the hydraulic fluid can also be responsible for pressure losses in hydraulic support systems. This problem is much more prevalent on mobile roof supports than longwall shields, since each mobile roof support contains a hydraulic power supply, and the fluid which is heated by the pump is not cooled to ambient temperatures before it is pumped into the hydraulic cylinders. The steel in the cylinder then acts as a large heat sink to reduce the temperature of the fluid, which causes a reduction in volume and an associated pressure loss. The thermal expansion coefficient for commercial hydraulic oils ranges from 0.0003 to 0.0005 per degree F (2). Hence, a 10 degree change in temperature will change the volume of the fluid by 0.3 to 0.5 pct. On a mobile roof support with two-stage hydraulic cylinders, this would translate into a pressure loss of about 1,200 psi. Unlike the pressure losses caused by mechanical component leakages which result in no load carrying capacity, the temperature-induced pressure losses will decay to some non-zero value, meaning the support capacity will be diminished but significant capacity will remain.

Another source of pressure loss in hydraulic support systems is an extension of leg cylinders in the absence of additional roof/floor loading. This can be due to fracturing of immediate roof or floor strata or more likely the compaction of roof or floor debris (broken rock or loose coal). On two-leg longwall shields, the leg pressures can also be reduced if the canopy or base slides along the roof or floor due to the horizontal component of the leg forces. An analysis of the 700-ton longwall shield that has been used in previous examples, indicates that 0.11 in of extension of the operating height is sufficient to cause a 1,000 psi drop in leg cylinder (bottom stage) pressure. Like the temperature induced pressure losses, unintended extension of leg cylinders due to non rigid roof and floor boundaries tend to reduce the supporting force, but not completely eliminate it as is the case with the component leakages previously described.

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DETECTION OF COMPONENT FAILURES

The first point to remember about hydraulic leaks is that in all cases there is a loss of pressure, and this pressure loss will continue until there is no pressure in the cylinder (assuming the leakage rates are greater than the pressure increases caused by any additional roof loading). A consistent decline in pressure on each operating cycle after the support is set against the mine roof and floor is an indication of a hydraulic leak. Significant pressure losses during idle shifts are also good indications of cylinder problems.

Observations of the relative positions of the cylinder staging can be used to identify cylinder problems and the cause of hydraulic leaks. As previously described, these leaks are often internal with no visible external fluid leakage. The first indication of internal leakages is when the bottom stage is consistently fully extended. Recalling the previous discussion on multi-stage cylinder operation, the bottom stage should be fully extended **only** on operating cycles which establish a **new maximum operating height**. Hence, on the majority of operating cycles, the bottom stage should **not** be at full extension.

Another indication of problems is when there is a large difference in stage extensions of leg cylinders on the same support. For example, on longwall shields the left and right leg should be at nearly the same extension. Some differences in extension may result from uneven roof heights across the width of the canopy, but these differences should be small and will tend to balance themselves out over the course of many operating cycles. A large difference, such as those shown in figure 9, is an indication of a hydraulic leak that can result in no support capacity. Table 3 depicts the stage movements associated with each component failure (leakage). For two-stage hydraulic

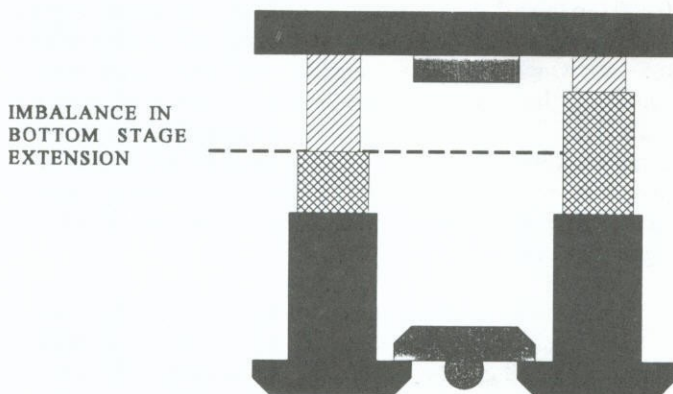


Figure 9.-Imbalance in stage extension is an indication of hydraulic leakages.

cylinders, it is noticed in this matrix that the bottom stage displaces upward or downward due to specific problems, while there is no movement in the top stage. This matrix can be used to provide insight into the nature of the problem. For example, when the bottom stages are consistently fully extended, the matrix indicates that this condition is likely caused by a bad staging check valve or bad top stage seals. Likewise, bad bottom

stage seals will likely result in the bottom stage being extended less than the bottom stage on adjacent cylinders.

Table 3. Relative movement and resulting hydraulic pressure of the bottom and top stages for a two-stage hydraulic cylinder for specific component failures (source of leakage) for static loading conditions.

COMPONENT FAILURE	Top Stage Movement	Bottom Stage Movement	Bottom Stage Pressure	Top Stage Pressure
Staging check valve	None	Up	0	0
First stage seals	None	Down	0	0
Second stage seals	None	Up	0	0
Pilot operated check valve	None	Down	0	0
Yield valve	None	Down	0	0

Combinations of component failure will produce a resultant movement of the bottom stage that depends on the pressure changes in the bottom and top stage. For example, failure (leakage) of the bottom stage seals causes the bottom stage to move down, while failure (leakage) of the top stage seals causes the bottom stage to move up. If the leakage rates are equal, the bottom stage will move up if the volume of the bottom stage is greater than the volume of the top stage, and the bottom stage will move down if the volume of the top stage is larger than the volume of the bottom stage.

Problems with the staging check valve can be isolated by fully collapsing the shield and monitoring the leg pressures on the operating cycle after the support is reset against the mine roof and floor. The requirement is to have the bottom stage fully extended when the support is reset. In this configuration with a staging check valve that is functioning properly, the pressure in the bottom stage will not change significantly until the force in the top stage due to additional roof loading overcomes the setting force developed in the bottom stage (see figure 10). An immediate increase in pressure in the bottom stage indicates that the check valve is leaking sufficiently to not allow the pressure in the top stage to be intensified.

REPAIR AND MAINTENANCE OF HYDRAULIC CYLINDERS

All hydraulic support systems will require repair and rebuild sometime during the life of the roof support system. Since these systems are operating at high pressures, safety should always be exercised when working with hydraulic cylinders. Only 300 psi of hydraulic pressure is required to produce a stream of hydraulic fluid that can puncture the skin and cause serious bodily harm. In addition, bacteria in hydraulic fluids can be toxic and cause serious infection and health problems if left untreated.

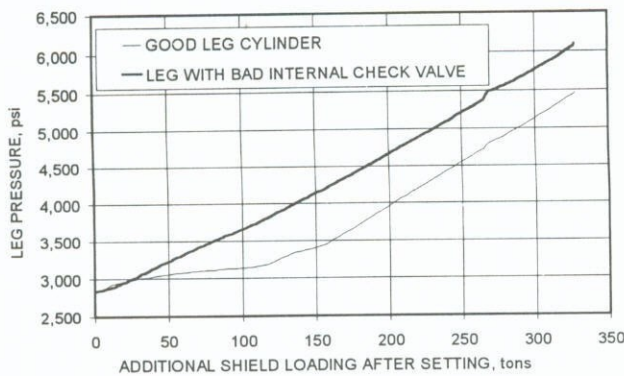


Figure 10.-Bottom stage pressure response with bad stage checking valve.

Safety should also be exercised when removing the hydraulic cylinders from the roof support. Both longwall shields and mobile roof supports will become unstable once the hydraulic cylinders are removed, and it is critical that the structure be properly supported when removing these supporting elements. Repairs should be made by a reputable shop with expertise in hydraulic systems. It is critical that the cylinders be load tested before and after the repair to make sure that all components are working properly. Rebuilding of a hydraulic cylinder will generally entail replacing all seals and check valves, machining and rechroming the cylinders bores and pistons to bring them back to design specifications.

When the cylinders are rebuilt, the ports are often plugged to keep the fluid in the cylinder. One common mistake that is made is not to remove the retract port before attaching the pressure line and extending the cylinder. The bottom stage area is typically greater than the retract annulus area by a factor of 8 to 10. If the retract port is blocked when extend pressure is applied to the cylinder, the pressure in the retract annulus will be intensified by a factor of 8 or 10. For example, a 4,000 psi setting pressure will produce 40,000 psi of pressure in a blocked retract annulus. This magnitude of pressure will literally rip the threads off the end cap of the cylinder casing.

CONCLUSIONS

Hydraulic roof support systems are used extensively in underground mining for critical roof support in many areas. The performance characteristics of these hydraulic supports need to be fully understood to optimize the use of these hydraulic support systems and to evaluate problems when they occur. A proper understanding of these high pressure systems is also necessary to prevent injury during repair operations.

The following conclusions are made regarding the performance characteristics of hydraulic support systems and their impact on ground control capabilities.

1. Setting forces are reduced whenever the bottom stage is fully extended, which occurs on operating cycles which establish a **new maximum operating height**. The setting force will always equal the pump pressure times the area of the largest

stage which is not fully extended. Force reductions of 40 to 50 pct are common for two-stage hydraulic cylinders. Reductions as large as 70 pct are common for three-stage cylinders when both the bottom and middle stages are fully extended.

2. The stiffness of a support controls the development of the support loading in response to the ground movements that cause roof and floor convergence. Several conclusions are made in reference to the stiffness of hydraulic support systems.
 - (a) Stiffness is not significantly affected by the setting pressure.
 - (b) Generally, stiffness is a nonlinear function of operating height, decreasing exponentially as the height increases. However, in multi-stage cylinders, the stiffness is also dependent on the combination of stage extensions. The same stiffness can occur at different operating heights, and certain combinations of staging can produce greater stiffness at higher heights.
 - (c) Stiffness is primarily controlled by the diameter of the hydraulic cylinder with increased stiffness as the diameter of the cylinder increases. In multi-stage cylinders, it is the smallest (diameter) stage that largely controls the stiffness of the support. Higher capacity longwall shields tend to be stiffer due to the use of larger diameter hydraulic cylinders. Two-stage systems should be used whenever possible, since two-stage systems are stiffer than three-stage systems for the same (bottom stage) cylinder size.
3. The supporting force of a longwall shield is reduced by approximately 10 pct each time it yields due to excessive hydraulic pressure which opens the yield valve. Yielding causes an increase in shield stiffness due the reduction in cylinder length.
4. The support loading (both setting forces and a large percentage of the post setting load development) cannot be determined from bottom stage pressure measurements whenever the bottom stage is fully extended.
5. Both internal and external hydraulic leakages will reduce the support capacity to zero under static loading conditions. The time required for complete loss of load carrying capacity depends on the initial loading and the leakage rate, and thereby ranges from a few minutes to a few hours.
6. Observations of the relative positions of the cylinder staging can be used to identify cylinder problems and the cause of hydraulic leakages. One tell-tale sign of internal leakages is when the bottom stage is consistently fully extended. Another sign is when there are several inches of difference in staging on leg cylinders on the same support. Leakages associated with the staging check valve can be isolated by fully collapsing the shield and monitoring the leg pressures on the operating cycle after the support is reset against the mine roof and floor. An immediate increase in pressure in the bottom stage indicates that the check valve is leaking sufficiently to not allow the pressure in the top stage to be intensified.

Finally, safety precautions should be made when working with these high pressure hydraulic support systems. The fluid pressure is more than adequate to cause serious bodily harm.

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Support structures should be properly stabilized during repair work. Incorrect connection of hydraulic hoses to the cylinder can result in unintentional damage to the hydraulic cylinder and potential injury to the mine worker. A well trained person should be conducting the removal, installation, or repair of hydraulic support systems.

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A Decade of Mobile Roof Support Application in the United States

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Huntington, WV

ABSTRACT

Second, or retreat mining with Mobile Roof Supports (MRS) has now been part of coal mining in the United States for a decade. Their utilization has evolved into mining applications which vary in seam heights from 38 inches to 170 inches, and overburden depths from outcrop to beyond 1200 feet. This paper will detail the pillaring plans, lift sequences, and machinery utilized in these operations and the rationale for the plans. A comparison of the current productivities achieved will be outlined.

INTRODUCTION

The utilization of Mobile Roof Supports in pillar extraction has now been a part of coal production in the United States for over ten years. At this writing, approximately 43 sets of supports are in service in coal mining regions across the country. In the last two years, the number of systems in use has almost doubled. This is a result of the safety, productivity, and economic gains that can be realized with the automation of the timbering cycle of retreat or second mining.

Retreat mining can be very productive, but because of the nature of roof caving, can be unpredictable and risky for both personnel and equipment. Mobile Roof Supports are set in place of the wing, roadway and, in some cases the breaker posts. They provide protection for the mine personnel and the face equipment while pillar extraction takes place. Timber setting in the production cycle is eliminated. When utilized in conjunction with remotely controlled face equipment, they allow the pillars to be extracted without the slow cycle of splitting the pillar and greatly reduce the probability that the continuous miner will be caught by a roof fall. Mobile Roof Supports have also been operated as walking shields for the recovery of longwall shields during longwall moves.

HISTORY

The first prototype of a Mobile Roof Support (MRS) was

developed through the guidance of the Spokane Group of what was then the U. S. Bureau of Mines. This unit was a modified roof bolter chassis with two cylinders for roof support mounted within the chassis and two additional cylinders. These cylinders were attached to arms, which could be jettisoned from the chassis (figure 1). The roof support cylinders lowered to the mine floor and raised the rubber tired chassis off the floor. A pod was attached to the top of the cylinders which spread the contact area against the mine roof. This first generation unit was tested in 1979 at Inland Steel, Sesser, Illinois with unsatisfactory results.

The second generation system was modified to a crawler drive and the three point contact pods on the chassis roof cylinders were replaced by rails. Each cylinder on the support was rated at 50 tons capacity. This system was tested with much more favorable results at Southern Utah Fuel Co., near Salina, Utah, from April to November in 1984. A single support was set in the entry, and one in the pillar split. They replaced all but two of the wing and roadway posts.

The concept evolved into a machine with a smooth canopy, similar to a longwall shield. This design direction was followed by several mining equipment manufacturers. Prototype units, constructed by Voest Alpine Mining and Tunneling, were observed in operation in 1987 at the Middlebult Colliery at the SASSOL mining complex near Secunda in the Republic of South Africa. These supports, referred to as "Breaker Line Supports" are utilized in pairs, with two supports set in each entry. Structurally, the BLS are a crawler driven chassis with a flat, lemniscate guided canopy supported by four cylinders. This support was rated at a total capacity of 600 tons. The first commercial application of the BLS in the U. S. was in early 1988 at the Donaldson operation of The Valley Camp Coal Co., near Charleston, West Virginia. In 1988, J. H. Fletcher & Co. began production of a set of 600 ton supports that were installed later in that year, in the mines of Martin County Coal, near Inez, Kentucky. This mobile support was the result of design and prototype evaluation conducted at Fletcher since 1984.

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The application of MRS to retreat mining in Australia began in 1987 and has continued successfully to date with many systems in use. However, the details of their use in Australia will only be briefly covered in this paper.

APPLICATION/OPERATION

Mobile roof supports are set in the entry and crosscut around the pillar to be extracted. They are utilized in pairs and generally positioned with one slightly ahead of the other in an arc, similar to that described by wing timbers (fig. 2). The continuous miner extracts the wing to the right. After the lift is completely mined, the gob, or goaf side MRS is retracted a few inches from the roof and trammed forward approximately one-half the length of the support, or 6 to 7 feet. The roof plate is then reset. The adjacent MRS is lowered and trammed forward a similar distance, then reset against the roof. This process continues until the supports are positioned for the next lift. This cycle time should require one to four minutes and can usually be accomplished while the continuous miner is repositioned and its cable slack pulled back. This method, where a pillar is extracted only from one side or entry, is referred to as the outside lift, or one sided lift approach. This lifting sequence is applied generally in seams with 600 feet of overburden or less, although its use has been observed in some areas with very strong coal and competent immediate strata at depths well over 1000 ft.

Mining of the lifts continues until the pillar has been completely extracted from the entry. The MRS in the entry are then positioned at the intersection. The outby end of the pillar is then mined with the other pair of MRS (figure 3). The final stump or pushout is extracted with four supports set in close proximity in the crosscut and intersection (figure 3). After the pushout is completed, the four supports are moved together, and set against the roof in the intersection for their protection, and as they are removed from the intersection, they are trammed to the same positions to start mining in the next pillar.

Mobile Roof Supports are also used to extract the pillars by mining lifts to the left and right out of the same entry or breakthrough. This is referred to by a variety of different terminology's, such as twinning, left and right, Christmas treeing, tree topping, and other appellations too numerous to relate. This plan is the most commonly applied technique for MRS, being utilized on approximately 65 to 70% of the sections.

Twinning requires that the lifts be kept to the proper width dimension as mining takes place in the entry. If this is not done, the operators will be working with an unsupported, open lift to their backs. Coordinating the continuous miner operator's moves and those of the MRS operator can alleviate this problem.

Move time for the MRS is shorter per lift when twinning, and fewer moves are required per ton mined than in the outside lift plans. Because this method is applied in deeper seams, or higher grind, weak coal beds, the pillars are generally larger and several lifts must be mined from the back, or breakthrough side

of the pillar (fig. 4). In twinning, the two MRS positioned in the crosscut play a more important, and in some seams, a critical role in the protection of men and equipment. Figure 5 shows the positioning of the supports while the mining of the final stump, or pushout takes place.

The broad spectrum of mining plans that have been written and applied are variations of the outside lift and twinning lift sequences. These plans have been altered and expanded to adapt them to specific mining conditions, the face equipment in use, and the concerns of management, the workforce, and the regulatory authorities in the various mining regions.

MRS have been utilized in thicker seams that have been bench mined. The upper portion of the seam is extracted as the panel is driven and supplemental rib support is installed during this development. When the pillars are retreated, bottom coal is extracted for a distance of one to two rows of pillars and a row of pillars is extracted with high seam MRS by either the outside or twinning lift sequence.

Pillar extraction has been performed with Mobile Roof Supports both above and beneath mined out longwall panels and areas that had been previously retreated with timber.

BENEFITS OF MRS UTILIZATION

The MRS provide an upward, active force on the immediate roof strata. When two supports are positioned and set in an entry, there is a total active force of between 300 and 600 tons applied to the roof. This results in the normal cave line of the roof being pushed back into the gob. The difference in the cave line of the roof with timber and the cave line with MRS is also shown in figures 2 & 5.

The control of the roof in the immediate area of the pillar extraction provided by the Mobile Roof Supports can yield substantial benefits in a broad range of areas that are critically important to mine operators. These include safety, productivity, and mine cost. The positive aspects resulting from the application of Mobile Roof Supports are summarized as follows.

- Forcing the cave line further into the gob allows for a wide or thicker fender to be mined on retreat. When radio controlled continuous miners are utilized, "fenders" as thick as 40 feet have been mined on retreat. This eliminates the costly and relatively unproductive cycle of splitting the pillar and then supporting the pillar split.

- A higher percentage of the reserve can be mined on retreat. Production is derived without the roof support material and the infrastructure costs that accompany mine development and receding faces.

- The exposure of workers that is associated with setting turn posts, and in some regions, breaker timber, is eliminated. Radio remote control of the MRS and the proper installation of cable hangers that can break away from the roof can ensure this.

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Where breaker posts are set, an MRS can be utilized to protect those setting these posts. It is the opinion of many mine personnel that the benefits of breaker posts are not offset by the hazard and expense of installing them. An increase in the overall level of safety may be achieved by not setting this support and keeping the workforce out of the area; as is the case in Southwestern Virginia. However, in those seams where overrunning caves are a factor, breaker posts should be maintained.

- The continuous miner produces at a higher average rate. With older, timber retreat plans, coal fenders were only 15 to 18 feet thick and the continuous miner spent a large portion of its production cycle turning into the lift, with the cutter drum only partially engaged. The thin fenders were then mined through quickly. The miner operators spent the majority of the shift in the unproductive cycle of turning the miner into the coal rib. With MRS, the continuous miner is able to mine a large volume of coal without tramping to another place. Accordingly, the efficiency of the haulage setup becomes even more critical. Those mines with very efficient haulage layouts, especially continuous haulage, can produce substantially higher output on retreat, as compared to development.

- Operating the supports realizes a dramatic reduction in the usage of timber and the accidents that are often associated with timber handling. This includes the cost of the posts and the labor to supply and set the prop. The operation of MRS eliminates the need for roadway, wing, and in some regions, breaker posts.

- Less time is required to set up for a lift with MRS than when setting wing posts. Especially when mining lifts both left and right, the MRS can be advanced before the miner is ready to turn the next lift.

- MRS allow for higher reserve recovery. A higher percentage of the coal in the pillar is recovered safely with MRS than with timber. A larger pillar can be safely and more completely extracted. Fewer stumps are left remaining and those that are left are smaller.

- Complete pillar recovery and the ability to mine a larger pillar results in a more stable pillar line. Generally, the sections utilizing mobile roof supports do not have rides, squeezes, and pressures overriding the immediate face.

- Fewer personnel are required on a crew retreating with MRS than on a developing panel, or on a retreat panel with timber.

- Due to the more complete recovery associated with the operation of roof supports and the larger pillar size, caving becomes more regular and somewhat predictable. The hazards associated with unexpected or premature caving are substantially reduced. However, no guarantees are made or implied in this matter. Caving patterns are, by nature, individual to each mine and can vary according to the strata present at any given location.

- When unexpected caving does occur, the fall is stopped with a very high degree of reliability by the MRS. Roof falls which would have caught the continuous miner and required two days to a week to recover, are reduced to no delay or less than two hours for recovery. The recovery process with MRS is also much safer than with posts and cribs. MRS have often been used as auxiliary support in these situations, both to increase the level of safety and to speed the recovery process.

- Raw material can be mined at a higher yield on retreat. The support provided by the MRS, combined with a more stable pillar line contributes to less deflection in the roof, minimizing the contamination of the product by draw rock. A more stable pillar line can decrease floor heave and minimize the need to load floor material.

The operation of Mobile Roof Supports represents a very different mining situation to many mine managers and equipment operators. There are several factors which should be recognized, accepted, and dealt with for reliable and safe operation of the pillar line while utilizing Mobile Roof Supports.

- Always position the MRS as close to the continuous miner for maximum protection of the equipment and personnel. Do not try to mine more than one lift without advancing the pair of MRS. However, allow adequate space for the miner operator to rapidly exit the lift and ensure that the miner is not "wedged" by rib sloughage. Do not use them separately. Work the MRS in pairs only.

- Retraction of the roof plate (canopy) can trigger a fall. Do not lower more than one MRS roof plate at a time when advancing the machine to another lift. If this occurs, or caving begins while advancing the support, reset the roof plate immediately and let the support "ride out" the fall.

- The lack of timber (breaker posts are knocked out when the MRS are first positioned in the entry) does not allow the warning that is given in a full timber plan. This circumstance can create discomfort among those accustomed to using wood.

- Since a very high percentage of the pillar is recovered with MRS, it is sometimes advisable to leave a small stump, "peg", or fender of coal to give warning or protect the equipment, especially while moving out of the pushout area. These must be sized to crush out later. Each mine represents a different situation regarding the size and placement of these protective stumps.

There is no substitute for training and experience in full retreat mining. This is especially true when the initial break of the main roof occurs on the panels. Do not become over confident with the operation of any roof support. The MRS provides support of the immediate roof only, and cannot "hold up the entire mountain" as is sometimes commented.

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MINING PLANS FOR MOBILE ROOF SUPPORTS

The first full retreat pillaring plan utilizing Mobile Roof Supports in the U. S. was issued to Donaldson Mine Co. in 1987. It required line timbers to be set and although figure 6 does not show it, a staggered face was required to be maintained. The line timbers were removed as a requirement as long as three rows of bolts, two feet longer than the normal anchorage length were installed in the intersection. Only the outside lift method was allowed (no twinning or left/right lifts) and lifting off the barrier was not permitted.

A modified rib pillar type of extraction, first observed in South Africa, was utilized to pillar a barrier up to 200 feet thick between the five entry panels, (figure 7). Overburden depth varied from outcrop to 550 ft. The entries were spaced on 100 foot centers, and breakthrough centers were 50 ft. Pillar extraction proceeded from left to right across the last open break. This orientation of the pillar has been referred to as "laid down". There is very little time required to move and set the MRS up in the next pillar and haulage was augmented by a permissible feeder that kept the haul distance short. This plan, with some variations, is still in use in West Virginia.

Today, Mobile Roof Supports have been utilized in many different seams varying in depth from outcrop (mountaintop mine) to over 1700 ft. of cover. They have been applied in seam heights from 38 inches to 13 feet. Twinning is the most widely practiced lift sequence (fig. 4). There are many plans that achieve similar goals as the rib pillar. They mine by lifting off the barrier pillar or driving short rooms into the barrier and recovering the fenders/pillars in those rooms. These minimize the haulage distance. Note haulage distance is the major limitation to the rib pillar type of plan (figure 8). The majority of pillar orientations are with a short distance between entries and longer breakthrough distances, or with the pillars "standing up". This can allow the crosscuts to be driven through in one cut on development with a remotely controlled miner, and also requires fewer stoppings (figure 9).

Generally, a set of Mobile Roof Supports is comprised of four machines. There are several plans which require only two MRS. The two MRS plans are approved for thinner and more shallow seams. These plans have usually not allowed mining on the back side of the pillar. However, several plans have recently been approved which combine timber (set for the mining of the lift from the breakthrough) and MRS (see figure 10).

The various retreat lift patterns have been used with all types of haulage, including shuttle cars, battery haulers, and continuous haulage. The haulage plans (figures 11, 12 & 13) generally require pulling the pillars from the outside entries to the center conveyor entry. In some seams this can result in stress concentrations in this belt entry as the pillars or fenders remaining there are extracted. This situation can be moderated by modifying the type of bolts installed on development, altering the timing of the belt back, and leaving strategic stumps that will later crush out. Other plans are being utilized for recovering

barrier pillars in the outcrop of mountaintop mines, where surface mining has previously occurred (fig. 14).

MRS have been operated on super sections with two continuous miners and two sets of MRS working the face simultaneously. Other super sections operate with one set of MRS. The continuous miners are not operated at the same time, and one develops fenders for retreat in the barrier. The second miner retreats the face pillars and the barrier fenders as they occur in the sequence (fig 15).

After observing the operation of MRS on numerous retreat panels at a variety of operations, the following trends in output rates have been noted. Whereas the comparisons can not always be directly made due to the wide range of scheduling options, equipment selection, mining conditions and worker motivation, some generalizations can be made. Retreat productivity with continuous haulage has been observed to peak at approximately 900 ft. per shift and can average between 700 and 800 ft. per shift. In lower seams, a single continuous miner with shuttle car haulage, 38 inches of working height, and an 8 1/2 hour shift has produced around 300 ft. per shift.

Retreat with MRS has taken place in several seams with previous second or longwall mining in adjacent seams both above and below the active works. Pillaring is currently taking place in seams that were judged too difficult or hazardous to retreat with timber plans.

The applications of Mobile Roof Supports has expanded to their use as walking shields in the recovery of longwall shields. This has been done at several mines, the first being the Eagle Energy Mine, near Van, West Virginia.

AUSTRALIAN APPLICATIONS

MRS have also gained wide acceptance in Australia. In these applications, a set consists of three supports (figure 16). Lifting of the pillar begins with two MRS. After the first lift has been extracted, the continuous miner is set to the side of the entry, or backed out of the place, and a third MRS is positioned next to the other two.

The use of MRS is currently being evaluated in several countries besides the U. S. and Australia for coal and hard rock mines.

PILLAR DESIGN

Sections that apply the technology of Mobile Roof Supports experience higher productivity and substantially improved strata control when the pillars are sized and developed specifically for the retreat process with MRS. Proper design can ensure that development and retreat productivity and production are balanced according to the overall roof support costs at a particular mine. Most operators are now finding that, in most seams, retreat productivity can substantially exceed that obtained while driving. Retreat tonnage is placed in an even more favorable cost perspective if the overall considerations of

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yield, supply cost savings, and increased economic life of the reserve are factored into the equation. Proper pillar design is an essential for achieving a safe and productive pillar line.

THE INTERESTING DAY

No discussion of Mobile Roof Support application should be complete without addressing the problems that can occur when premature caving locks the support in place. This problem can occur with inexperienced crews and, with much less regularity, the fully initiated.

It can be stated that, depending on the specific conditions and the training of the crews, getting an MRS "caught" is an infrequent problem. A Fletcher MRS has a powerful tram drive system that enables it to tram out, without assistance from a large majority of these problems. In approximately 10% of these occurrences assistance is required in the form of a pull from the continuous miner. Generally, an 1 1/4 inch wire rope, or a short piece of longwall chain is used. A retriever should be available within an hour from the section.

In all but two occurrences during the last three years by the author, proper and safe extraction of the support has required less than 20 minutes.

CYLINDER PRESSURES

Set pressures on the roof plate vary between 25% and 50% of initial yield pressure. If set pressures are too high, fracturing of the immediate roof can occur in fragile roof areas. A set pressure too low can result in the support settling into the floor, or roof strata while mining takes place, with a resulting loss of pressure and a decrease in the active support applied to immediate roof. If the pressures are reduced due to the relaxation of soft material in the mine floor, the support must be reset.

Large, 6 inch gauges are connected to the front and rear pairs of the roof plate cylinders. These are accurate until the bottom stage of the cylinder is fully extended. However, the pressure gauge readings cannot be inferred to reflect or indicate the full range of status of the roof. In many mines, the roof plate builds pressure, often near completion of a pillar or prior to a fall. In many others, the gauges show little increase, if any, during the mining cycle. Increasing pressure recorded by the gauges is sometimes a warning of an impending fall. However, no change in pressure should not be taken as a sign that all is entirely well. Pressure gauge readings have also decreased just prior to a fall.

The differential pressure, or rate of pressure change is a more representative indicator of the roof behavior. In order to provide a more accurate and representative warning of the progression of roof convergence and hence the impending cave of the goaf, J. H. Fletcher is currently working with the Spokane Group on a differential load rate warning system. This prototype monitor should be installed on a support and placed into service in July or August, 1998. It will measure the rate of change in pressure imparted onto the roof plate assembly of the

MRS and provide an audible and visual warning of this change.

CURRENT DESIGN STATUS

The earliest supports were intended to replace the rows of breaker posts, both in design and function. The post type support had obvious problems when the immediate roof was broken or caved upon the retraction of the cylinders. Advancing a single machine with no adjacent support in place proved risky all too often. This design was replaced early on most drawing boards by the smooth canopy type of design that is seen today with the MRS utilized in pairs for additional support while advancing the MRS.

A Mobile Roof Support consists of (Figure 17) a roof support plate (canopy), lemniscate linkage with a caving shield incorporated into it, support cylinders, chassis, side curtains, and plow.

Some of the early supports trammed slowly and were frequently stopped by debris left in front of the machine. Caving often damaged side protection. Cable reels were not included and the cable, which had to be "figure eighted" on the plow was bulky, hard to control, and sometimes damaged. Gauges to read cylinder pressure were nonexistent or small and difficult to read. Hydraulic systems were difficult to keep on stroke and components were hard to access, especially in poor rib and roof conditions. Machine size and the expected benefits of the concept were thought to limited applications to working heights of approximately seven feet or greater.

Reliability is a critical requirement for an MRS. A bulky, unpredictable support can mean more than just lost production on the pillar line. A mobile support is subjected to severe duty and must be capable of surviving the repeated abuse dealt in this environment. The mobile roof support manufactured today is an electrohydraulic unit with a 50 or 75 horsepower motor driving a piston hydraulic pump. All functions are hydraulically operated and controlled by radio remote. The unit has hydraulic tram motors equipped with variable torque/speed control. These motors are equipped with valve blocks that reduce tram speed from a normal speed of 75 fpm if impediments such as debris or hills are encountered, and automatically increases the available torque to handle such obstructions. This is especially valuable in situations where caving has flushed loose rock around the support or the continuous miner has left a deep pile of coal in front of the MRS.

The plow contains the electrical enclosure, cable reel, and hydraulic valve chest. The location of all controls in the plow makes troubleshooting safer and easier. It is raised and lowered by cylinders capable of raising the crawlers off the ground. High strength steel linkage connects the plow to the chassis and is designed to allow the use of up to a 300,000 lb. retrieval systems.

Control of the support is through a radio transmitter/receiver whose outputs are directly connected via intrinsically safe

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circuitry to the valve bank and its pulsar type solenoids. This eliminates interface relaying and enhances reliability. Pilot valve blocks are also eliminated and the total hydraulic system is less contamination sensitive. Radio control is supplemented by a full function demultiplexed pendant control. The transmitter can operate up to eight MRS and the pendant can control any machine to which it is connected. Various safeguards are in place to make sure that each support can be operated by only one remote at any time.

Roof plate cylinders are available in two or three stage versions, with the three stage preferred because of the wider operating range that it affords. Cylinder capacity can be 150 or 200 tons yielding a total support capacity of 600 or 800 tons. Mines with deep overburden or extremely competent strata overlying the coal seam generally select the 800 ton support. Each cylinder contains a low volume yield valve and a high volume atmospheric relief, or rock burst valve to protect the MRS in case of heavy or sudden loading. It should be noted that the MRS has gone through several design evolutions and is comprised of high strength steels. Compromising the strength of the steel during repairs and maintenance can dramatically affect the durability and reliability of the support.

As each mine has individual characteristics, a wide variety of height ranges is manufactured in either 600 or 800 ton capacity. In thin seams, a 32 inch (collapsed height), 600 ton support (fig. 18) is utilized. The lowest MRS built to date in the 800 ton size has been a 42 inch collapsed height machine.

At the other end of the spectrum, 600 and 800 ton MRS have been manufactured which ranged up to 174 inches (fig. 19). The higher systems can also have a tilt frame incorporated into the structure. This allows the controlled tilting of the canopy in higher seam applications and ensures that the roof plates of adjacent supports do not collide if the MRS is trammed over uneven ground or loose debris (figures 20 & 21). The higher MRS have been utilized in thicker seams that were mined in benches, or separate levels on development versus retreat.

An additional feature that can be incorporated into the support is the ability to alter the height of the support by changing cylinders, and/or lemniscate linkage. This allows the MRS to be utilized in multiple seam applications with a fraction of the capital required for several sets of supports. Canopy and cylinder extensions are also available for minor or localized increases in working height.

The roof support plate (canopy) is constructed of 100,000 psi steel and is a total of 7 inches thick. This provides a rigid canopy that does not deflect under the roof load, and ensures a smooth surface for the MRS to slide from under caved roof. The ability to independently tram from caved areas is a major advantage of a Mobile Roof Support. The caving shield, incorporated onto the lemniscate linkage, translates the vertical loading of the gob or goaf into a horizontal direction and assists in bringing the support out of the gob. This has been referred to as a "squinter" by operators utilizing the support. The powerful tram system, the rigid canopy, and the caving shield all work to

ensure that the support exits the gob under its own power.

SUMMARY

The use of Mobile Roof Supports continues to expand rapidly for both retreat mining and longwall shield recovery. As reserve depletion forces the mining of thinner seams in deeper strata, and in more adverse roof conditions, the need for productive second mining alternatives will increase. Mobile Roof Supports, when properly applied, can allow the economic recovery of reserves in bedded strata with a substantially higher degree of safety and improved productivity. J. H. Fletcher & Co. plans to play a primary and integral role in this field by providing a Mobile Roof Support that is custom designed to meet the needs of the mining industry.

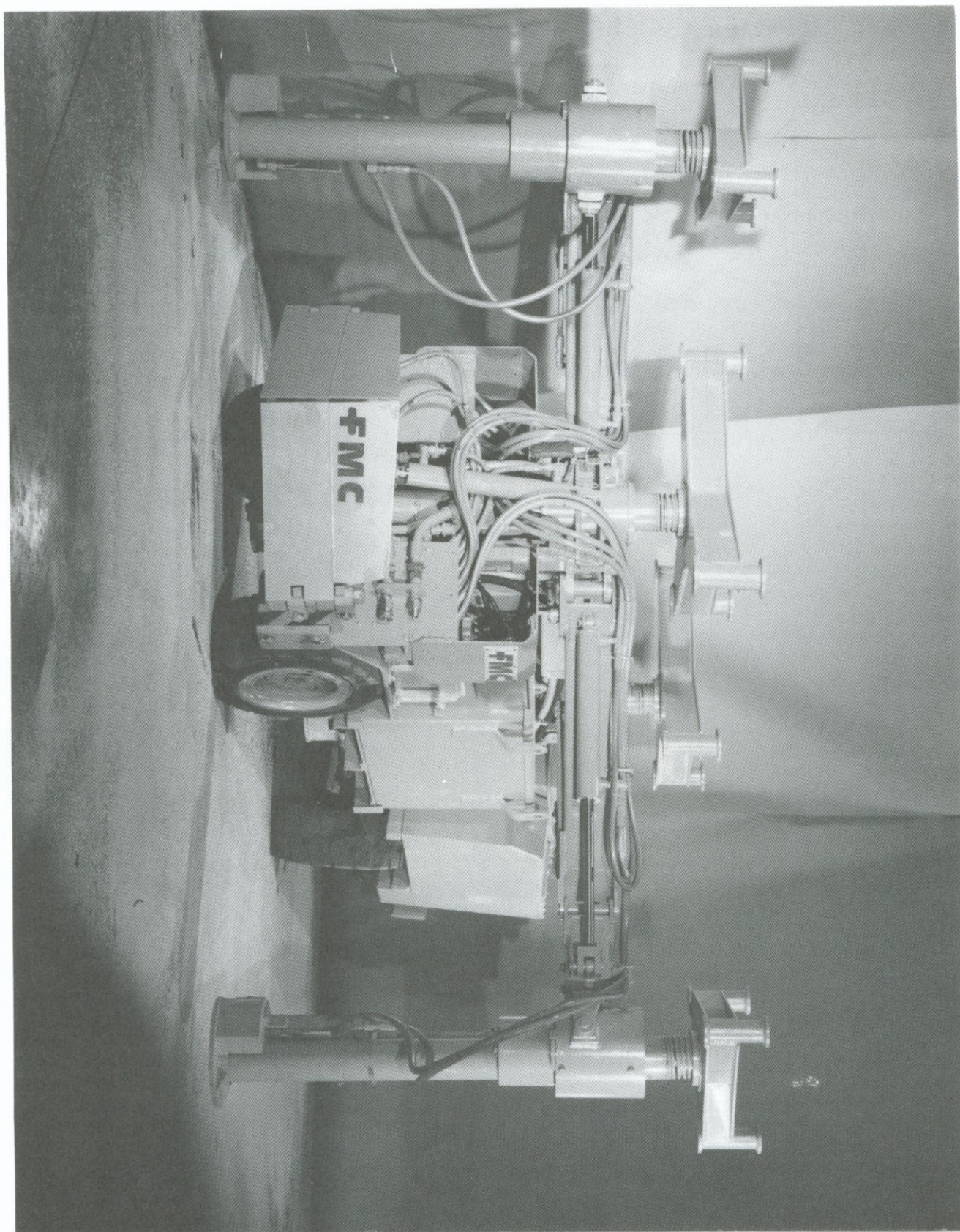


Figure 1
"First generation Mobile Roof Support"

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FIGURE 2

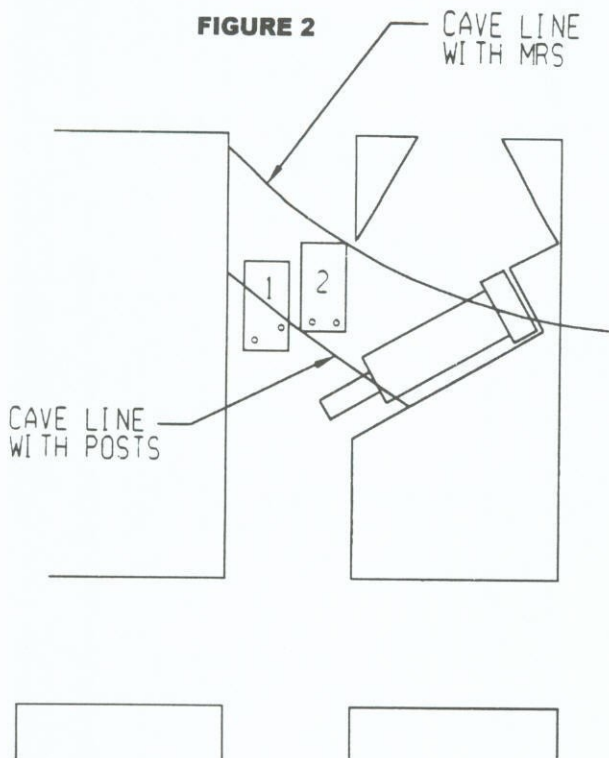
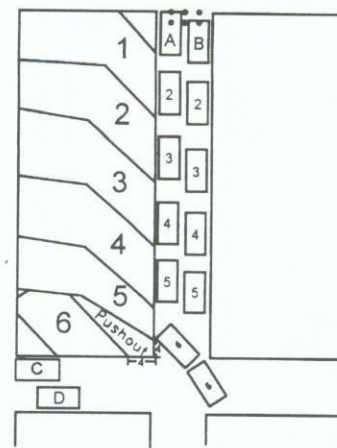
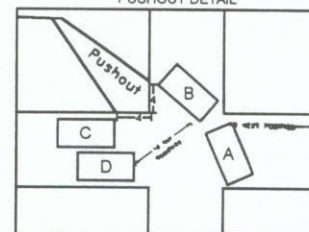


FIGURE 3



PUSHOUT DETAIL



PILLAR EXTRACTION USING THE XMAS TREE METHOD

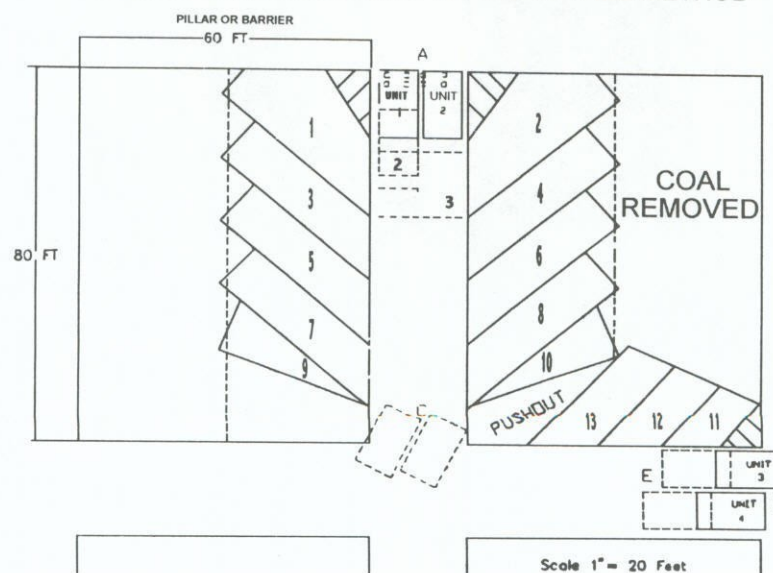


FIGURE 4

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FIGURE 5

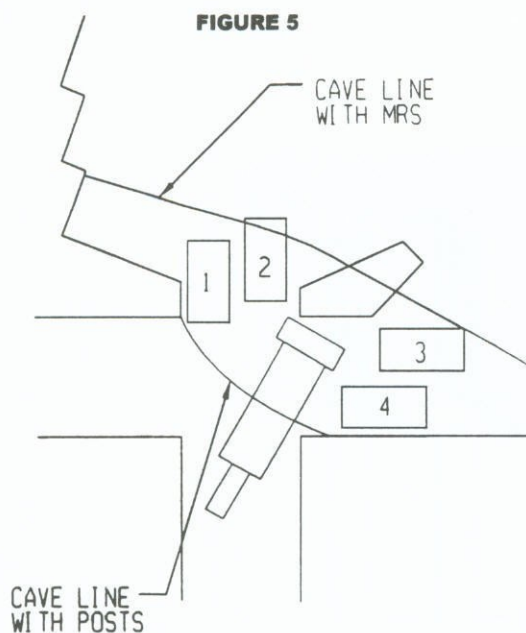


FIGURE 6

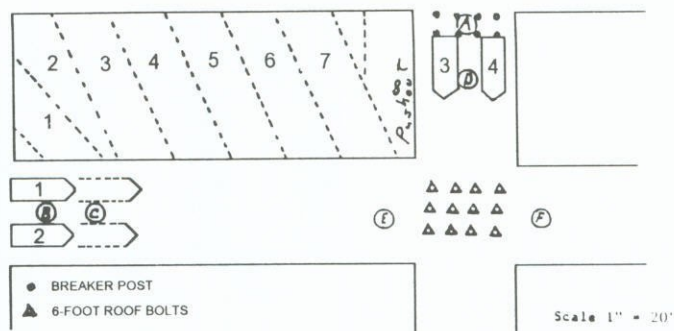


FIGURE 7

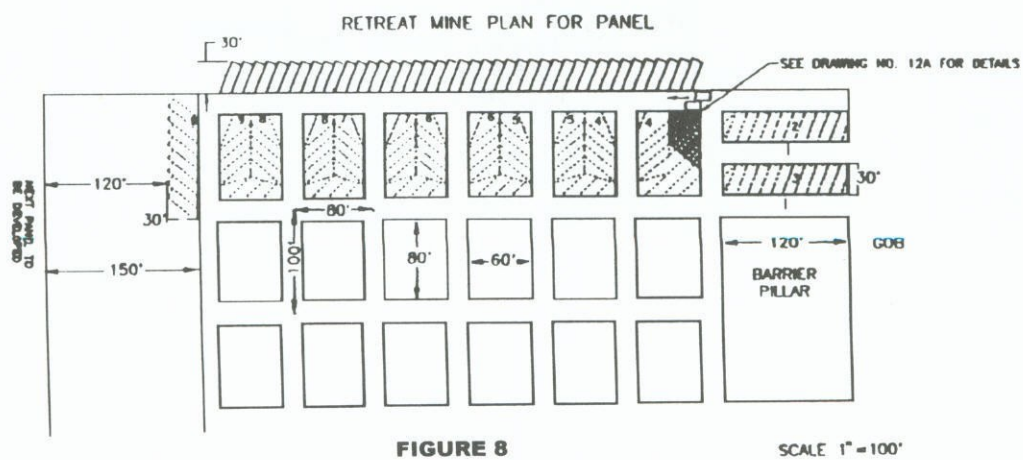
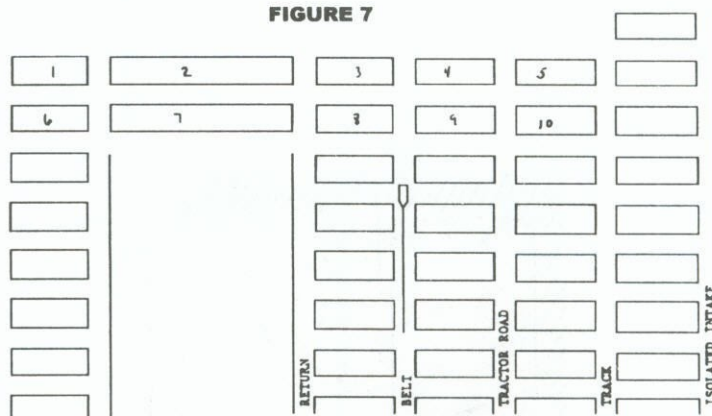


FIGURE 8

SCALE 1" = 100'

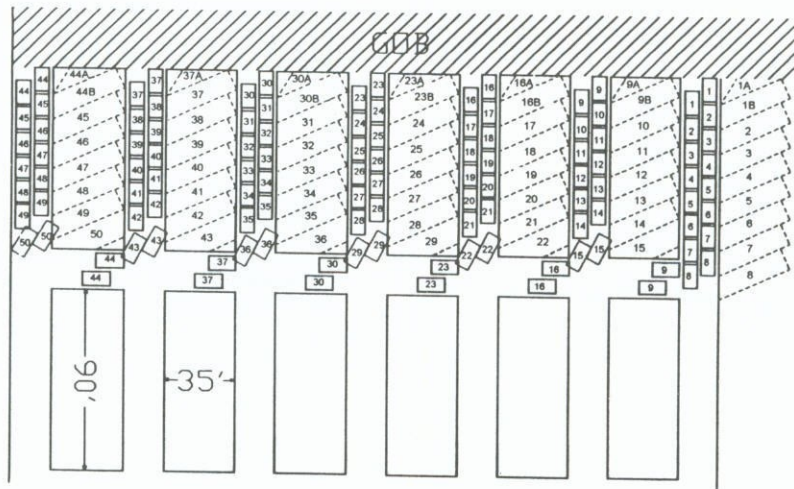


FIGURE 9

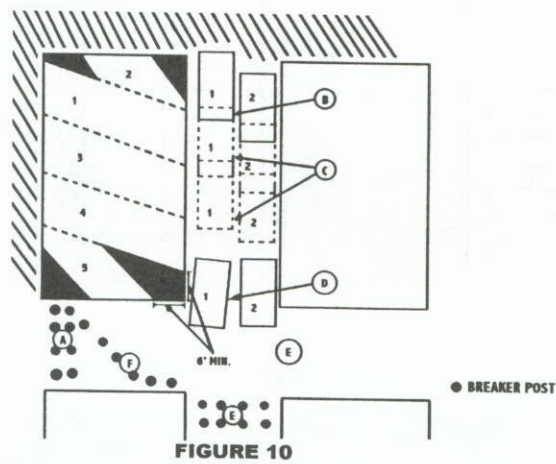
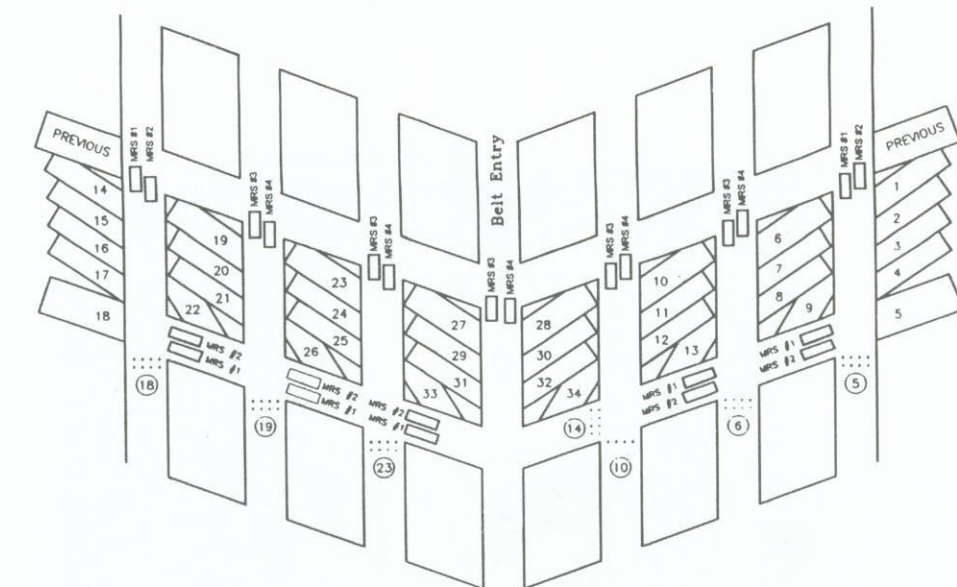


FIGURE 10

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- Legend**
- (18) — MRS (Mobil Roof Support)
 - 18 — CUT SEQUENCE
 - (18) — INDICATES POST TO BE SET PRIOR TO MAKING CUT INDICATED
 - :::: — TIMBERS

FIGURE 11

PILLAR RECOVERY PLAN DEEP-CUT USING MOBILE ROOF SUPPORTS SCALE: 1 IN. = 20ft.

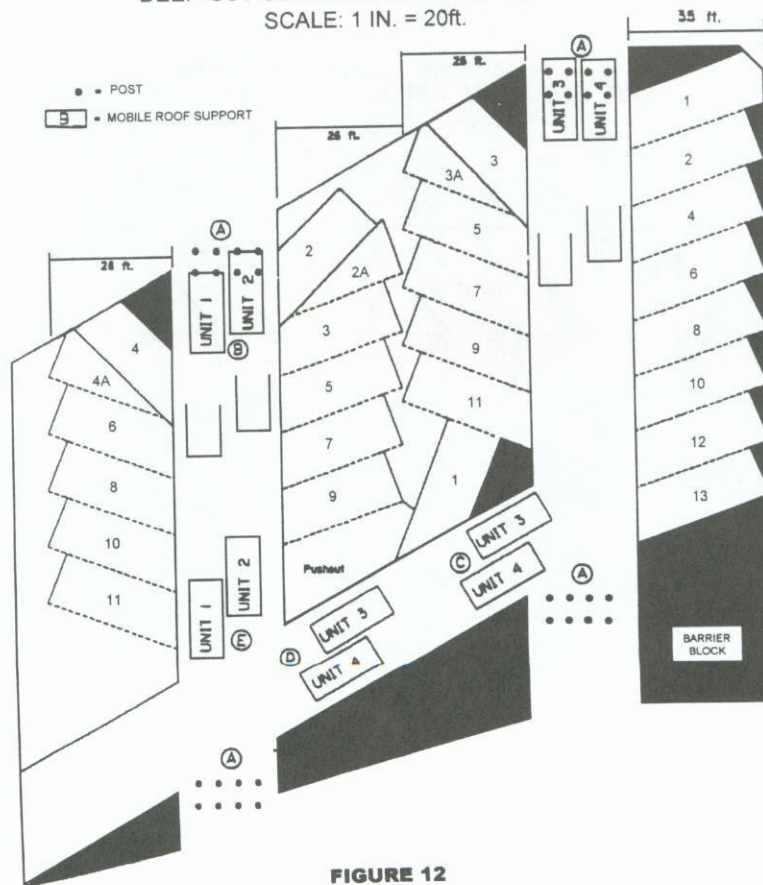


FIGURE 12

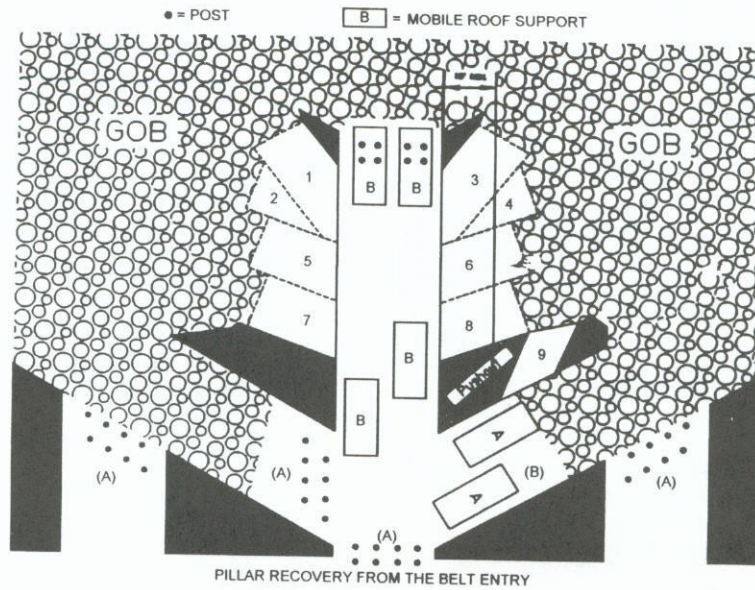


FIGURE 13

BARRIER PILLAR RECOVERY
DEEP CUT PLAN USING MOBILE ROOF SUPPORTS

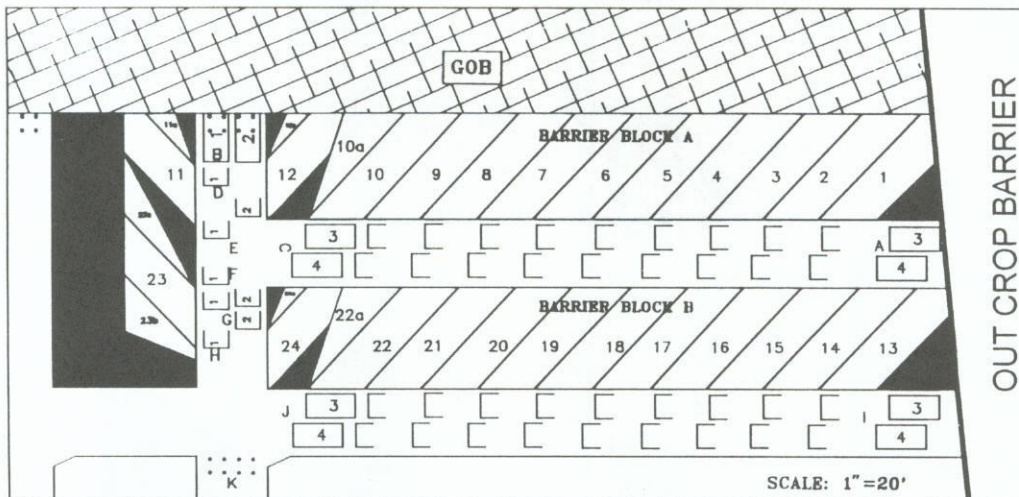


FIGURE 14

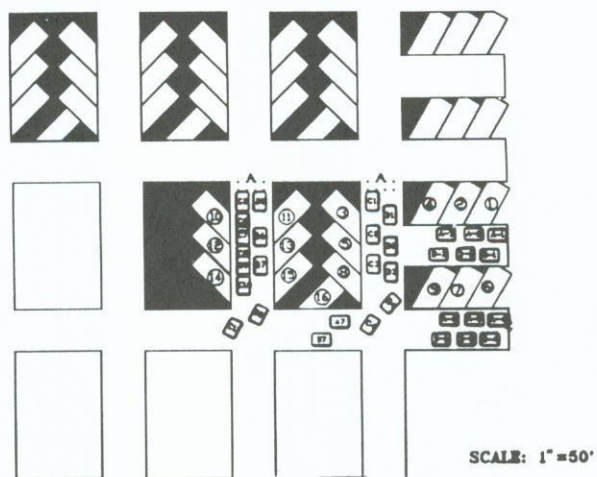


FIGURE 15

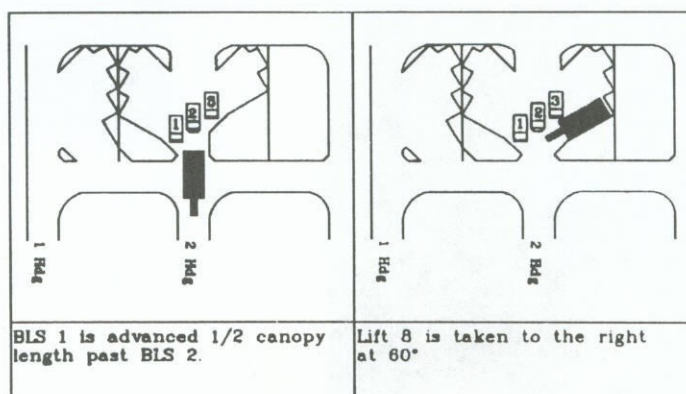


FIGURE 16

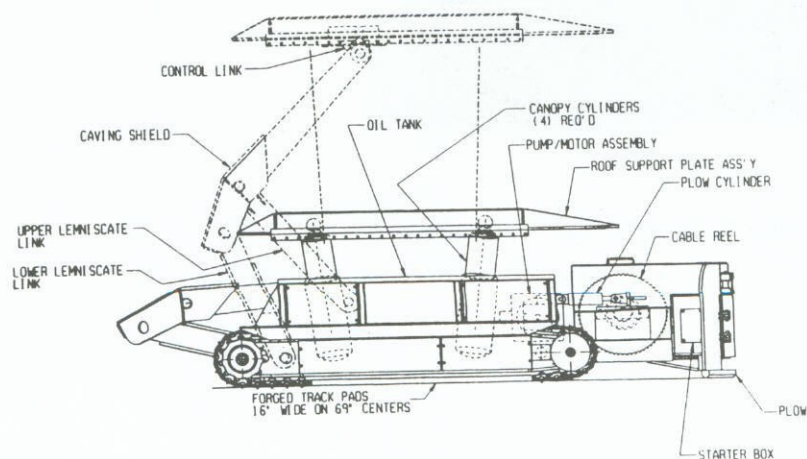


FIGURE 17

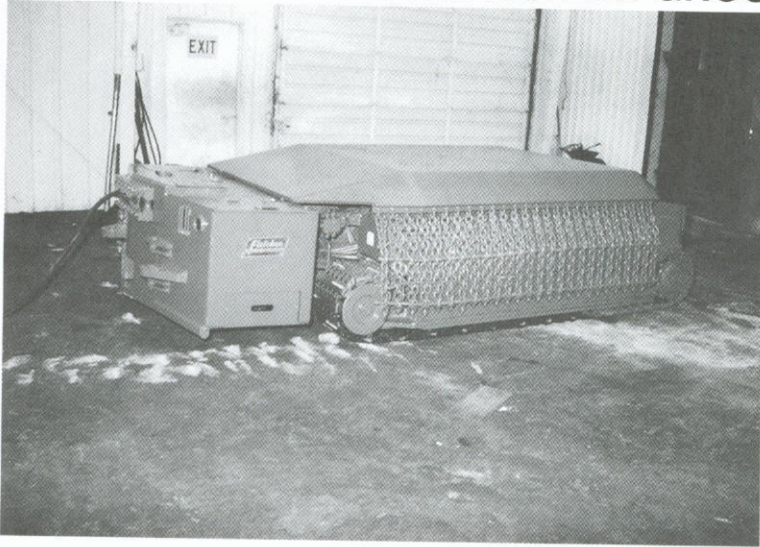


Figure 18



Figure 19



Figure 20
"800 Ton MRS on the Pillar Line"

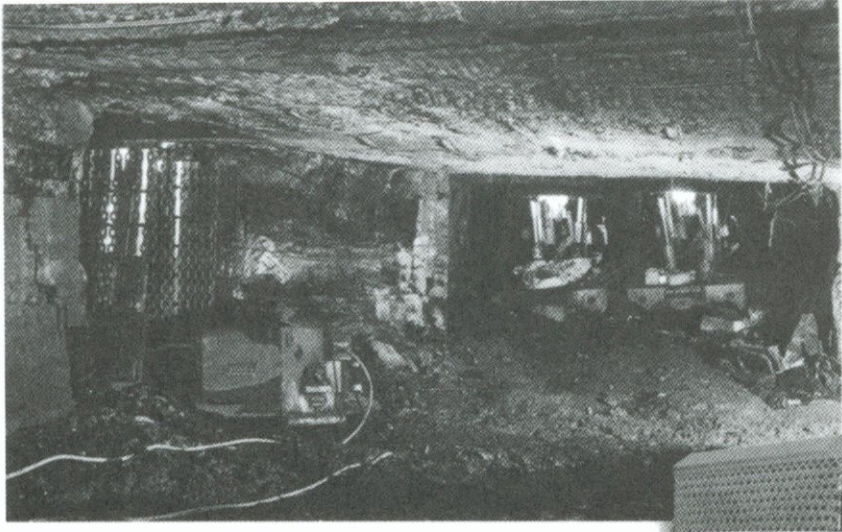


Figure 21
“800 Ton MRS set up for the final lift”

A Critical Study of Strata Behaviour During Extraction of Pillars in a Thick Coal Seam

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ABSTRACT

This paper presents analysis of strata behaviour with special reference to convergence and stress variation during an experimental trial of extraction of pillars with cable bolts as major support system in 6.5 - 8.0 m thick coal seam. Critical span for roof falls was estimated through empirical models and also evaluated by numerical models. In situ strata behaviour studies during 1992-96 for about 70 local/major falls revealed inadequacy of the available literature for interpretation and prediction of the roof falls. Convergence acceleration and variation of induced stress based on continuous monitoring data showed distinct anomalies and potential for better understanding of strata mechanics during pillar extraction.

INTRODUCTION

Global limits to consider a coal seam as "thick seam" change from 2.25 m in Japan to a maximum of 4.8 m in India. China, Poland, Turkey and Hungary adopted 3.5 m as the general limit, while the average of France, USA and Canada is in the range 3-4 m. Thus, 3.5 m has been accepted as the international limit for a thick seam (1). Extraction of thick coal seams, in general, and seams developed on pillars in particular, has posed serious challenge to the mining engineers. This is specifically due to insufficient resistance of conventional support system and associated strata control problems in openings higher than 4.8 m thickness in Indian mining scenario.

Bord and pillar mining being adopted in India since its inception is contributing nearly 95% of the total underground production. In all, strata movement has been accounted for about 30% of the total underground accidents due to fall of roof (2). Nearly 50% of the accidents are in depillaring areas, and about 15% are

due to abutment pressure. In all, 1/6 of the accidents are attributable to lack of prior knowledge of unsafe conditions and unavoidable. This indicates the need of detailed technical examination of methods of extraction and strategies to be adopted for improved safety, productivity and conservation. About 1,500 million tons of coal standing on pillars of thick seams in various subsidiaries of Coal India Limited (Fig 1) has been awaiting suitable technology and consistent experiments are in progress for optimum extraction of the otherwise unworkable reserve. A typical example of New Chirimiri Ponri Hill (NCPH) mine, Chirimiri area of South Eastern Coalfields Limited (SECL) is self revealing.

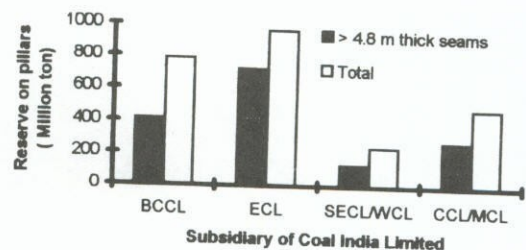


Fig 1: Coal reserve standing on pillars of thick seams in some of the Indian coalfields

The seam #3 of 6 to 10 m within NCPH leasehold of Chirimiri area was developed on pillar along the floor. Nearly 9 million tons of good quality coal has been locked in pillars, where roof coal band invariably caved in, endangering the safety of the workers and the workings. As a result, development height while formation of pillars was limited to 3 m, and the final extraction has been permitted up to 4.8 m. Therefore, overall recovery was only about 50% and decreases drastically with increase in thickness of the seam. At this juncture, an innovative methodology was conceived

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by Central Mining Research Institute in India with cable bolting as a major support system in one pass so far not possible by any new technology when the extraction of this reserve was in jeopardy. For the purpose, strata behaviour monitoring was deemed to be very essential, and four depillaring panels were monitored for about 1000 days during experimental trial of the novel concept.

EXPERIMENTAL PROGRAMME

The target of the strata behaviour studies has been to obtain guidelines regarding the following issues in the experimental trial:

- * Cables may loosen under the influence of roof coal blasting and may not serve as effective support.
- * Overriding of the extracted area (goaf) may occur with the weakening of stooks due to increase in height of extraction, and
- * The anchorage of bolts across the immediate roof may delay the goaf settlement and thereby transfer additional stress over the face and in advance.

To be able to clarify these issues, the research program is split in three steps:

- i) Evaluation of performance of cable bolts before and after blasting of the roof coal.
- ii) Qualitative and quantitative observation of strata behaviour in advance workings and goaf, and
- iii) Analysis of strata behaviour vis-a vis roof falls for better understanding of strata mechanics during extraction of pillars.

GEOMINING DETAILS

The NCPH Colliery is located in Surguja district of Madhya Pradesh, India (Fig 2). It is connected to Indian railways via Annapur and Manendragarh railway stations. In view of typical hilly terrain and eroded topography, continuity of five workable coal horizons is disturbed in the area.

Experimental trials were conducted in four panels; #15, #16, #17 and #18 of seam #3. The top-most #1 seam is exposed along the top of the hills and unworkable over the experimental site. Extraction of pillars for the time being was confined to Seam #2 and #3 only. Seam #2 was depillared over the experimental panel #18 with 30 m parting while other panels are devoid of any overlying workings. Seams have been approached through adits, and only 4.8 m bottom section of 6-10 m thick and flat coal seam #3 was being worked, leaving rest of the coal during final extraction of the pillars. Range of geominig

parameters in the experimental panels are as follows:

# Pillars in a panel	22 to 53
Seam thickness (m)	6.5 to 8.0
Panel size (m x m)	115 x 135 to 200 x 220
Pillar size (m x m)	22.5 x 22.5 to 30 x 30
Width of gallery (m)	4.2 - 4.8
Height of gallery (m)	2.5 - 3.0
Depth cover (m)	33 - 256 m

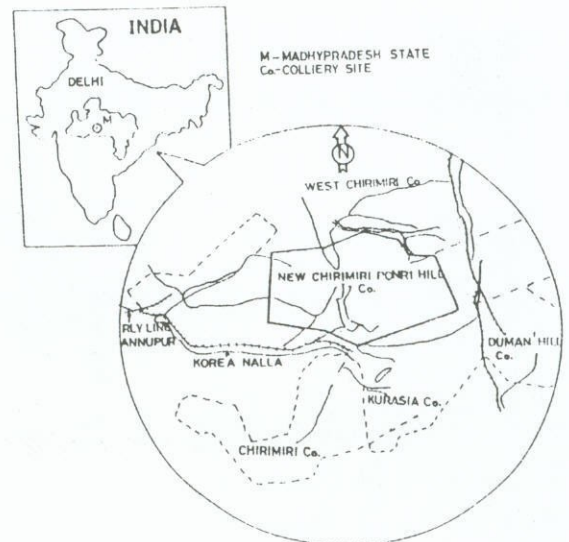


Fig 2: Location of experimental colliery site.

The seam is overlain by a thin band of shale and 10 cm thick graphite layer. About 3 m thick coarse grained sandstone roof formed the immediate roof followed by 12 m thick medium to fine grained sandstone with shale bands as main roof. Average Rock Quality Designation (RQD) and compressive strength of the immediate roof is about 50 and 20 MPa, respectively in depillaring panel #16. CMRS Rock Mass Rating @ for the coal roof is 56, whereas it is 70 for the sandstone roof (3). Comparatively poor roof conditions are noticed in other experimental panels.

METHOD OF MINING

The seams under the above geominig conditions are normally worked by hydraulic stowing in ascending order. Well established depillaring practices in India involved division of pillars in stooks and extraction of stooks by half moon, open end pillaring, split & fender pillaring, pocket & wing method of depillaring and slicing. However, the area of Central Indian coalfields is familiar with the last



alternative. In view of non availability of stowing material at the mine site, caving method of splitting and slicing was followed for the experimental trial, and coal evacuation by Side Discharge Loaders (SDL). Details associated with geomining domain, method of mining, strata monitoring and performance of the experimental trial in different panels is illustrated elsewhere (3). Conventional system of supports including props, cogs and cross stitching were used in addition to cable bolts in all the depillaring panels (Fig 3).

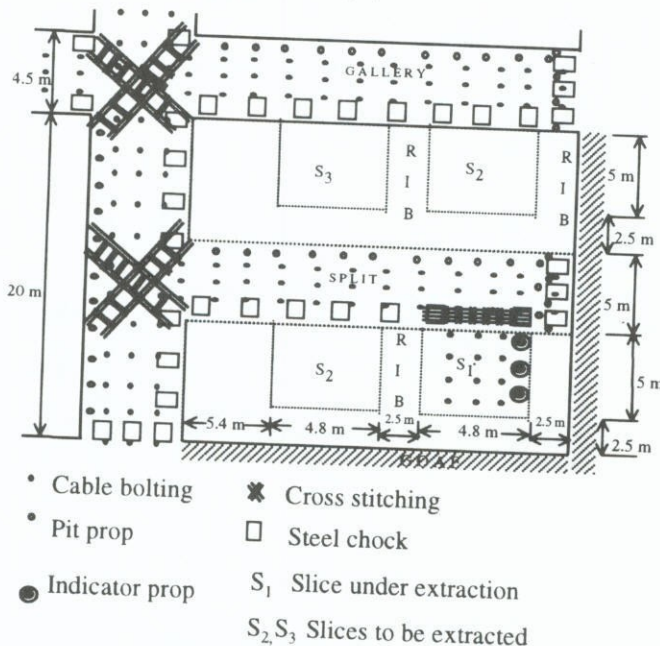


Fig 3: Support system adopted for experimental trial in 6.5 - 8 m thick coal seam

Extraction line was kept diagonal by simultaneous extraction of series of pillars; 1, 2, 3, 4 and so on as shown in Fig 4. Coal preparation at the face was with conventional drill and solid blast operations. Salient steps involved in the extraction of pillars are as follows:

- Cable bolting of all the development galleries at 1.2 m interval, and upto 1.5 m in the sandstone roof of the panel before start of the depillaring operation.
- Systematic Support of Roof (SSR) of the development galleries using pit props 1.2 m apart on one side, and cogs at 2.4 m interval on the other side of galleries upto two pillars in advance of the goaf line.
- Drivage of 5 m wide split gallery for dividing pillar in two halves and its SSR as practiced in development galleries including cable bolting.
- Drivage of 4.8 m slice from the goaf edge leaving 2.5 m thick fender (rib), and support of low height roof by pit props on the goaf side in addition to cable bolts.
- Support of slice edge by steel chock and cross bars,

and the goaf edge by skin to skin steel chocks. Support of junctions by cross stitching with eye bolts and tight laggings.

- Extraction of roof coal by blasting of cable reinforced coal band.

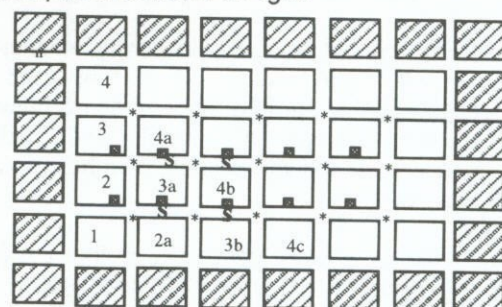
Verical support is removed from the depillaring area before blasting when the cable bolts remained as the only support for the heightened roof. As the safety of the present method is entirely dependent on performance of cable bolt of 1.5 m long remaining in sandstone roof after blasting of the roof coal, extensive pull tests were conducted as an integral part of method of mining to ensure its efficiency.

INSTRUMENTATION

Qualitative as well as quantitative nature of strata behaviour was observed since beginning of the experimental trial. Four panels were equipped with the following set of instruments during depillaring to full height.

- Convergence indicators (telescopic, suspension and remote type)
- Load Cells (mechanical and remote type)
- Borehole extensometers (magnetic ring and LVDT type)
- Stress capsules (Vibrating wire type)
- Strain meters (Vernier type)
- Pillar dialation indicators (Tape extensometer type)

The convergence and load cell stations of non remote type were installed at all the junctions before starting of extraction of pillars. Instrumentation in a typical panel is shown in Fig 4.



■ Set of remote type Convergence meter, load cell, Stress capsule and Borehole extensometer

S Set of strain and dialation meters

* Set of telescopic type convergence and mechanical type load cell stations

▨ Barrier pillar

1, 2, 3... Diagonal sequence of pillar extraction

Fig 4: Location of strata behaviour monitoring instruments in a typical extraction panel



For continuous monitoring of the strata behaviour parameters through remote type instruments (i to iv as above), computerized data logger was commissioned in panels #17 and #18. The system could be used successfully in panel #18 after rectification of some operational problems encountered in panel #17. To minimize the loss of monitoring data, regular manual record of the readings was also continued for all the transducers. About 70, 80, 40, 30, 10 and 5 numbers of instruments i to vi, respectively were regularly monitored during the trial.

Location of the instruments was adjusted such as to remain safe even after winning of the roof coal to full height. Sets of remote indicators of i to iv type were installed at strategic positions near mid of the panel (Fig 4); nearly two pillars in advance and monitored up to last when the rib failed or cable of remote connection damaged in the goaf. The cable from this were taken through conduit pipe laid in a notch made along the floor making all efforts to protect them against the SDL scourage.

EFFECTIVENESS OF THE SUPPORTS

Degreased 7 stranded old haulage ropes of 22 mm diameter having 22 ton breaking and 15 ton yielding capacity were used as tendons for cable bolting. A mixture of cement and water was used at 1:0.45 by weight for grouting in 43 mm diameter holes utilizing pneumatic injection pump (3). Hardcrete was used for quick setting of the grout in slices. Laboratory testing of the grout mixture indicated 10, 15 and 20 MPa compressive strength after 1, 7 and 28 days, respectively.

Effectiveness of cable bolting in the development galleries was analysed by many investigators (4,5,6). However, effect of blasting on the performance was not fully understood. Therefore, anchorage of cables up to 10 ton was ensured in a few galleries well in advance of the experimental trial, and regular anchorage tests were also conducted on 10% of about 20,000 cable bolts installed during the experimental trial. As Peng and Guo (7) put it "when the roof is consisted of strata of low deformation moduli or is subjected to high insitu stress or mining-induced abutment stress, perfect bonding may not hold because very large shear stresses are developed at the bolt/rock interface that exceed the shear strength and bond failure will inevitably occur. Obviously, under this circumstances, if the interface is still forced to conform with perfect bonding, it will lead to over-reinforcement of the roof". Similar opinion was put forward by Kaiser et al., (8). Maximum induced stress over the stook near goaf edge of the panel #15, #16, #17 and #18 was 1.1,

2.1, 1.8 and 24 MPa, respectively. The cables remaining in sandstone roof after blasting of roof coal under these stress conditions sustained pull load of 10 tons without any perceptible failure of the bond, indicating adequacy of the support system.

Rock mass classification -Q of Barton et al., (9) has been extensively used for estimation of support density for different coalfields of India (10). Model studies on strata reinforcement with bolting indicated improvement of bending strength of bolted roof (11), and attempts are also made by the first author of the present paper to consider the improved quality of roof for estimation of support density in depillaring panels. For the purpose, improvement of Q due to cable bolting (Qs) was estimated as per Trevor and Jose (12) following Eq 1.

$$Q_s = 12.5 Q^{0.45} \quad (1)$$

The rock load assessed for development galleries was 2.6 t/m² as per the weighted rock mass rating-®. Rock load and support density estimated with due consideration to improved quality-Qs of the bolted strata for panel #16 is presented in Table 1. This indicates rock loads of 2.8 t/m² for original galleries, which is in agreement with the load computed as per the rock mass classification-®.

Table 1
Estimated rock loads and support density for extraction of pillars in panel #16

Site	Likely rock loads (t/m ²)	Support density (t/m ²)	Safety factor
Splits/galleries	2.8	10.8	3.8
Junctions	6.2	9.8	1.6
Slices	4.9	5.7	1.2
Goaf edges	15.9	34.7	2.1

Safety factor exceeded 2 for the galleries/splits and goaf edges while slices and junctions were secured well within 1.2 -1.6 safety factor. Pit props of 20 ton capacity were set at about 1 t setting load. Maximum induced load on the pit props was 11, 12.5, 7.6 and 18 t in panel #15, #16, #17 and #18, respectively before occurrence of main fall indicating its adequacy in the geomining conditions.

STRATA BEHAVIOUR OBSERVATIONS

In the initial stages of depillaring till extraction of pillars 2 and 2a, influence of workings was up to 10 m in advance of the goaf line. The zone of influence

extended upto 30 m in advance during major roof falls while extraction of fourth series of pillars (Fig 4). For the purpose of this paper "roof fall" is defined as natural caving and subsequent settlement of the goaf. *Local falls* refer to caving of immediate roof while the term *major fall* denotes caving of the strata overlying the immediate roof. The goaf settlement was very smooth with ribs, cable bolts and the goaf edge supports playing major roll in keeping the caving zone away from the working places. Convergence and induced stresses observed at the time of roof falls is discussed below.

Convergence

About 250 records of convergence were available for different monitoring stations before local/major falls in the four depillaring panels. The distance between the monitoring station and the goaf edge (s) was in the range of 5 to 50 m, and about 30 records were also available for the convergence inside the goaf. Convergence along the goaf edge before first major fall in panel #15, #16, #17 and #18 are respectively 26, 20, 36 and 97 mm (Table 2).

Table 2
Convergence observations prior to first major fall in various depillaring panels

Panel no	Depth Cover (m)	Area of extraction (m ²)	Maximum convergence Along the goaf edge (mm)
15	33-58	4,100	26
16	37-70	5,900	20
17	46-103	4,900	36
18	119-256	5,400	97

About 60% of the falls are associated with rate of convergence (r) within 0.5 mm/day in advance galleries of the four panels. r value within 10 m from the goaf edge, prior to the roof fall was more than 2 mm/day in the incidence of about 3% of the falls. The number of monitoring stations located within 10 m, 10-30 m and more than 30 m from the goaf edge at the incidence of the falls are 75, 100 and 125, respectively. About 85% of the data sets showed less than 10 mm convergence along the goaf edge up to the time of roof fall. Influence of working on advance galleries beyond 30 m from the goaf edge was not perceptible.

Typical variation of convergence of a 2.5 m high and 4 m wide advance gallery during depillaring in panel #18 with seam thickness about 8 m is depicted in Figs. 5a-5c. Maximum convergence reached up to 97 mm within 40 days of observations during movement of goaf edge from 2nd to 4th series of pillars as in Fig 4.

Initiation of roof movement was not perceptible up to 23 days of monitoring. Roof fall occurred at 13:00 hours on the 17th December 1995. Fig 5a shows cumulative convergence vs, time in days from initiation of roof movement. Although the trend of convergence shows increased slope on 17th day after initiation of roof movement, such data could not be used for clear understanding of strata mechanics immediately prior to the roof falls.

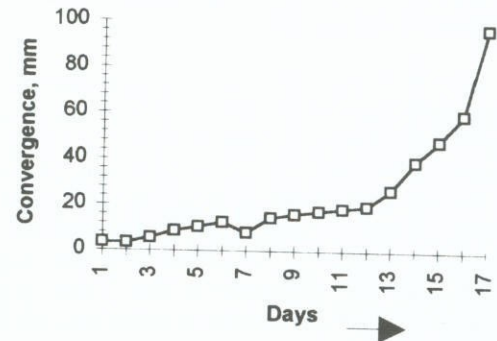


Fig 5a: Variation convergence prior to a major fall

In view of inadequacy of daily convergence plots in previous studies for interpretation of roof falls, convergence velocity and convergence acceleration with reference to short interval of time were also computed. Convergence velocity increased to 3 mm/hour with irregular variation since 14:00 hours on a day prior to the major fall (Fig 5b).

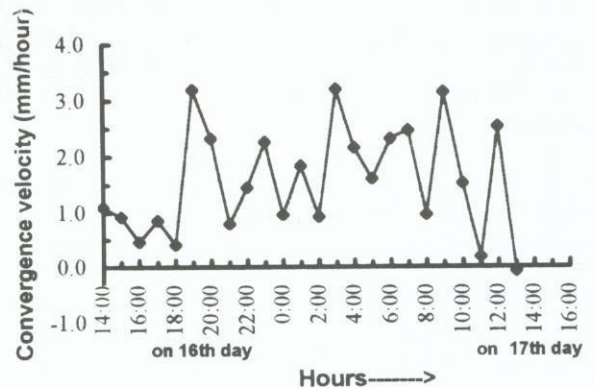


Fig 5b: Convergence velocity before a major fall

Although convergence velocity showed irregular trend indicating abnormal movement of roof, divergence phenomena associated with elastic deformation was apparent during the falls. Elastic deformation was also noticed in Jharia coalfields (1), but this phenomena is often missed and subsequently ignored due to non availability of continuous monitoring data. In view of the availability of continuous data in the experimental trial, variation of convergence acceleration at short intervals was also studied for better understanding of strata

movement. Convergence acceleration was within 0.5 mm/min² upto 11:00 hours, and drastically increased thereafter (Fig 5b).

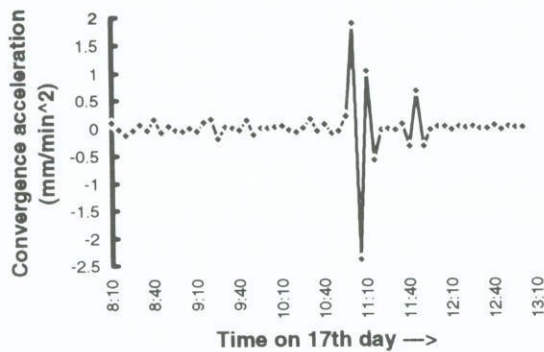


Fig 5c: Convergence acceleration before a major fall

The above figure shows about 2 mm/min² convergence acceleration indicating distinct anomaly at 11:00 hours; 2 hours prior to the fall. This observation indicates scope of prediction of roof falls and warning system for the purpose. Variation of induced stress also showed distinct anomalies and supported the potential application of continuous monitoring system for prediction of fall.

Induced stress

Variation of stress over the rib surrounding the above convergence station and located 25 m inside the goaf is presented in Fig 6a & 6b. Induced stress reached up to 25 MPa at 10:30 hours. Destressing phenomena associated with divergence is very prominent before the fall followed by another peak at 13:00 hours with maximum induced stress of 28 MPa.

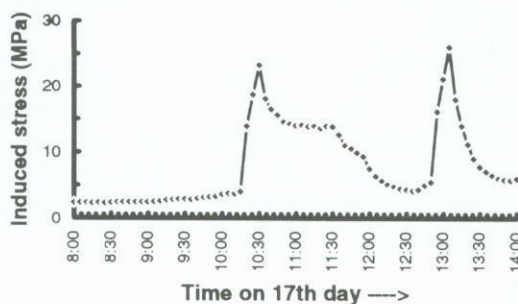


Fig 6a: Variation of induced stress over a rib prior to the major fall

Variation of induced stress was about 2 MPa/min² at 10:30 hours followed by destressing to 1 MPa/min². However, at 13:00 hours, variation of induced stress was exceeding 2 MPa/min² (Fig 6b). This phenomena of stress anomalies combined with convergence acceleration may be used for prediction of the roof falls and cosequently extended to give prior knowledge of

the unsafe conditions to minimize accidents during depillaring. Although microseismic techniques are useful for the purpose, similar anomalies could be recognized through the continuous strata monitoring for interpretation of strata behaviour during roof falls.

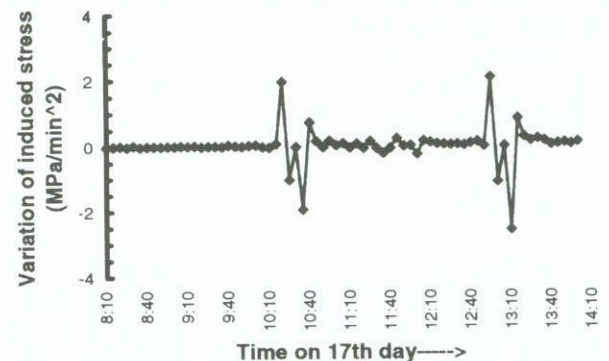


Fig 6b: Variation of rate of induced stress over a rib prior to the major fall

STRATA MECHANICS

In view of extensive qualitative observations and theory, the roof strata fail due to tensile fractures, while the sides suffer with shear failure. Based on such observations and previous studies on strata mechanics, attempts are made for estimation of critical span before roof falls, trend of roof movement and stress distribution through observational, empirical and numerical approaches in the present geominig conditions.

Critical span for roof falls

Nether roof strata in goaf before first major fall behaves as a beam clamped around by the panel barrier and the goaf edge support/rib adjoining the working face. Using this analogy, critical span for a competent layer clamped all around under uniformly distributed load can be derived on the basis of tensile strength of the competent layer. However, the end conditions of clamped beam will change to an edge supported beam due to development of tensile cracks on the upper surface near the ends of the beam.

Beam analogy has been discussed in different forms to understand the strata mechanics by many investigators. Singh (1) says "A mine roof, even if resolved to an equivalent beam, differs from a fixed end, self loaded beams as it always carries a load of the overlying mass until delamination due to differential deflection. By that time, the bending strength drops to a certain residual value". Accordingly, stability index was proposed and used for longwall panels. However, it was not applied for depillaring panels. Similarly, cavability Index developed at CMRI was also not put to use for

pillar extraction due to empirical nature of relations based on longwall mining.

Based on plate theory, equivalent face advance (A_{eq}) was established and statistically applied to nine of longwall and six of depillaring panels in Indian geomining conditions (13). The analysis was also made more general by considering the Q classification, which takes in to account of several other joint parameters besides RQD of the rock mass:

$$A_{eq} = 9.12 Q^{0.42} \quad (2)$$

For the geomining conditions of panel #16, A_{eq} has been estimated, and corresponding critical span is 42 m as per the plate theory. In practice, local falls occurred at 15 m span of the goaf while main fall at 45m span in conventional depillaring of previous worked panels without any cable bolts. Unlike these results local falls in the experimental panel #16 occurred at about 25 m span, while main fall occurred at about 45m span in the goaf indicating delay of local fall due to cable bolting of the nether roof.

Empirical models

Prediction of strata behaviour by theoretical analysis become unreliable due to almost impossibility of simulation of the real field conditions in mathematical, physical or numerical models. Thus, empirical formulation, based on in situ measurements of strata behaviour parameters, is an accepted way to estimate the strata behaviour. Initial expressions representing the behaviour of openings were usually time-dependent, logarithmic or exponential functions derived based upon curve fitting to the experimental data obtained from convergence measurements carried out in underground.

The trend of gallery closure with reference to bord and pillar development was empirically related to rock mass rating, roadway span and rock dry density (14). Unal (15), Ozkan et al., (16) determined that the roof behaviour can be best represented by an exponential function considering the parameters; the face advance coefficient- A, the time dependent coefficient-B, and a constant controlling the duration where convergence reaches to its maximum value-c. A constant depending on rock-mass quality, roof span and stress factor was also established.

Jayanthu et al., (4) investigated the behaviour of coal mine galleries supported with roof bolts and cable bolts based on regular monitoring of 10 convergence stations, 6 bore hole extensometers and three tensemag- instrumented cables. Cable bolting was

found to be effective in controlling adverse strata behaviour while reducing the convergence of the development gallery by 50% as compared to the conventional roof bolting. When Ozkan et al., (16) criteria was applied to these conditions, maximum convergence could be estimated within 5% deviation while the convergence upto 60 days of development was overestimated. The coefficients A,B and C for these conditions are found to be 2.9, 13.7 and 6.8 respectively. Maximum tensile load of about 4 tons was measured on the cable (6).

The above studies are based on observations in development workings. These are not suitable for prediction of convergence in advance galleries due to simple logic that influence of the extratcted area is not considered as exists in depillaring operations. It was observed that cumulative convergence of advance galleries due to increased area of goaf is generally influenced by the following parameters:

- i. Distance from goaf edge (s)
- ii. Period of observation
- iii. Characteristics of overlying strata
- iv. Span in the goaf
- v. Depth cover
- vi. Goaf treatment
- vii. Extraction height
- viii. Width of the advance gallery
- ix. Method of extraction
- x. Support system

Detailed analysis of the strata behaviour with special reference to convergence during extraction of pillars in 8-10 m thick seam lying at a depth of 120 m, indicated quadratic relation between face advance and convergence (17). Observational data was related to hourly monitoring of convergence by microprocessor based instruments during 1995-97 in two extraction panels. For the present data of panels #15 and #16 at a depth cover within 70 m, the parameters v to x has no big variation in the study area. Roof movement in advance galleries was found to be in good correlation with the face advance and the general trend of movement with distance from the goaf edge also showed quadratic relation (Fig 7).

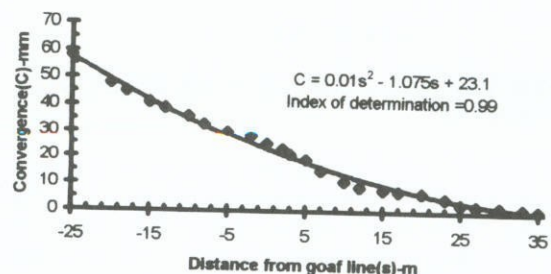


Fig 7. Variation of convergence during depillaring



Numerical models

Numerical models with provision of continuum analysis were found to be useful for better understanding of stress distribution while discontinuum models have scope for wide application. Various stages of depillaring in the experimental trial was simulated in numerical models using finite difference code of continuum analysis. Analysis of strata behaviour during pillar extraction was carried out at depth cover (H in m) ranging from 50 m to 200 m (3). The model assumed simple mine geometry and did not include any previously mined galleries in the vicinity of the depillaring panels. In the absence of the insitu stress measurements in the coal field, the following norms were adopted for estimation of insitu stress field prior to the excavation of the area.

$$\begin{aligned}\text{Vertical stress} &= \gamma \cdot g \cdot H \\ &= 0.025 H \text{ Mpa}\end{aligned}\quad (3)$$

$$\text{Horizontal stress} = 3.75 + 0.015H \text{ Mpa} \quad (4)$$

where,

$$\begin{aligned}\gamma &= \text{specific weight of overlying rock mass (2.5 g/cm}^3\text{)} \\ g &= \text{Acceleration due to gravity (9.81 m/s}^2\text{)}\end{aligned}$$

Bolted rock mass was considered as a homogenization media for simulation of the immediate roof. Improvement of Young's modulus of rock mass due to bolting [E (h)] was estimated as per Bernaud et al., (18):

$$E(h) = E(r) + V(r) \cdot E(b) \quad (5)$$

where,

$$V(r) = \text{Voluminous proportion of reinforcement}$$

$$E(b) = \text{Young's modulus of bolt material}$$

$$E(r) = \text{Young's modulus of rock mass}$$

Maximum vertical stress over the developed pillars at 53 m depth cover was 1.7 MPa, while it was 2.0 MPa over a stook after splitting. Maximum stress reached up to 5.5 MPa over a rib within the goaf immediately before the first main fall conditions i.e., after extraction of two rows of pillars exposing nearly 50 m span in the goaf. However, the maximum stress experienced by a rib during the first local fall condition remained within 4.5 MPa. At 200 m depth cover, maximum vertical stress over the rib with 50 m span in the goaf was about 9.5 MPa, while the stook adjoining the goaf experienced up to 1.4 MPa.

Distinct element methods were used to model the discontinuum behaviour [strata collapse, slip,

separation on bedding planes, and cracking across the strata] which forms a key part of the mechanical response of rock mass to mining, treating the situation as plain strain problem (19). The extent of zone of yielding was up to about 30 m in advance workings, when two rows of pillars are extracted. Zones of tensile failure were negligible till a pillar is extracted, while tensile failure of the nether roof started after exposure of about 40 m clear span in the goaf. After extraction of two pillars exposing about 50 m span, major fall conditions were observed. Preliminary analysis on limited observations of stress based on fracture mechanics by Kramer (20) showed comparatively more stress over the pillar/stook/rib adjoining the goaf. Attempts are being made by first author of the present paper for further analysis of strata behaviour through application of fracture mechanics based on extensive data on stress variation during depillaring in Indian geomining conditions.

PREDICTION OF ROOF FALL

Prediction of roof falls in goaf during extraction of coal was attempted in Indian geomining conditions by many investigators on the basis of convergence monitoring data (13,21). Empirical indices for prediction of falls are based on daily convergence data. Exact information about sharp changes in convergence a few hours before and after the fall was not available from the manual systems. The following indices were applied to the daily convergence data, and the variation of these indices was studied for prediction of roof falls and a criteria is postulated based on continuous monitoring data:

$$r / r' > 2.0 \quad (6)$$

$$r - r'' / r' > 1.5 \quad (7)$$

$$r - r''' / r' > 1.5 \quad (8)$$

$$\frac{R - R'}{R' - R_0} \times \frac{N - 4}{4} > \text{cut off value} \quad (9)$$

where,

r' = Average daily convergence at the site up to the previous day of fall

r'' = Daily convergence on the previous day of fall

r''' = Daily convergence on a day when the ratio (as in eq. 7) is less than 1.5

R = Last reading before fall indicating cumulative convergence (mm)

R' = Reading of the fourth day prior to the last reading

R_0 = Initial reading in mm prior to initiation of movement



Variation of indices for prediction of roof falls

The eq. 6 could predict only 50% of the falls, while eq.7 has probability of indication of 25% of the falls. The condition of eq.8 was applicable in 60% of the cases. About 60% of the falls occurred within four days of eqn.8 value exceeding 2. The following criteria was also applied to two panels extracted by blasting gallery method in 8-10 m thick and found to indicate 80% of the falls with cut off value of 3 (17).

$$\frac{Rh - Rh'}{Rh' - Rho} \times \frac{Nh - 3}{3} > \text{cut off value} \quad (10)$$

where,

- h = Rate of convergence in mm/hour
- Rh = Last reading before fall indicating cumulative convergence (mm)
- Rh' = Reading of the third hour prior to the last reading
- Rho = Initial reading prior to initiation of movement in mm

Analysis of convergence data of the experimental trial in the light of the above criteria showed 70% probability of prediction of roof falls. However, analysis of the continuous monitoring data related to various parameters for the depillaring panel #18 showed potential of convergence acceleration for indication of impending fall three hours prior to its occurrence. Interpretation of the data vis-avis roof falls has been found to be more appropriate by convergence acceleration and stress around the workings as illustrated in Figs 5c and 6b. Although continuous monitoring system was applied during wongawalli system of pillar extraction by Follington and Hutchinson (22), the data was limited to the goaf edge supports only. This typical phenomena of convergence and stress through continuous strata monitoring for the first time during depillaring may be used for further understanding of strata mechanics.

CONCLUSIONS

Strata behaviour has been critically studied with special reference to variation of stress, convergence and efficiency of support system during the experimental trial of extraction of pillars with cable bolting as major support in four panels of 6.5 – 8.0 m thick coal seam. Cable bolts are found to be effective under the influence of roof coal blasting. Anchorage of bolts across the immediate roof delayed the local fall. Local and major falls occurred after exposure of about 25 m and 45 m critical span in the goaf.

Convergence of the advance galleries and goaf followed quadratic relation with respect to the distance from the goaf edge within the limitations of geominig conditions of the trial. These relations are being modified considering additional parameters. Potential use of convergence velocity and acceleration combined with variation of induced stress on the basis of continuous monitoring data is demonstrated for prediction of roof falls three hours prior to its occurrence and better understanding of stratamechanics.

The experimented method has a scope of application to over 800 million tons of coal in Indian coalfields, and also in suitable conditions elsewhere. In view of interchangeability of rock mass classification systems, studies on strata mechanics with special reference to extraction of pillars may be integrated through collaborative projects for global understanding and adaptability of the thick seam mining technology.

ACKNOWLEDGEMENT

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Progression of Longwall Gateroad Support as Conditions Change at Lodestar Energy's Baker Mine

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ABSTRACT

Longwall gateroad support can be supplied by a variety of methods. The system of support also needs to be flexible as conditions change. This is especially true in the complex geological conditions of the west Kentucky #13 seam. Methods of gateroad support used in this seam will be presented along with the successes and failures of the systems as conditions changed.

Lodestar Energy's gateroad support progression through the seven longwall panels retreated to date will provide an insight into planning for future panels.

INTRODUCTION

The Lodestar Energy - Baker Mine is located in western Kentucky. The mine is in Webster County near the towns of Dixon and Clay.

This mine is in the Kentucky #13 seam which has slickensided claystone immediate roof and a sandy shale main roof.

The geology in the area is made complex with the occurrence of sandstone lenses in the interburden between the #13 seam and a 1.5 foot thick rider seam. The rider seam also varies in distance above the primary seam from 6 feet to more than 30 feet. The proximity of the rider seam affects the strength qualities of the main roof. Also present in this mine are linears which have provided challenging problems for longwall gateroad support design. Another factor which played a key role in support is a high horizontal stress that develops cutter roof during the development of the entries.

The depth of the seam in the area of the Baker Mine ranges from 800 feet to just more than 1000 feet deep. The coal thickness is variable and ranges from 4.5 feet to more than 9 feet. The Clay Channel cuts the reserve to the eastern boundary and several smaller channels along with the Rough Creek Fault limits the reserve to the North.

The Baker Mine was primarily mined by the room and pillar method until February of 1995 when the first longwall was introduced to the seam. The success of the longwall has kept this mine operational during difficult marketing times (Fig. 1).

BLOCK 1 (The First Three Panels)

Standard Block 1 Support

The first block was laid out to fit between a main and previously second mined room and pillar panels. The panels were 8000 feet in length and the face width ranged from 680 feet to 710 feet. The gateroad support for the first panels was set up to use a system of $7/8$ inch rod trusses on four foot centers in both the headgate and the tailgate. The truss system was enhanced with two rows of 6 by 8 inch timbers on five foot centers. This system was used in the headgate and tailgate of Panel "A" in the first block. The headgate used Tensar mesh above the trusses to control loose rock in the headgate entry where personnel traffic is the heaviest. The gateroad support is shown in Figure 2.

Gateroad development is key to keeping a successful longwall running. The "B" panel headgate was being developed during the development of "A" panel's gates and was 4000 feet into its development when the first panel started its retreat.

Problems in First Panel - Horizontal Stress

Problems that showed in the first panel were not severe but showed that the system needed some improvement. The first problem was horizontal movement in the roof. It was anticipated that this panel would experience the most severe stress on the tailgate corner as the panel retreated. We believed that the trusses would help the control of the cutter material and the progression of the cutter in the concentrated stress field. What we witnessed was enough lateral shift in the first four feet of the roof to shear some of the angle bolts, but in most instances the shift was great enough to dislodge the truss hoop from the shoe. This caused the trusses to drop to the floor as the longwall face approached. See Figure 3. This failure occurred in a zone 10 to 100 feet outby the

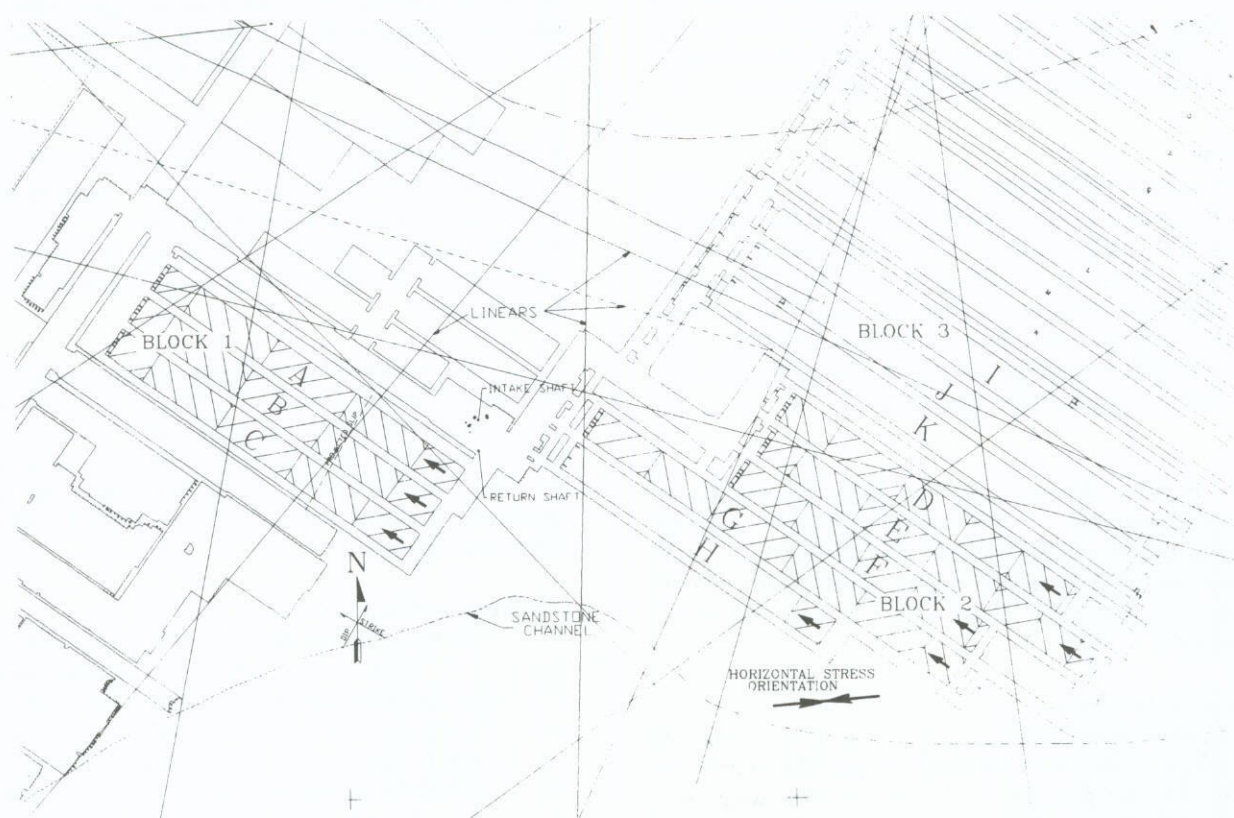


Figure 1
Layout of Longwall Panels Block 1 Through 3

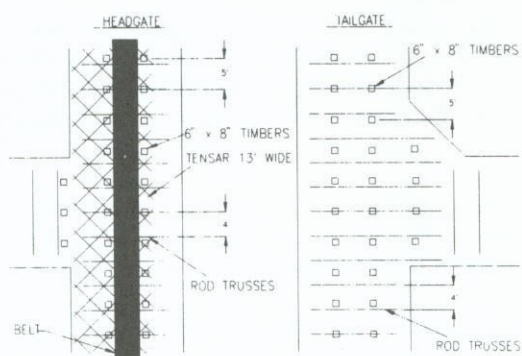


Figure 2
"A" Panel Gateroad Support

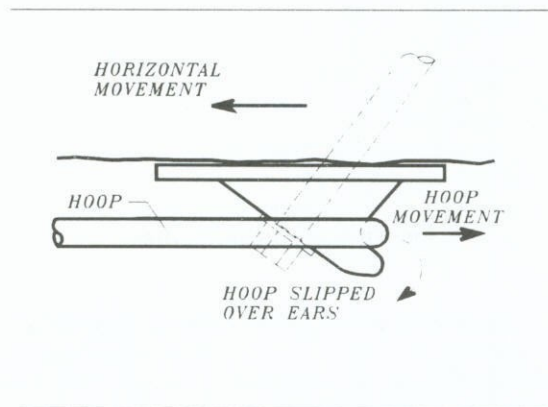


Figure 3
Rod Truss Failure Mode

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longwall face. Not every truss acted in this fashion but more than 40% of trusses on the first panel failed prior to the face passing the truss. This did not result in any failures of the first tailgate. However, it did cause concern especially in intersections.

Floor Heave

The second problem was with floor heave. In the first 1200 feet of retreat of the first panel, floor heave destroyed the timbers set in the tailgate entry. The timbers punched into the floor or were shifted off of vertical to the point where they provided very little support. In areas where floor heave was minimal or nonexistent, the timbers worked as planned. This problem was addressed with the use of cribs in the first 1000 feet of retreat on the tailgate in subsequent panels.

Linears

The first block of panels had several linears that had shown severe roof conditions in the mains and the adjacent developments. Roof support was enhanced in these areas with trusses on two foot spacing and the incorporation of cribs instead of the timbers. This method met with good success in the first block of panels.

Cutter Material Blockage

The cutter on the tailgate caused more problems than the control of the rest of the entry. This problem caused us to invoke our tailgate blockage plan due to the material that would be generated along the cutter. The material accumulated to the point where face personnel felt they would not enter the tailgate due to conditions of that corner. See Figure 4.

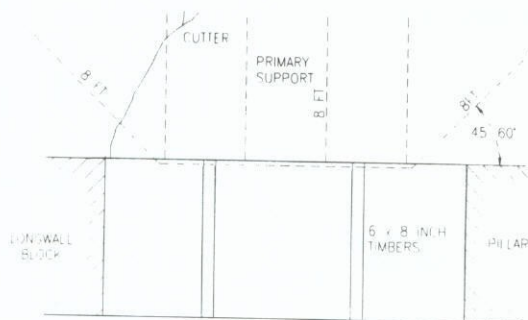


Figure 4
Location of Tailgate Cutter

Our problem was that wood secondary support could not be set close enough to the rib to control the cutter material. The shearer would cut out any wood before the access from the face reached that zone. The rod trusses that did not fail seemed to help control this material. The fail rate was a concern. This problem inspired the testing of cable trusses as a possible answer to the rod truss failure problem.

Headgate Support System

The support design for the headgate entry of the longwall was successful from the outset. The system consisted of rod trusses installed on four foot centers with a thirteen-foot wide

Tensar mat installed the length of the headgate on the travelway side of the longwall beltline. In addition to this support, two rows of 6" x 8" timbers were installed along the beltline.

During the retreat of the first panel, the use of the timbers the entire length of the beltline was an obvious over design and was decreased to just timbering intersections along the beltline for the remainder of the development of Block 1. The timbers were removed during the retreat of the mobile tailpiece and used as support in the crosscut between the headgate beltline and the adjacent entry.

The reaction of the headgate support was different than that experienced in the tailgate. The headgate was protected from the horizontal stress that was concentrated in the tailgate. This shadow zone from the stress showed less material spalling from the cutter and no trusses being dislodged from horizontal movement. The condition of this entry was ideal in all situations experienced in Block 1.

First Testing of Cable Products

Test areas were established for the use of cable trusses at the end of "B" panel in the headgate entry and the future tailgate of "C" panel and in 4000 feet of the last headgate in Block 1, "C" panel. The test areas showed that the cable truss installed in line with the primary support did not fail prematurely during horizontal shifting and worked successfully in the shadow of adjacent gob areas. The cable trusses used in these areas were three-piece trusses with two eleven foot angled cables and a thirteen-foot horizontal member. The cable thickness tested was 0.6 inches. Pull tests with 3.5 foot resin equivalent in a 1 3/8 inch finished hole broke cable strands between 28 and 30 tons.

BLOCK 2 (The Next 5 Panels)

Geological Features

The next five panels had several geological features that posed problems not faced in Block 1. Block 2 had areas where the rider seam dropped down to within 10 feet of the primary seam. The roof material was expected to be less stable than the typical Kentucky #13 roof.

Another feature not prevalent in Block 1 was areas of thick underclay. These areas were not identified adequately by the core drilling that was completed in this block of panels.

This block of panels also encountered the effects of the Clay Channel to the east. The channel provided for splits in the coal and increased seam heights. Also, the occurrence of sand lenses in the main roof increased.

The feature that had the largest effect to change the gateroad support in this block of panels was the linears. See Figure 5. The linears at the mouth of panels "D," "E" and "F" caused such instability in the main roof material that these panels would have a progressive evolution in intersection support that is not being used in any other mine to this author's knowledge.

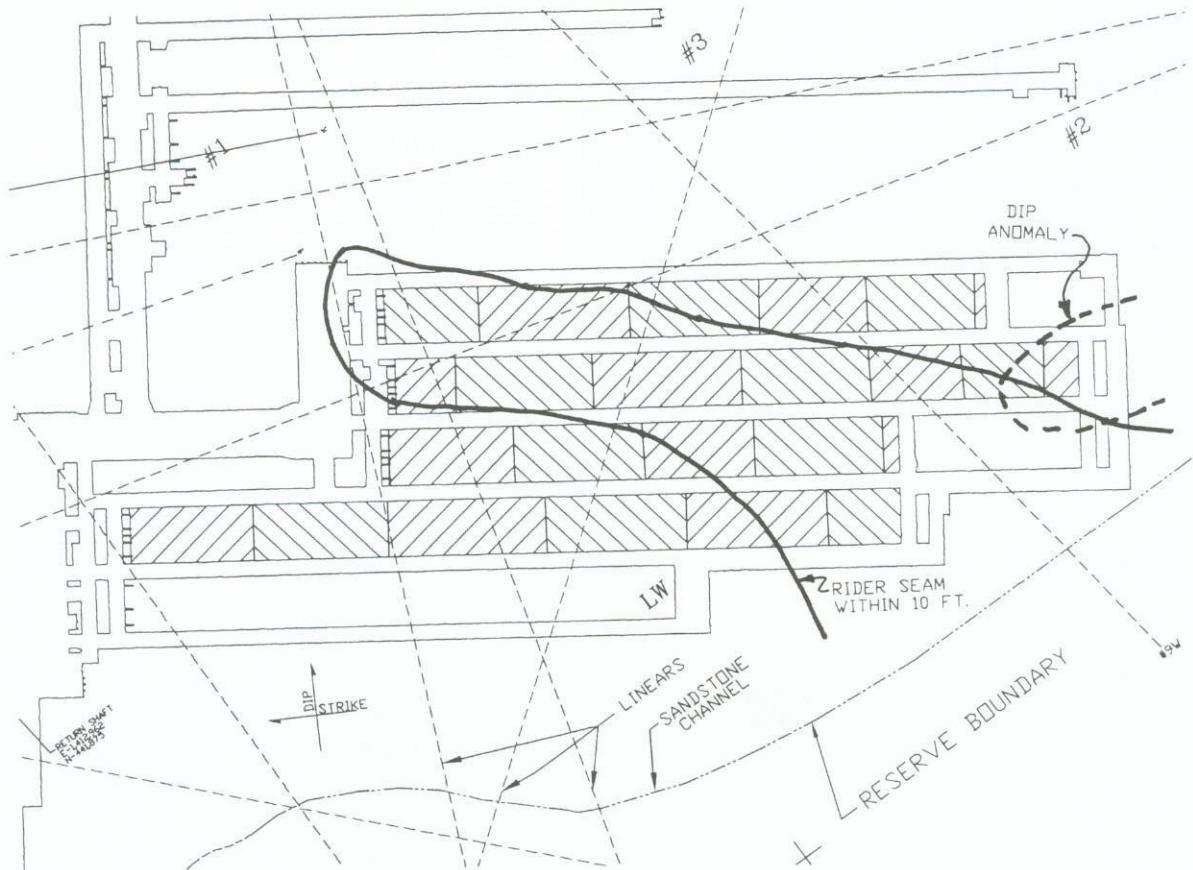


Figure 5
Geological Hazard Zone Locations
in Block 2

A problem that occurred in this block of panels provided a lesson in the adverse effects of horizontal stresses when they are concentrated by the staggering of panel startup locations. These areas are identified in Figure 5. Each one of these issues will be covered in more detail.

Standard Gateroad Support for Block 2

The gateroad support in the first three gateroads in this block started out with cable trusses on four foot centers. The truss spacing was decreased to two foot centers in areas identified as linears. The zone of tighter spacing was identified to be 250 feet on either side of the projected linear. Four point cribs were installed in the intersections of the tailgate replacing the timbers. The non intersection portion of the tailgate was still trussed on 4 foot centers with cables and two rows of 6 by 8 inch timbers were installed on five foot centers. The headgate support remained the same except for the use of cable trusses. See Figure 6.

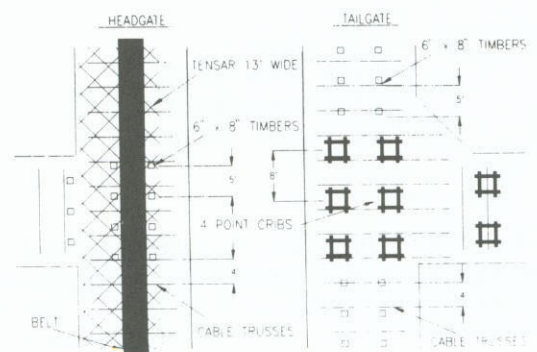


Figure 6
Typical Gateroad Support
at the Start of Block 2

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First Modifications to Intersection Trussing

Weak intersections became more evident due to the influence of the rider seam. Time related falls started to become a common place occurrence. A separation would occur at the rider seam resulting in a roof fall. The primary support was a two-piece 8 foot long, point anchor bolt. The bolts were grade 75 - $\frac{7}{8}$ inch rebar. The trussing pattern was modified in the intersections to incorporate long vertical cables installed in the center of the intersection. Two patterns were instituted. The first was used in all the tailgate intersections and in headgate intersections in linears. See Figures 7 and 8.

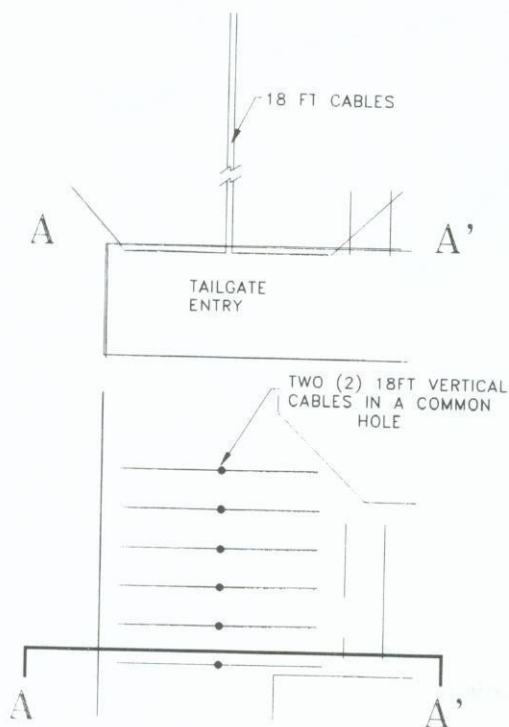


Figure 7
Cable Truss Pattern
In Tailgate Intersections and
Linears in the Headgate

The pattern in Figure 7 worked very well in the intersections in the tailgate. So well that the intersections were for the most part the best conditions in the tailgate. The second pattern was used primarily in the middle entry in the gateroad.

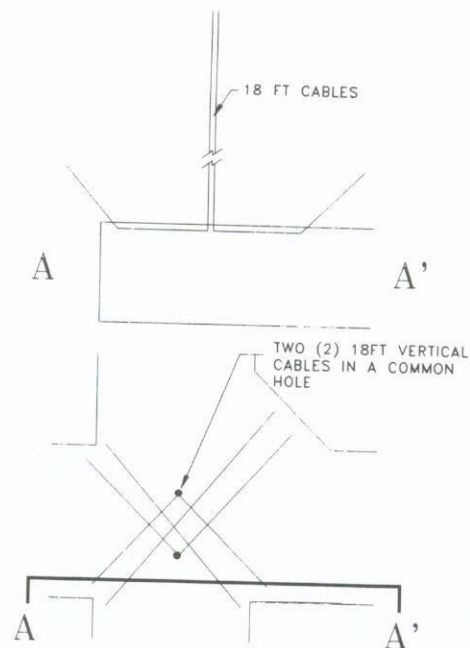


Figure 8
Cable Truss Pattern Using
Long Cables in All Other Gateroad
Intersections and Main Development

Additional Support on Block Perimeter

The first panel tailgate gateroad for "D" panel also had increased support in the outside entry against the solid block of coal. Sporadic roof falls in the first panel tailgate gateroad of Block 1 established the need for a better support plan for this entry. The decision was to support this entry just like the headgate belt entry. The entry was trussed with cables and meshed with Tensar. This entry was also timbered with two rows of timber on five foot centers. The intersections in the middle entry were cribbed on retreat. See Figure 9.

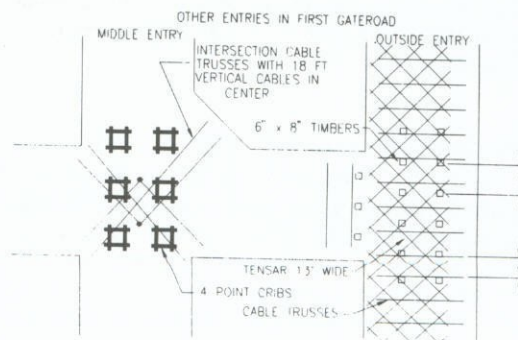


Figure 9
Support Pattern in Outside
Entry of First Tailgate Gateroad

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The use of this pattern did not achieve the expected results. Roof falls migrated from the gob through the crosscuts. Often they were stopped at the intersection cribs. However, several falls continued over the intersection cribs extending to the outside entry. The trusses for the most part held their own. However, due to a weakness in the pattern and no additional secondary support in the mouths of the crosscuts, other than carrier trusses, several intersections fell to the point where travel access to the back of these panels became too dangerous.

First Rider Seam Influence

During the retreat of the first panel in Block 2, it became evident that dead load conditions with separations at the rider seam and the thick underclay zones were too much for the timbers being used. The zones between intersections were for the most part littered with broken timbers. The loss of that support made the cable trusses carry more of the load than they were designed to support. Cable failures and severe cutting into the roof became a common sight in the entries between intersections. However, the intersections remained stable.

The gateroads being developed during this period continued to use intersection trussing and the cribs. The entry between intersections was no longer being trussed. This area was now being supported with Propsetter timbers and a mixture of four point cribs. See Figure 10.

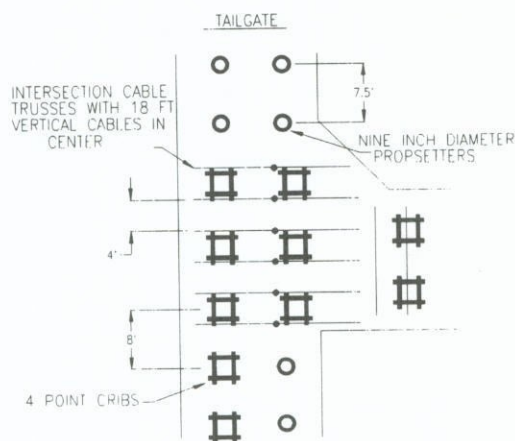


Figure 10
Modified Tailgate Support
Using Propsetter Timbers and Cribs

Block 2 Problems - Horizontal Stress

The retreat of "D" panel, the first in Block 2, saw several problems develop. The first was the high horizontal stress that was concentrated on the tailgate. This stress concentration kept the cutter on the tailgate active with large amounts of roof material falling out between the rib line and the first primary support. This activity caused the tailgate blockage plan to be enacted on a more frequent basis than in "A" Panel of Block 1. The cable trusses worked better than the rod trusses, but due to the failure of the timbers, they were not enough to control this entry. In the crosscuts, as the panel approached the cutout, several roof falls

occurred in the zone where the rider seam was closer than 10 feet above the main seam.

Staggered Panel Startup Areas

The retreat of "E" panel was the first panel with a staggered start up location. The stagger was considered due to additional coal reserves being located when drilling to locate the Clay Channel. The stagger created the problems of first panel horizontal stress effects and also the effect of concentrating the stresses in a closing window condition. See Figure 11.

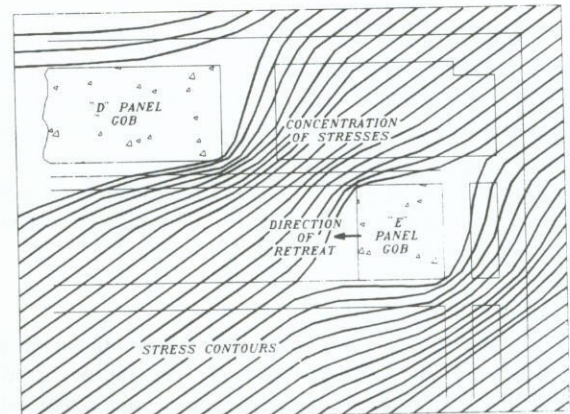


Figure 11
Stress Concentration in Staggered
Panel Startup Locations

As the longwall approached the startup location of the previous panel, horizontal stresses became the worst experienced in the five panels mined to this point. The cribs installed in the intersections were shifted as much as two feet off of vertical and the timbers installed on development of the tailgate had to be replaced with cribs recovered from the middle entry of the adjoining panel. This location was the first real roof fall that occurred on the tailgate of the longwall. The severe conditions followed on the tailgate until the face mined through a linear that crossed through the panel close to perpendicular to the horizontal stress field. Once through the linear zone conditions improved on the tailgate to the point that cutter roof material accumulation almost stopped. However, since the problems started on the tailgate, the six by eight timbers were replaced with nine inch Propsetters the remainder of the retreat of this panel.

Zone of Anomalous Seam Dip

Problems occurred in the tailgate of "F" panel even before the "E" panel was completely retreated. The first two thousand feet of the "F" panel tailgate began to show signs of closure before "E" panel had reached midway on retreat. An anomaly in the dip of the coal seam occurred in this area where the tailgate for "F" panel started squeezing. The floor clay was thicker and the coal seam was also thicker in the dip anomaly.

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Gateroad Failure Prior to Longwall Retreat

The factors caused by the dip anomaly accumulated to the point that the pillars rashed severely and floor heave closed the entry from a mined height of more than nine feet to less than seven feet. This movement prompted the installation of cribs in addition to those installed on development. Even the cribs installed at this time did not control the closure. When the entry had reached less than six feet in several areas, and there was no room for any additional secondary support, this area was abandoned as part of the "F" panel. New setup entries were driven to set up the "F" panel faceline.

Within a week of the decision to move the startup, the original startup faceline fell in two places. This event was the first fall on a startup faceline. The proximity of the rider seam was identified as the cause. The original faceline for the "F" panel had been open longer than any faceline developed to that point. The fall on that faceline inspired the use of long center cables in all intersections along the startup faceline. This pattern is the same as that used for tailgate intersections

First Failure in the Headgate

The first roof fall on the headgate beltline occurred in the last 1000 feet of the "E" panel retreat. This fall occurred in a linear on the inby side of an intersection. After reviewing the support in the area of this fall we determined that the trusses installed in the standard pattern of Block 1 and the start of Block 2 were not adequate to prevent the occurrence of a fall due to the nature of irregular roof in this area. The truss pattern had been tightened to two foot spacing for the linear, but the material still fell. The cross piece cable was not in contact with the roof and in some instances was in excess of two feet from the roof. The modification of center vertical cables would have provided additional center entry support and more roof contact. The support in linear zones in the headgate was then modified to the same pattern installed in the tailgate intersections through the entire projected linear zone. See Figure 7.

Modification to the Cable Truss Intersection

After three failures occurred in developing areas of the mine of the intersection design (shown in Figure 8), a design alteration was incorporated to enhance the support in the center of the intersection and reduce the installation cost. The enhancement went in two steps. The first enhancement incorporated twelve holes. Four of the holes in the center of the intersection were eighteen feet in depth and eight holes on the corners were eight feet in depth. This method worked so well that it was incorporated mine wide, both in mains and gateroads. See Figure 12.

The design was again modified to reduce cost and installation time. The layout is basically the same, but to reduce the number of holes and total feet of hole drilled, each corner only has a single hole drilled to a depth of ten feet. This change reduced the total feet of drilling by twenty-four feet. The cost of the cable installed in these holes was also reduced, due to the use of two bare cables in each hole instead of cables with resin cages. See Figure 13.

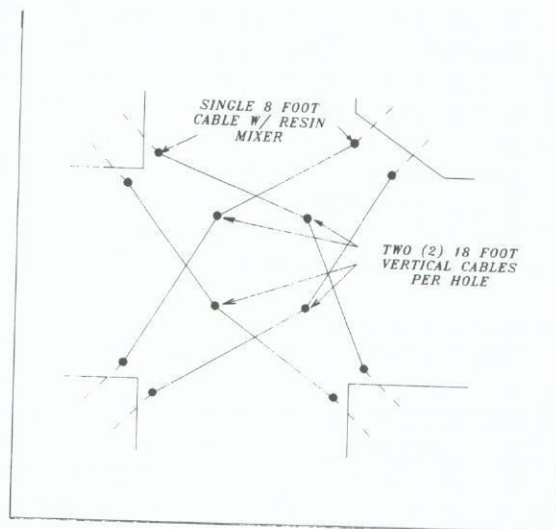


Figure 12
Modified Cable Truss Layout
for Intersections

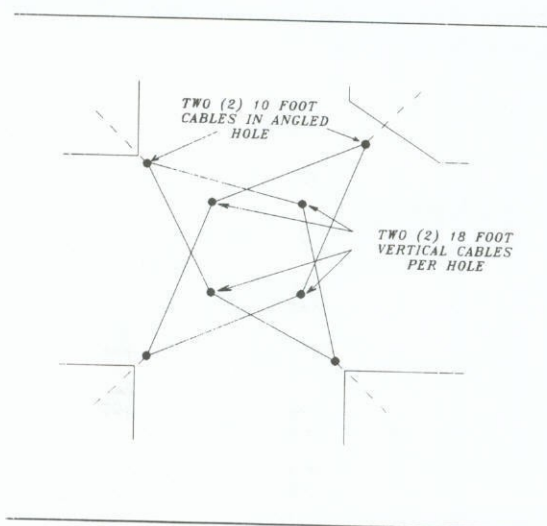


Figure 13
Cable Truss Pattern with
Less Total Drilling

The pattern was altered for use in the tailgate entry, so the system remains effective as the longwall retreats through the intersection. See Figure 14.

All three of the cable truss systems identified in Figures 12, 13, and 14 are being used to date and have worked successfully in linears, gateroad intersections, and severe roof conditions associated with the rider seam proximity.

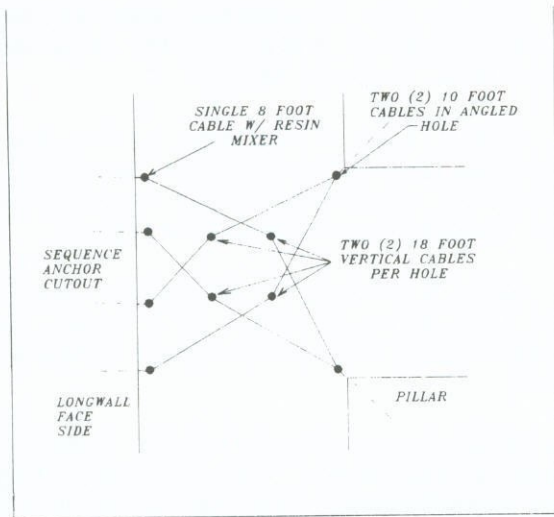


Figure 14
Modified Cable Truss Pattern
for Tailgate Use

Severe Conditions Associated with Underclay Thickness

Several zones in this Block of panels have experienced underclays greater than three and a half feet. This underclay thickness represents the point where the wooden gateroad support is likely to fail.

In zones of greater than 3.5 feet of underclay, the floor heave increases to the point that cribs are shifted out of plumb, timbers are broken and Propsetters are shifted so far out of vertical that they are ineffective. Poor wood quality has also been a problem.

Along with the problem that shortened the "F" panel, a zone in the headgate of "G" panel made the two rows of cribs installed in two of the four entries look like the roof was ready to fall for a length of eight hundred feet. This area was re-supported with Strata Products - Link -N- Lock cribs. This method was selected due to reduced space for construction and their higher support capacity.

Since the area was re-supported, the new cribs have not taken any weight. This led us to believe that the destruction of the initial crib installation was related to floor heave.

In an attempt to identify the zones of thicker floor clays, we have instituted a program of drilling the floor during development. The floor is drilled to the depth of an underlying limestone band which helps to determine the thickness of the underclay. In thicker zones, tighter spacing, larger footers, or increased capacity wood supports will be used for now.

Switch to Fully Grouted Primary Support

With experiences in Block 1 and the start of Block 2, horizontal roof movement was shearing the primary support system. The installation of an eight-foot two-piece tension resin

system in all developments since 1992 was the norm. The excess movements in the gateroads on retreat sheared the pins just below the coupler. This failure led to the testing of fully grouted primary support. Since the eight-foot bolt had been used for so long, a gradual reduction in length was instituted on one gateroad development unit in Block 2. The bolts went from seven and a half to seven and finally to six feet in length.

The effect in the gateroads has been very favorable. There are no longer primary roof supports hanging out of the roof when horizontal movement concentrates on the tailgate. The resin bolts have also provided better immediate roof skin control than was experienced with the tensioned bolts.

Testing of Cable Bolts and Mats on the Tailgate

With cost effectiveness and quality of wood products decreasing, other methods of gateroad support are being tested. In an area where hardwood products are readily available, methods tried in the western United States are not readily accepted. Doubters require the proof of the product to be stacked up against years of experience. Now that quality and price have to be watched closely, the use of cable bolts and mats has shown some economic viability.

Two areas were tested in the tailgate entry of "G" panel. The first zone was in an area of thick underclay and the rider seam was within eight feet of the main seam. We tested a pattern with two hundred feet of mats on six foot centers with three, 0.6 inch, fifteen foot long cables per mat. This entire area used seven feet of resin in the anchor. The last two hundred feet of this test area had a pattern of mats on eight foot centers with four cables per mat. It also used seven foot resin anchors.

The entry, in the first area, showed closure due to floor heave. Subsequently, cribs were installed in the intersections. Cribs and Propsetters were installed in the entry between intersections. Due to the length of resin anchor used in the hole, the amount of elongation available in the cable was reduced to between 3.5 and 4 inches. The horizontal movement experienced in the area exceeded the amount of elongation. Cable failures were experienced. Twenty percent of the cables installed in the first area failed prior to longwall retreat in the area.

As the longwall retreat approached the first test area, 44% of the cables had failed. Both the wooden cribs and propsetters, along with the cable bolts and mats, looked severely tested. However, for as badly as the first area reacted, the second test area showed dramatic improvement.

The second test area was installed fifteen hundred feet out by the first test area. This test area was completed with mats on eight foot centers and four fifteen-foot cable bolts in each mat. The first two hundred feet used a seven-foot resin anchor while the last two hundred feet used three and a half foot resin anchors.

The second test site had less than 1% cable failure prior to the longwall entering the test zone. As the longwall progressed through the test area, a failure of 15% of the cables was experienced. The failure of the cables was not typical, in that only

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2% was due to cable breakage. The other 13% was experienced as the wedge system experienced enough loading to cause the wedge and coupler to slip off the cable prior to cable breakage.

This slippage problem was evaluated by the product manufacturer and has been resolved by improving the strength of materials used and the method which the wedges are installed.

Noticeable improvements were observed in both test areas in the control of roof spall material and reduction in the material from the cutter side of the entry.

The installation of a tailgate entry with cable bolts and mats has just begun in the first tailgate of Block 3. The system was enhanced to utilize mats on four foot centers with four fifteen-foot cables in each mat. The resin anchor being used is three and a half foot long. This system is being installed along approximately 5000 feet of the first tailgate in Block 3.

CONCLUSION

The selection of a single method of longwall gateroad support may not meet all the conditions that lie ahead in the retreat of a longwall panel. Some methods are too costly, while others seem too risky. The best method for selection is to evaluate the known problems. Select the problems that have the largest adverse effect, identify the areas where you anticipate those conditions, and design a system that should control those conditions. One thing is certain, the forces of nature will provide both expected and unexpected results. Evaluate your successes as well as your failures.

Cable products have provided the flexibility to enhance the gateroad support systems being used at the Baker Mine. Our goal is to enhance the gateroad support systems through the use of products that provide good economics and support characteristics. As wood products become more costly and of poorer quality, our efforts to reduce this products consumption continue.

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Application of the Coal Mine Roof Rating, Derived from Drill Core, in the Roof Support Design of a Coal Belt Conveyor Tunnel

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ABSTRACT

The Coal Mine Roof Rating (CMRR) is a practical rock mass classification system for use in design of mine openings. Data can be obtained for this purpose from exploration drill cores if simple geotechnical logging and rock mechanics testing is conducted. The CMRR is an excellent method to quantify rock characteristics for engineering purposes. It bridges the gap between geologic description and engineering design. The CMRR was utilized in the design of a 10,000 ft. long coal belt conveyor tunnel connecting an Eastern Associated Coal Corp. mine and preparation plant in Boone County, West Virginia. Data gathering and analysis techniques are discussed involving the calculation of the CMRR and its application in the design of the tunnel, which was driven through a thin coal bed zone. The preliminary findings have correlated well with actual roof and mining conditions encountered during driving the tunnel and it is concluded the CMRR is an effective method to utilize geologic data from exploration core holes for engineering analysis and design.

INTRODUCTION

The Matewan Tunnel Project involved the construction of a coal conveyor belt tunnel to connect the Harris No. 1 Mine and the Rocklick Preparation Plant. The life of operation of this tunnel is projected to be 15 years. It was critical that roof conditions were accurately characterized so that proper roof support could be installed for long term stability. The tunnel design is a three entry system mined through a thin coal zone (seam thickness < 3.6 ft.) with a mining height of 6.5 ft. required for the center belt entry. Eight exploration core holes were drilled near the tunnel projections for this evaluation. A location map is shown in Figure 1. Data and testing from this

drilling was used to calculate the Coal Mine Roof Rating (CMRR) as proposed by Molinda and Mark (1) with adjustments made in the calculation of the unit rating based on the analysis of the tested rock strength data. Computer generated contour mapping and graphical geological cross sections through the mining zone were utilized to evaluate this data. The CMRR was applied in roof control design and in defining the optimum mining zone for the belt entry.

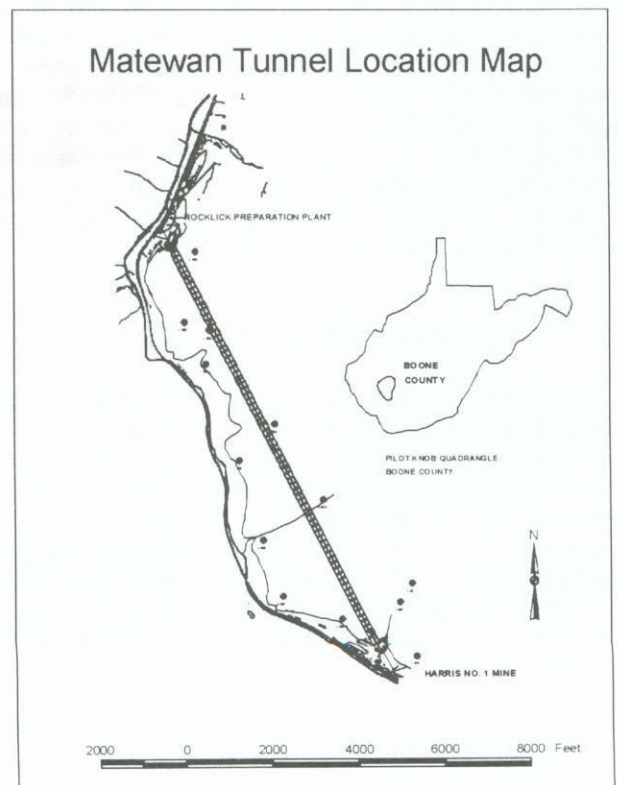


Figure 1. Location Map.

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SITE GEOLOGY

The Matewan Tunnel is driven through the Matewan coal bed horizon. This zone is approximately 35 feet above the Eagle Coal of the Kanawha Formation. In the area of the tunnel, the Matewan Seam ranges from 2.5 ft. to 3.6 ft. with minimal in seam rock partings. The immediate roof is approximately 20ft. of massive shale overlain by approximately 50 ft. of massive sandstone. The generalized geologic section is shown in Figure 2. This seam was selected because of its consistent thickness, structure and uniform roof over the projected routing of the tunnel. The Eagle Seam contained several splits with variable parting thickness, which would have made maintaining a consistent mining zone very difficult. Also there is no over or under mining in this area of the Matewan Seam.

The shale unit making up the roof of the Matewan Seam is locally referred to as the Cannelton Shale. The depositional environment of the Cannelton Shale is a lower delta plain bay-fill sequence. There have been no sandstone channels identified within this stratigraphic unit over the limits of the study area. In any evaluation of coal mine roof conditions, an accurate geologic depositional model must be established to identify transitional shale zones adjacent to sandstone channels. These areas are prone to more roof control problems and need to be identified. A geologic model should be incorporated into any CMRR evaluation especially if isopleth mapping of CMRR values is considered. The computer contouring of CMRR point data by itself may not accurately portray roof conditions. The depositional model for the Matewan Tunnel is a massive bay-fill shale and computer contouring of the CMRR values should present an accurate assessment of roof conditions.

EXPLORATION DRILLING

All exploration drill holes are continuous (NQ-2) core with a 2 inch diameter. For the eight holes drilled to gain data for CMRR analysis, all core 20 ft above and 10 ft. below the Matewan Seam was boxed in cardboard core boxes. The coal core was extruded into a wooden core box according to standard practice. Additional rock core was retained and tested above the 20 ft. zone on two of the core holes to gain data for comparative purposes. Core logging was conducted by a geologist with discontinuity logging done in accordance with CMRR procedures to include discontinuity type, spacing and condition. Discontinuity sets include bedding planes, joints, slickensides, coal spars, siderite nodules, etc.

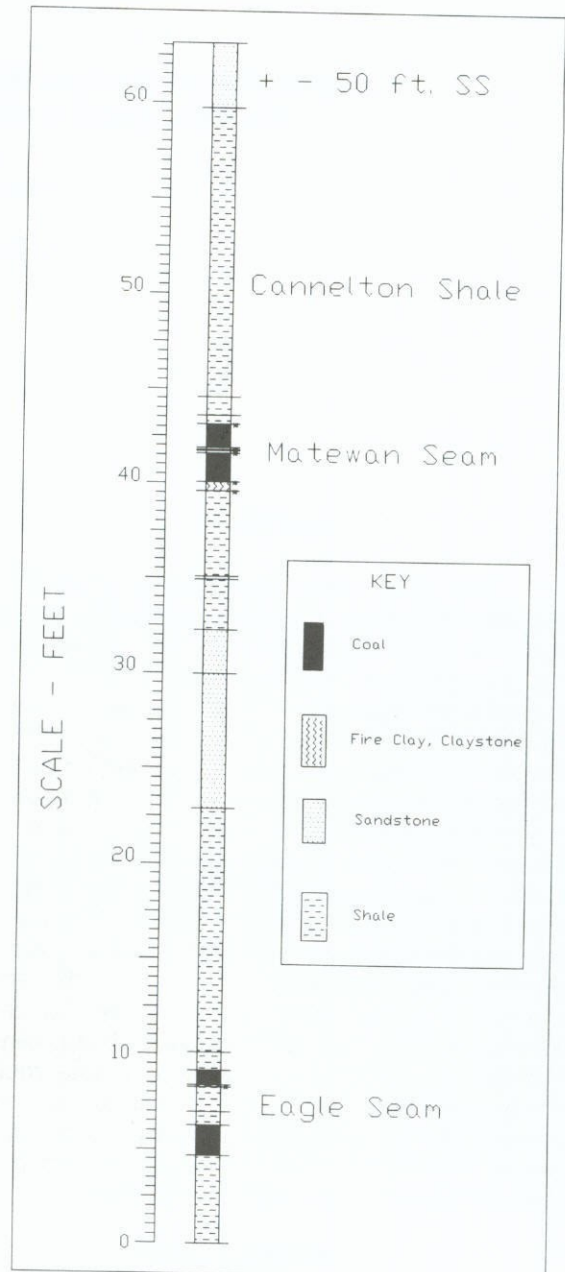


Figure 2. Generalized Geologic Section.

ROCK MECHANICS TESTING

The following tests were conducted on the retained rock core samples. These tests were conducted according to International Society of Rock Mechanics (ISRM) procedures at the Commercial Testing and Engineering Company (CTE) rock mechanics testing facility in Beckley, WV. A total of 574 samples were tested.

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Rock Mechanics Tests

1. Uniaxial Compressive Strength (UCS)
2. Indirect Tensile Strength (ITS)
Brazilian Test
3. Axial Point Load Strength Index (Is50a)
Core is tested perpendicular to bedding.
4. Diametral Point Load Strength Index (Is50d)
Core is tested parallel to bedding

The CMRR requires two rock strength values AXIAL STRENGTH and DIAMETRAL STRENGTH. The definition of these strength values and their relation to the above tests are as follows:

AXIAL STRENGTH - Intact rock strength that equals the Uniaxial Compressive Strength (UCS) and can be calculated with the Axial Point Load (Is50a) index multiplied by an index to strength conversion factor.

DIAMETRAL STRENGTH - This is what can be referred to as the lateral, cohesive or tensile strength of the rock. The strength value should be related to the indirect tensile strength (ITS) and can be calculated with the Diametral Point Load (Is50d) index multiplied by an index to strength conversion factor.

The CMRR proposes the use of the point load test (PLT) for rock strength determination. The PLT has been widely accepted in geotechnical practice as a practical means to determine rock strength (2). An index to strength conversion factor is required to calculate actual rock strength. The CMRR recommends the index to strength conversion factors determined by Vallejo et al (3). These were calculated for Appalachian coal measure rocks and classified by shale and sandstone lithology for samples obtained from surface mining sites. These indexes were calculated using the uniaxial compressive strength (UCS) and the axial point load index (Is50a). The index to strength conversion for shales is 12.5 and for sandstones is 17.4.

Simple statistical analysis of the 574 rock mechanics tests conducted for this project are summarized in Table 1. The sample rock types were classified according to Firm codes (4). Firm established a numerical system for classifying coal measure rock types using color photographs of core samples and is widely used in the Appalachian Coal Basin. Molinda and Mark (5) have done extensive work in rating rock strength utilizing the Firm classification

system. These Firm codes were further grouped into three categories based on grain size. These major rock categories are as follows:

1. **SHALE**- Shale and claystone, Firm 100 Series
2. **SILTSTONE** - Silty and sandy shales to include shale with sandstone streaks, Firm 300 Series
3. **SANDSTONE** - All grain sizes, Firm 500 and 700 Series.

Mean

	UCS	Is50a	ITS	Is50d
Rock Type	(psi)	(psi)	(psi)	(psi)
Shale	9452	464	1136	87
Siltstone	13448	718	1621	248
Sandstone	16427	804	1370	473

Standard Deviation

	UCS	Is50a	ITS	Is50d
Rock Type	(psi)	(psi)	(psi)	(psi)
Shale	3213	167	550	105
Siltstone	4343	214	412	169
Sandstone	3211	226	314	240

Number of Samples

Rock Type	UCS	Is50a	ITS	Is50d
Shale	34	40	23	52
Siltstone	76	75	71	80
Sandstone	28	32	31	32

Table 1. Statistical summary of rock mechanics testing.

One purpose of conducting UCS and ITS along with the Is50a and Is50d was to determine an index to strength conversion factor. This allows for comparison to Vallejo's findings and more importantly to determine an index to strength conversion factor for the diametral point load index (Is50d) and the ITS. This will allow for a "back calculation" of existing database ITS values to a Is50d index for future mine roof evaluations utilizing the CMRR. The diametral point load test had just recently become a standard test in our suite of rock mechanics testing. The existing database contains hundreds of ITS tests that need to be utilized in CMRR analysis. The diametral strength rating of the CMRR

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should be related to the tensile strength of the rock with an appropriate index to strength conversion factor for the Is_{50d} . Based on the statistical analysis, the following conversion factors and anisotropy indexes were obtained as shown in Table 2.

Rock Type	Is_{50a} to UCS	ITS to UCS	Is_{50d} to ITS	Anisotropy Index $Is_{50a} : Is_{50d}$
Shale	20.37	8.33	13.06	5.33
Siltstone	18.70	8.33	6.54	2.90
Sandstone	20.43	12.50	2.90	1.70

Table 2. Rock strength conversion factors and anisotropy index.

The Is_{50a} to UCS conversion factor is approximately 20 for all rock types and is the range proposed by ISRM (5). Although it is slightly more than the value of 17.4 proposed by Vallejo for sandstones, it does not show a lesser value for shales. The UCS averages approximately 10 times greater than the ITS and compares with a value of 10.5 determined by Hassani et al (7) using an extensive data set for coal measure rocks from the UK. But the data set in Table 2 does indicate a difference between shale and sandstone rock types for the ITS to UCS conversion. However the most interesting relationship is the comparison of the Is_{50} anisotropy index and the Is_{50d} to ITS conversion factor. These relationships are graphically illustrated in Figure 3.

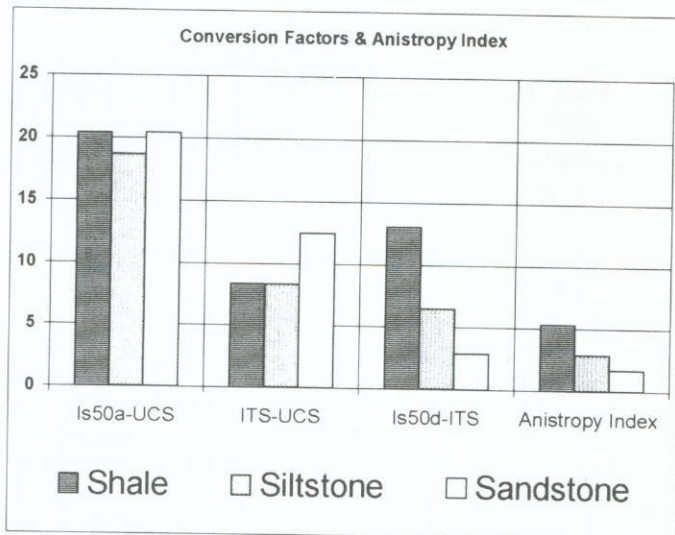


Figure 3. Rock strength conversion factors and anisotropy index.

The anisotropy index is the ratio of the Is_{50a} to the Is_{50d} . These values obtained here correlate well with the work of Molinda and Mark (5). In sedimentary rock,

high anisotropy indicates weak bedding plane cohesion, as is the case in argillaceous or fissile shales and claystones. For a given rock category, proposed previously as SHALE, SILTSTONE or SANDSTONE. The anisotropy index factor is directly proportional to the Is_{50d} to ITS conversion value by a factor of approximately two. This indicates that the ITS by itself for highly anisotropic rocks may not be a true indication of bedding plane cohesion. In other words the greater the anisotropy, the less likely will the load frame Indirect Tensile Strength (ITS) or Brazilian Test indicate true cohesive strength which is critical in mine roof characterization. It is important to note here that for a number of the ITS tests on SHALE category rocks, the sample did not break in the characteristic near center vertical fracture of the disc but rather separated along the bedding planes. These tests were considered invalid and the data was not used. This suggests that the point load test (PLT) would be more appropriate to determine bedding plane cohesive strength.

Based on this analysis, the following equations are proposed to convert the Indirect Tensile Strength (ITS) to the Diametral Point Load Strength Index (Is_{50d}) for application in the CMRR analysis:

1. Shales (Ferm 100 Series): $Is_{50d} = ITS / 13.0$
2. Siltstones (Ferm 300 Series): $Is_{50d} = ITS / 6.5$
3. Sandstones (Ferm 500 Series): $Is_{50d} = ITS / 3.0$

This will allow utilization of existing rock mechanics ITS data in cases where diametral point load testing was not conducted.

Adjusted Index to Strength Conversion Factors

The CMRR was proposed with the Point Load Test (PLT) as the recommended method for determining rock strength. This was so expensive load frame testing would not be required and coal companies would be more receptive to using this system in mine roof analysis and design. It is the author's opinion that the PLT should remain as the recommended method to determine rock strength if appropriate indexes to strength conversion factors are used.

The problem associated with the index to strength conversion factors determined by Vallejo is that the testing related only the axial PLT to the UCS. No relationships were established for tensile strength and the diametral PLT yet the Unit Rating (UR) calculation for the Diametral Strength Rating uses the axial index to strength conversion factors.

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For this project, point load index (Is50) to strength conversions factors were derived from analysis of the rock strength data discussed in the previous section. These indexes to strength conversion factors are applicable to this project and may have correlation with similar rock types in the southern Appalachian coal fields. More work is warranted to determine if these factors are universal for coal measure rock types.

The point load indexes (Is50) to strength conversion factors used for the Matewan Tunnel Project are as follows:

- 1. Axial Strength (AS) = Is50a X 20 or UCS value. This is for all rock types (SHALE, SILTSTONE, SANDSTONE).**
- 2. Diametral Strength (DS) = Is50d X 8.0 (SHALE / SILTSTONE)
Diametral Strength (DS) = Is50d X 12.5 (SANDSTONE).**

The values 8.0 for SHALE / SILTSTONE and 12.5 for SANDSTONE were derived from data set relating the ITS to UCS. This converts the Is50d strength index to an "equivalent compressive strength value" which is analogous with the current calculation of the diametral using the axial strength indexes of Vallejo. However, this method is a more applicable indicator of diametral strength because it incorporates tensile rock strength in the calculation. An "equivalent compressive strength value" is necessary to use the existing unit rating tables for diametral strength rating based on the strength ranges of the table. The values of this table do not correlate directly to tensile strength.

CMRR Calculation Procedures

The CMRR calculation procedures followed for this project are taken directly from publications by Molinda and Mark (1,5,8). These procedures are outlined here as they were applied to the Matewan Tunnel Project. Rating and adjustment tables as well as calculation sheets found in these publications will not be shown in this paper. One should refer to these publications for this and additional information. It should be noted here that CMRR was first established by Molinda and Mark from gathering information from underground exposures and later modified for use with exploration drill core and this paper deals with the later. However the CMRR was calculated using the underground procedures in several locations of the Matewan Tunnel for comparison with the predicted values from the drill core data. This comparison will be presented later in the paper.

The CMRR is calculated in two steps. First a rating number is calculated for each distinct rock unit in the roof. Then these unit ratings are combined with the

application of adjustment factors to calculate a CMRR value for a given bolted roof interval. The roof is classified with a rating number from 0 - 100. These rating numbers are categorized as follows:

0 - 45 Weak Roof

45 - 65 Moderate Roof

65 -100 Strong Roof

This 0 - 100 rating system is also used by the Geomechanics Classification or Rock Mass Rating (RMR) System as proposed by Bieniawski (9). Newman and Bieniawski (10) had applied this system in the evaluation of coal mine roofs. Urnal (11) utilized the RMR to establish guidelines for the selection of roof support in coal mines. The CMRR and RMR, both empirical design methods, are similar, as rock mass classifications systems in many ways and their rating values are comparable. The main difference between them is that the CMRR has a provision for incorporating bedding plane cohesive strength through the diametral strength rating. Some of this work with the RMR has been incorporated into the design analysis using the CMRR values for this project.

Unit Rating Calculations

For each core hole, 20 ft. of roof was divided into units based on lithology and strength characteristics. The coal seam itself and 2 ft. of floor rock were defined as separate units. The inclusion of the seam and floor as units departs from the intended use of the CMRR. But this was seen, as a possible method to delineate an optimum 6.5 ft. mining zone required for the belt entry. In this analysis the unit rating is applied as a rock mass classification system to determine the zone that would be easiest to cut with a continuous miner. The actual Point Load Strength Indexes, Is50a and Is50d, along with the UCS and the converted ITS values were averaged for each rock unit.

An explanation of the tables used in the Unit Rating Calculations is outlined below. These tables can be found in a paper by Mark and Molinda (8). The unit rating calculations were carried out utilizing computer spreadsheet software.

1. **Discontinuity Rating** - This table defines a rating value from 17 to 70 and is based on the discontinuity spacing, condition and diametral strength. This table was placed in the spreadsheet as a "lookup table".

2. **Diametral Strength Rating** - This table rates the Diametral Strength (DS) which equals the Is50d

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multiplied by 12.5 for sandstone's and 8.0 for shales as determined by this project's data set. The rating values range from 25 to 70. Instead of using a lookup table where a single diametral strength rating was determined from a range of diametral strength values, a linear regression analysis was done on the values in this table and the resulting equation was used in the spreadsheet calculations. This equation is as follows:

$$\text{Diametral Strength Rating} = \text{DS} \times .0087 + 21$$

(With a maximum value of 70 used.)

3. **Multiple Discontinuity Set Adjustment** - This table provides for an adjustment in the range from -2 to -5 for multiple discontinuities or diametrical strength ratings that fall below a certain value. This was placed as a "lookup table" in the computer spreadsheet

4. **Axial Strength Rating** - This table rates the Axial Strength (AS) which equals the I_s50a multiplies by 20 for all rock types. These rating values range from 5 to 30. Similarly as with the diametral strength rating a linear regression analysis was also done on these values and the following equation was used in the spreadsheet calculations.

$$\text{Axial Strength Rating} = \text{AS} \times .0016 + 8$$

(With a maximum value of 30 used.)

The above rating values are combined in a step by step process to calculate a Unit Rating value. An example of the computer spreadsheet with input data and calculations for the Unit Ratings is shown in Table 4.

A computer generated geologic cross section through the core holes was constructed to include 20 ft. of roof, coal seam and 5 ft. of floor. This cross section was imported into a CAD (computer aided drafting) software package for further analysis. In the CAD software, the unit rating values were posted adjacent to each corresponding lithologic unit on the cross section. By visual inspection of each core hole and accounting for the floor and immediate roof unit ratings, a 6.5 ft. mining height was delineated in the CAD drawing which minimized the amount of competent rock mined as based on higher unit rating values. From the top of this 6.5 ft. mining zone, the roof units for the CMRR calculations were identified. An example of this procedure is illustrated in Figure 4.

Calculation of the Coal Mine Roof Rating (CMRR)

The unit rating analysis identified the roof bolting zone to be in most cases one unit consisting of a silty to sandy massive shale. The calculation of the CMRR was also accomplished by using a computer spreadsheet. This spreadsheet incorporates all tables and calculations as found in Molinda and Mark (1). In this case where the bolted interval is a single unit, the CMRR calculation is quite simple but when multiple lithologic units comprise the bolted interval, calculation of the CMRR with a spreadsheet is most efficient with the unit rating calculation table "linked" to the CMRR calculation table. The CMRR spreadsheet format is illustrated in Table 5. The required input values are in the shaded boxes. All others are calculated or extracted from the Unit Rating (UR) table.

UNIT RATING CALCULATION SHEET													
Hole	V-992C												
				Axial		Diametral	DISCONT.	DISCONT.	DISCONT.	AXIAL	DIAMETRAL	MULTI.	
UNIT	STRATA	THICK	I_s50a	STRENGTH	I_s50d	STRENGTH	SPACING	RATING	ADJUST.	STRENGTH	STRENGTH	DISC.	UNIT
NAME		FEET	PSI	(AS) PSI	PSI	(DS) PSI	1*	(DR)	2**	(ASR)	(DSR)	ADJUST.	(UR)
ROOF3	SH,SDY	15.18	784	15680	335	2680	8	52	0	30	45	FALSE	75
ROOF2	SH,DRK	0.96	367	7340	67	536	0	24	0	19	24	-5	38
ROOF1	SH,BLK	0.47	320	6400	82	656	2.5	31	0	18	27	-4	41
SEAM	CO	3.07	68	1360	0	0	2.5	26	0	10	21	-5	26
FLOOR1	CS	0.50	524	10480	219	1752	2.5	40	0	24	37	-2	59
FLOOR2	SL,SLY	4.62	723	14460	273	2184	8	48	0	30	40	-2	68
1* Values for discontinuity spacing: < 2.5" = 0; 2.5" - 8" = 2.5; 8" - 24" = 8; 2' - 6' = 24; > 6' = 60.													
2** Roughness Adjustment for non-slicked sided discontinuities: If discontinuity is wavy, 3; if it is jagged, 6.													

Table 4. Unit Rating calculation spreadsheet format.

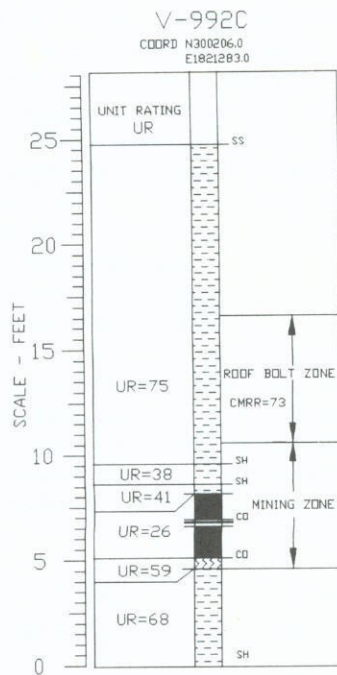


Figure 4. Delineation of mining zone with Unit Ratings.

Roof Rating (CMRR) Calculation Sheet				
		North	East	
Hole No.	V-992C	300206	1821283	Seam:
Mine:	TUNNEL			MATEWAN
1) Calculate the weighted average of the Unit Ratings (RRw)				
Rock Type	UR	Thick. (ft.)	UR* Thick	
6	0	0.00	0	
5	0	0.00	0	
4	0	0.00	0	
3	0	0.00	0	
2	0	0.00	0	
SH,SDY	1	75	6.00	450
Bolted Interval (BI) =		6.00	450	
			RRw =	75
2) Calculate Strong Bed Difference (SBD)				
Largest (UR)	0	Strong Bed (SB)		
(SBD)	-75			
3) Calculate the Strong Bed Adjustment				
				0
4) Calculate the Unit Contact Adjustment				
				0
5) Calculate the Groundwater Adjustment				
				-2
6) Calculate the Surcharge Adjustment				
				0
			CMRR =	73

Table 5. CMRR Calculation Spreadsheet

Comparison of Conversion Factors

A comparison of the CMRR values calculated with the adjusted index to strength conversion factors derived from the rock mechanics testing done in this project and the values proposed by Vallejo and recommended in the procedures of Molinda and Mark are shown in Table 6.

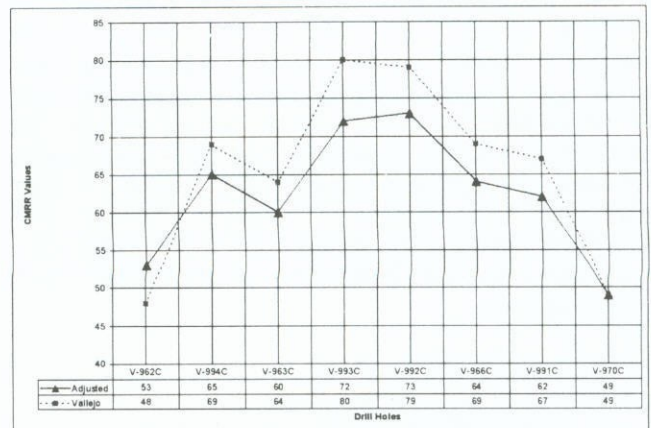


Table 6. Comparison of CMRR values calculated using different index to strength conversion factors.

The only conclusions that can be drawn with this limited data is that the index to strength conversion factors proposed by Vallejo and their current application in the calculation of the CMRR result in an overall average value that is approximately 5% greater than when using the conversion factors derived from this project's test data. This difference would not be considered significant when calculating rock mass classification values. The agreement between these two approaches indicates that either may be utilized with the end result being similar even though the method of calculating the Is50d factor is slightly different. This correlation does support the utilization of the Indirect Tensile Strength (ITS) to Diametral Point Load Index (Is50d) conversion formulas proposed earlier as a means to calculate the CMRR with existing database ITS values.

Analysis and Design with the CMRR

The application of the CMRR to roof control design is probably one of the most challenging areas in mine engineering. Any design work with the CMRR can only be considered as an empirical approach. In order to formulate an effective design for coal mine roof control, several approaches should be considered combined with sound engineering judgement. These approaches should compliment each other and input from each should contribute to the final design considerations. Bieniawski (9) categorized four types

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of methods that can be employed in the evaluation of mine roof stability. These methods are geological, numerical, observational and empirical. In the design analysis of the Matewan Tunnel, geological, numerical and empirical methods were used.

Geological methods deal with roof lithology mapping through depositional modeling with delineation of transition zones, measuring of joint orientations, fracture trace/lineament analysis and orientation of horizontal stress. For this project, the geological analysis included determining the depositional roof model to be a massive bay-fill silty to sandy shale with no sandstone channel cutouts and therefore no transition zones and measuring rock joint orientations and linear analysis in the vicinity of the tunnel. A map of the rock joint orientations, lineament and iso-line of the CMRR values is shown in Figure 5. The belt entry crosses all major joint orientations at angles between 7 and 80 degrees.

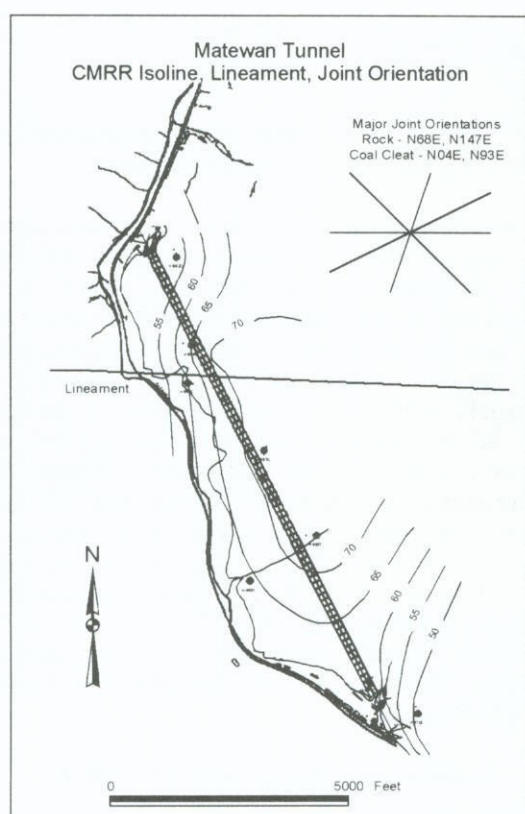


Figure 5. CMRR Iso-line, lineament and joint data.

The numerical methods included a finite element analysis conducted by Jenmar Corporation and using standard bolt load determination formulas. The empirical methods involved applying the CMRR value in analysis derived from the Geomechanics Classification (RMR) analysis using CMRR values and

comparison of successful designs in nearby mines with similar roof characteristics. The values of the CMRR that lie within the projections of the tunnel ranged from 53 to 73 with an average of 64. This value is in the upper range of what is classified as moderate roof. As seen from the map in Figure 5, the CMRR values are greatest in the center portion of the projections and decrease towards each portal. The CMRR was calculated at several locations in the tunnel adjacent to drill holes where the CMRR was calculated from core. There were no roof fall exposures and the information was gathered as best as possible from the immediate roof and evaluated according to the procedures outlined in Molinda and Mark (1). This comparison is shown in the Table 6. Both methods produced similar results with the calculation involving core to be slightly greater. The observed roof conditions overall would be considered very good in comparison to other mines in the same general area.

Hole Number	CMRR (CORE)	CMRR (Underground)
V-992C	73	71
V-993C	72	67
V-994C	65	63

Table 6. Comparison of CMRR values.

An average CMRR value of 64 for the Matewan Tunnel was used in the following design analysis.

The CMRR value is designed to rate the bolted interval and its ability to form a stable mine roof. In the case of a single thick unit, as it is in this evaluation, a 3 ft. bolted interval would have the same rating as a 6 ft. bolted interval. In order to determine and appropriate bolt length, empirical design procedures proposed by Urnal (11) using the concept of rock load height based the Rock Mass Rating (RMR) of the roof and the length of the span were used. For this analysis, the CMRR value was substituted for the RMR value in the calculation. Based on rock load height, a minimum bolt length of 1/2 the rock load height is proposed as well as two alternative, empirically derived bolt lengths based on roof span. For the CMRR value of 64 the rock load height was calculated to be 6.12 ft. for a proposed entry width of 17 ft. The proposed bolt lengths would be 3 ft., 5.6 ft. and 6.6 ft. with an average of 5.1 ft. In nearby mines in the No. 2 Gas and Powellton seams 4 to 5 ft. fully grouted resin bolts have been used successfully in similar roof lithology and CMRR values calculated from core testing database UCS and ITS values. The finite element analysis conducted by Jenmar concluded that horizontal and vertical tensile stress would develop 1.5 ft. to 2.8 ft. into the roof and that this stress concentration may cause bedding plane failure and separation between roof layers.

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The decision was made to evaluate a standard bolt pattern of 4 ft. row spacing and 4 bolts per row with a 6 ft. fully resin grouted grade 40 No. 6 headed rebar. This design recognizes the average bolt length recommended with the RMR analysis and previous experience with this type of roof but increases the bolt length to 6 ft. to gain additional anchorage. Based on this pattern, the reinforcement factor (RF) of this design was calculated as 1.64 assuming a maximum of 4 ft. of possible rock load caused by the vertical and tensile stresses indicated in the finite element analysis. This allows for a full 2 ft. of anchorage approximately 1 ft. above this stress zone. In conjunction with the 6 ft. bolts, a 14 ft. T5 channel strap is installed in each row for better load distribution and skin control. The belt entry cross section with this design is shown in Figure 6. Spray application of a commercially available weathering inhibitor was done prior to rock dusting on the roof and shale portion of ribs in the belt entry.

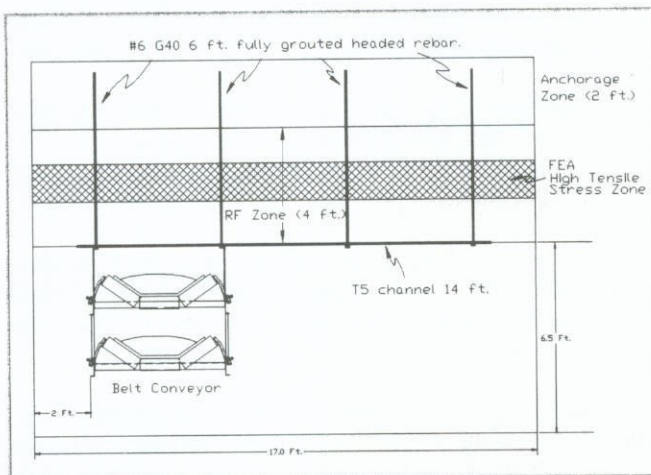


Figure 6. Belt Entry Roof Control Design.

The CMRR value was also evaluated through two statistical analysis involving the prediction of intersection roof falls and supported stand-up time.

Predicted roof fall probability as proposed by Mark et al (12) deals with intersection roof falls and is based on a multiple regression analysis of CMRR values and intersection spans. It predicts the number of intersection roof falls per 10,000 ft of driveage. The analysis of this project's average CMRR value of 64 and entry width of 17 ft. yields a zero probability of intersection roof falls.

Another method of quantifying the stability of mine roof developed by Newman and Bieniawski (10) using the Rock Mass Rating (RMR) was also evaluated. This method used statistical techniques to determine the supported stand-up time for a mine opening based on

62 case histories from two northern Appalachian coal mines. This method uses the RMR value and an adjustment for horizontal stress and reinforcement factor (RF). For this project the RMR value is equated to the CMRR, horizontal stress is assumed only that due to vertical loading and the $RF = 1.64$. This results in a supported rating value of 81 which corresponds to a stand-up time of 14.4 years which compares favorably to the projected tunnel life of 15 years.

Conclusion

The Coal Mine Roof Rating (CMRR) is an effective method for categorizing mine roof for engineering purposes. The CMRR value derived from exploration drill core provides a quantitative parameter useful in ground control design. It enhances the value of exploration core data and can be obtained cost effectively through point load testing. In determining rock strength in CMRR calculations the following should be considered.

1. Use appropriate index to strength conversion factors. If load frame test data are available, use UCS values as Axial Strength and convert Indirect Tensile Strength (ITS) values to Is_{50d} by the following equations:

1. Shales (Ferm 100 Series): $Is_{50d} = ITS / 13.0$

2. Siltstones (Ferm 300 Series): $Is_{50d} = ITS / 6.5$

3. Sandstones (Ferm 500 Series): $Is_{50d} = ITS / 3.0$

2. Then for the point load test strength indexes use either the conversion factors of Vallejo as recommended in the current CMRR procedures of Molinda and Mark (8) or those proposed here. Both methods yield similar results and the selection of either method should be based on engineering judgement involving the location, geology and type of data available for a particular project.

1. Axial Strength (AS) = $Is_{50a} \times 20$ or UCS value. This is for all rock types (SHALE, SILTSTONE, SANDSTONE).

2. Diametral Strength (DS) = $Is_{50d} \times 8.0$ (SHALE / SILTSTONE)
Diametral Strength (DS) = $Is_{50d} \times 12.5$ (SANDSTONE).

The refinement of this point load index to strength conversion factors is possible as more data are evaluated and would be a good area for additional research.

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The use of computer spreadsheets to calculate the CMRR is recommended especially when evaluating large amounts of data. A Lotus spreadsheet is available from NIOSH (U.S. Bureau of Mines) and is described in Mark et al (12). A custom built spreadsheet to handle special data import and export formats is very useful. Other computer applications to include CAD, data modeling and contouring assist in the design analysis using the Unit Rating (UR) and CMRR values.

A geologic model of the roof should be developed and used in conjunction with CMRR analysis. Likewise any numerical studies such as finite element analysis should be considered. The application of previous empirical work involving Rock Mass Rating (RMR) as related to coal mine roof as well as more recent CMRR studies should be used to evaluate proposed roof control design. A combination of methods should determine the final design.

The empirical approach is the most powerful aspect of utilizing the CMRR for evaluating roof control design performance. The CMRR allows comparison of roof characteristics and the performance of roof control design by building case histories from existing mines and then using this information to establish appropriate design in planned mines with similar CMRR values. This type of analysis would have the most value to coal companies and can only be effective if the roof is properly characterized through the CMRR and a database is established to include underground calculations and those derived from drill cores.

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The Effects of Reduced Annulus in Roof Bolting Performance

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ABSTRACT

The annulus thickness, for bolts anchored with resin, effects the systems final performance. Recent developments in the manufacturing process of these support systems, coupled with improved resin characteristics, have resulted in design changes to minimize hole drilling requirements and reduce resin usage. The amount of resin to properly anchor a 7/8-SRD (Special Rolled Dimensions) bolt in a 25.4 mm (1-in) diameter hole has been reduced to about one-third of the resin required to anchor a No. 7 rebar bolt (22 mm (7/8-in) diameter) traditionally installed in a 35 mm (1-3/8-in) diameter borehole. More importantly, laboratory and field tests, indicate that higher bond strengths and anchorage capacities can be realized with reduced annulus systems. These factors have a direct bearing on reinforcement efficiency and subsequent roof support performance.

INTRODUCTION

Millions of roof bolts are used each year to provide structural support in underground metal, nonmetal, and coal mines. An increasing percentage of these bolting systems are utilizing resin-anchor and resin assisted mechanical anchor bolts. The mechanisms by which resin grouted systems carry roof loads is still not well understood because the testing and evaluation procedures used for point anchored systems are not entirely valid for resin-grouted bolts. Significant advances in the areas of mine design and strata characterization utilizing analytical, empirical, and observational methods have been made during the last 10 years. These advances necessitate the requirement for innovative strata stabilization and control using various types of bolting systems.

Recent developments in resin grouted support systems, coupled with improved resin characteristics, have resulted in design changes to minimize hole drilling requirements and reduce resin usage. In fact, the amount of resin to properly anchor a 7/8-SRD (Special Rolled Dimensions) bolt in a 25.4 mm (1-inch) diameter hole has been reduced to about one-third of the resin required to anchor a No. 7 rebar bolt (22 mm (7/8-inch diameter)) in a 34.9 mm (1-3/8 inch) diameter borehole. As shown in figure 1, a resin savings of 40 percent can be realized for a 2.13 m (7 ft) long support system. The resin savings results in a decrease in the costs to fully encapsulate the bolts in their respective holes.

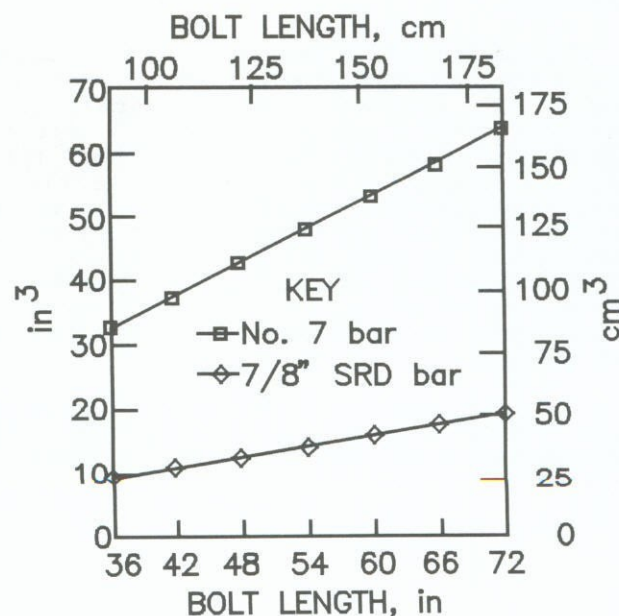


Figure 1. Resin volumes required for grouting No. 7 and SRD rebar bolts.

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While it is always prudent to save costs with a support system, it is imperative that anchorages can be successfully achieved throughout the entire loading process. This performance is called load transfer or the mechanism by which force is generated and sustained in the bolt as it is being loaded through strata deformation. In resin grouted bolts, the load transfer mechanism is dependent on the shear stresses sustained at the bolt/resin and resin/rock interfaces. The peak or ultimate stress and the system stiffness determines the response or effectiveness of the bolt to the strata behavior. The most important aspect of load transfer is the ability of the resin system to develop the full capacity or ultimate tensile strength of the bolt. The ultimate strength can only be achieved if the resin and host rock can achieve and sustain the shear stresses until the bolt fails (1-2).

Theoretical Load and Stress Transfer

Resin grouted support system consists of three materials, the bolt, resin grout, and host rock. Each component has very different material properties that effect the final performance of the system. Steel materials are ductile and have high ultimate strengths and modulus of elasticity, meaning they can sustain large loads and deflections. Resin grouts and the host mine rocks can be brittle and are not capable of sustaining high tensile loads. The coupling of these two different material types can result in shear failures. However, if the length of the resin encapsulation is long enough and the load transfer is complete, the full capacity of the steel bolt can be achieved.

The method most commonly used to assess the potential anchor strength is the pull-tests. The minimum length of final column length tested is 30-46 cm (12-18 inches), depending on the strength of the host rock. This length provides enough material to ensure adequate mixing throughout the column length but more importantly eliminates the possibility of the bolt yielding prior to the bond failure occurring. Pull tests on fully grouted bolts are not considered to be representative of the potential system performance.

The most useful relationship obtained from these types of tests is the determination of the peak shear stress and system stiffness (3). The shear strength capacity can be calculated by using the following equation:

$$\tau = \frac{\Delta F}{\pi \phi L}$$

where; ΔF = change in force over the encapsulation length, kN (lb)
 L = resin length, cm (in)
 ϕ = bolt diameter or borehole diameter, cm (in), and
 τ = shear stress MPa (psi).

The system stiffness, kN/cm (lb/in) or the ability of the system to resist load can be determined by applying this equation:

$$K = \frac{\Delta F}{\delta}$$

where; ΔF = change in force over the encapsulation length kN (lb), and
 δ = change in length, cm (in).

LABORATORY INVESTIGATION

The 7/8-SRD system has a core diameter of 20.6 mm (0.810 in) which results in an area of 20.3 mm² (0.515 in²). The ASTM 432 minimum yield and tensile strengths of grade 75 bar are 154 kN and 205 kN (34,700 and 46,200 lbs), respectively. Two series of laboratory tests were conducted to evaluate the stress transfer mechanics for the bolt installed in a one inch diameter hole. The first series of tests were performed by installing the bolts in a 25.4 mm (1 inch) "rifled" steel borehole. The rifling simulates the irregularities found in the host rock during normal drilling operations. For comparison purposes, bolts were installed with final grout column lengths of 23, 30, 38, 46 cm (9, 12, 15 and 18 inches). The average strength for nine 38 cm (15-inch) tests was 253 kN (57,000 lb). This was the length that was determined to be the most critical based on the results obtained at 23 and 30 cm (9 and 12 inches). The plot of the average loads obtained at each resin column length are shown in figure 2. The result of the average peak shear stress calculated using 38 cm (15 inches) of resin column at the borehole wall was 11 MPa (1,493 psi).

Additionally, two tests were performed with a No. 7 (22 mm (0.875-inch diameter)) in a 34.9 mm (1-3/8-inch) "rifled" steel borehole to examine the shear strength of the larger (6.4 mm (0.250-inch diameter)) resin annulus. The results of the 30.55 cm (12 in) resin column were resin/borehole interface failures at 214 and 209 kN (48,100 and 47,100 lbs) or an average of 212 kN (47,600 lbs). The peak shear stress calculated on the borehole, using the average capacity, was 7 MPa (918 psi).

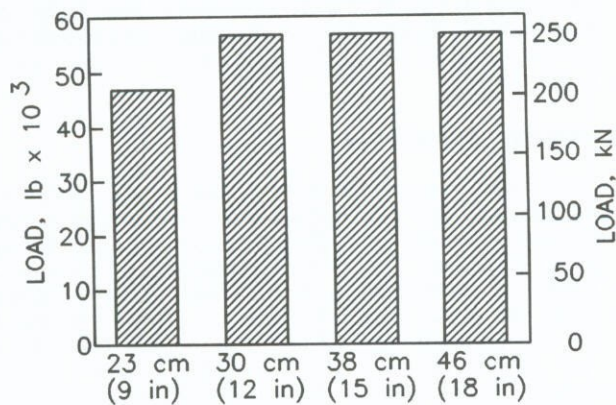


Figure 2. Average anchorage loads at the respective resin column lengths.

To ensure that complete mixing, samples of the tested bolt tubes were split and the observations of the resin component showed complete mixing and cure along the entire grout length.

To validate the results obtained from the steel tubes another comprehensive series of laboratory experiments were conducted in limestone blocks. A photograph of the testing system is shown in figure 3. Five SRD bolts were installed in a final hole diameter of 26.42 mm (1.04 inches). The bolts were installed with a final column length of 50.8 cm (20 inches). The resin was allowed to cure 10 minutes and then loaded to yield recording deformation measurements at 9 kN (2,000 lb) increments. The deformation recorded at 180 kN (40,000 lb) for the 5 bolts ranged from 5.28 to 7.77 mm (0.208 to 0.306 inches) and averaged 6.68 mm (0.263 inches). Two samples were installed with a 30 cm (12 in) column length and the bar was successfully taken to yield. The amount of deformation helps to evaluate the system stiffness, or the ability to transfer the load back into the rock mass. The average stiffness for the SRD samples installed with 46 cm (18 in) of resin ranged from 269 to 380 kN/cm (153,800 to 217,400 lb/in) and averaged 315 kN/cm (180,200 lb/inch) for the five samples.

To help evaluate the stress transfer mechanism and the effectiveness of bar deformations, three smooth bars with the same diameter were installed with the same effective resin column length. None of the bars achieved the required yield loads, the highest load was 124 kN (28,000 lb). The stiffness of the system was also "softer", as expected, averaging 223 kN/cm (127,300 lb/in).

The final phase of the laboratory investigation was the evaluation of the shear strength of the SRD and the No. 7 bolting systems. To evaluate this parameter, the bolts were installed in the

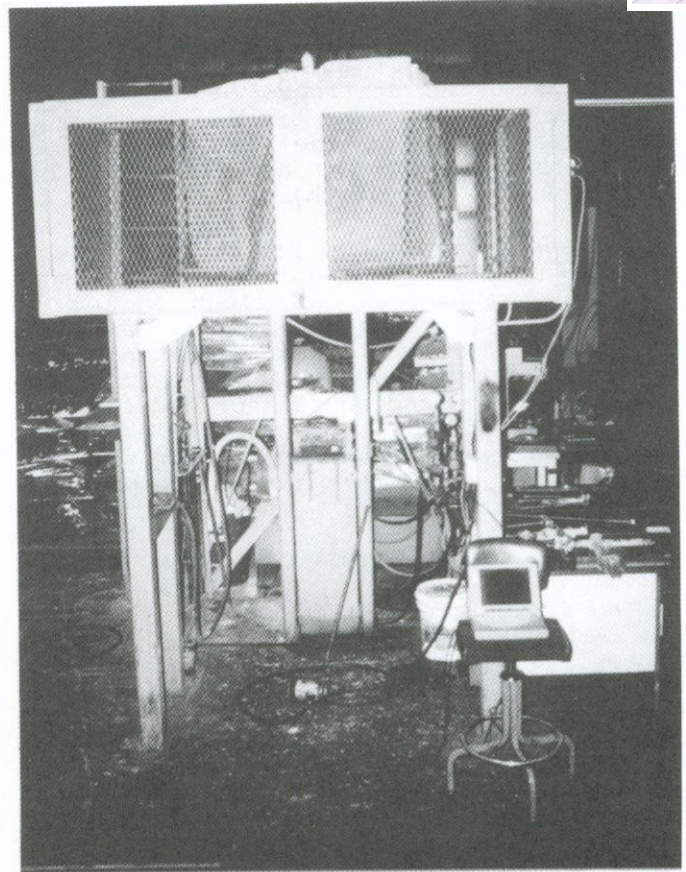


Figure 3. Laboratory test facility complete with drilling and installation equipment.

26.42 and 35 mm (1.04 and 1.375 in) diameter holes. The bolts were installed with 30 cm (12 in) of resin column length and pulled until bond failure occurred. The average shear strength, at the resin/rock interface, for the No. 7 samples was 1.71 MPa (231 psi) and 7.83 MPa (1057 psi) for the tested SRD bolts. The shear strength of the tested SRD bolts with the same resin column length was over 4.5 times greater.

LABORATORY CONCLUSIONS

Both series of laboratory experiments, in steel tubes and limestone rock, concluded that SRD bolts installed in a 25.4 mm (1-in) diameter hole with only 30 cm (12 in) of final resin column length could successfully transfer the applied load and develop the designed yield of the bolt. When the shear strength of the resin/steel tube and the resin/rock interfaces were compared with the No. 7 bar installed in a 35 cm (1-3/8-in) diameter hole, it was concluded that they were greater with the SRD system. Specifically, in the steel tube the shear strength of the SRD was 11.06 MPa (1,493 psi) and the No. 7 bar strength was 6.80 MPa

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(918 psi). In the limestone rock the shear strength for the SRD bar was 7.83 kN (1,057 psi) and the No. 7 strength was 1.71 MPa (231 psi). The stiffness of the systems also indicated the same trend. The average system stiffness of the SRD bar was 315 kN/cm (180,200 lb/inch) and the No. 7 bar was 100 kN/cm (57,100 lb).

The evaluation of the bar deformations, 1 mm (0.04 inches) high, ability to create an effective mechanical interlock system to develop the strength of the bar was also positive. The smooth bar tests indicated that the system stiffness and shear stress capacity was inadequate to develop the capacity of the bar. However, to meet this requirement with this size of resin annulus, higher bar deformations are not required.

FIELD INVESTIGATION

The field investigation was conducted at Jim Walter Resources, Inc. (JWR), No. 5 Mine, located near Brookwood Alabama, which is between Tuscaloosa and Birmingham. The mine operates under 610 m (2,000 ft) of overburden and is extracting the Blue Creek Coal seam of the Warrior Coal Basin. The mining methods employed include mains and longwall panel development by continuous miner units and one retreating longwall unit.

The Blue Creek coal seam contains a low sulfur, medium to high BTU coal which ranges in thickness from 145 to 213 cm (57 to 84 inches). The mining height ranges from 2 to 2.3 m (6.5 to 7.5 ft) with entry widths typically driven 6.1 m (20 ft) wide. The generalized roof strata is composed of a middleman, Mary Lee coal seam and main roof. The middleman is a sandy shale which typically ranges from 0.6 to 2.4 m (2 to 8 ft) thick; the Mary Lee coal seam ranges from 0.46 to 0.6 m (1.5 to 2 ft) thick and the main roof is a minimum of 9.1 m (30 ft) of competent sandy shale and sandstone.

The No. 5 mine has employed a full-bolting plan of primary roof support that consist of full-column resin grouted roof bolts (4). The typical support length is 1.8 m (6 ft). However, two-piece resin assisted, expansion anchor bolts (Double-Lok), 3 or 3.6 m (10 or 12 ft) long, are substituted when the immediate roof conditions weaken.

The testing program was designed to evaluate the anchorage capacity and system stiffness of SRD bolts installed in 3 different hole diameters and No. 7 bolts installed in a 34.9 mm (1-3/8 in) diameter hole. Another parameter that was evaluated during the installation was the amount of force or installation pressure required to push the bolt through the resin. A concern of the reduced hole diameters is the amount

of pressure required to install the bolt and the effects that the increased pressure may have on resin losses and the final resin column length.

Anchorage Capacities

In the field trials, 106.7 cm (42 in) long bolts were installed in a competent middleman roof. The resin column was adjusted based on the diameter of the hole and bar to achieve a final anchorage column length of 45.7 cm (18 in). The SRD bolts were installed in hole diameters of 25.4, 26.2, 28.6 mm's (1, 1-1/32, and 1-1/8-inches). The No. 7 bolt was installed in a 34.9 mm (1-3/8-in) diameter hole. All of the bolts were installed using the resin manufacturers recommended spin and hold times. The bolts were allowed to set a minimum of one hour before the tests were performed. The results, shown in figure 4, are the load-vs-deformation plots obtained in the field for the tested bolts. The final plots are the average of two tests performed for each bolt and hole combination. All of the bolts, installed with 45.7 cm (18 in) of resin anchorage, developed the yield strength of the bar. As shown in the plot, the SRD bolts installed in the various diameter holes show very little differences in anchorage capacity and system stiffness up to the 133 kN (30,000 lb) level. At that time the smaller annuluses resulted in less deformation and higher stiffnesses. As seen in laboratory experiments, the No. 7 bolt installed in the

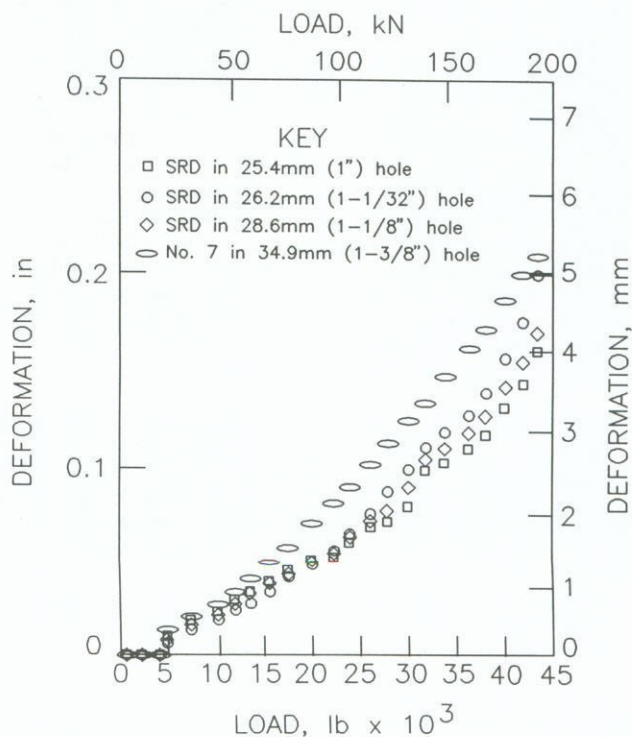


Figure 4. Load vs- deformation plots for field tests.

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34.9 mm (1-3/8-in) diameter hole was able to achieve the strength of the bar but the deformation was larger and hence, the stiffness was less. At a load of 178 kN (40,000 lb) the average deformation on the SRD bolt installed in the 25.4 mm (1 in) diameter hole was 4.55 mm (0.18) inches and the calculated stiffness is about 391 kN/cm (223,500 lb/inch). The deformation of the No. 7 bolt was 4.70 mm (0.185 in) and the stiffness drops to 378 kN/m (216,000 lb/in).

Installation Pressures and Resin Volumes

JWR had installed large numbers of bolts as primary support in their underground operations. A concern that had been noted was that when the SRD bolts were installed with the proper amounts of resin, there was no show of excess resin at the hole bottom above the bearing plate. This occurred in specific conditions where weak layers or slight strata separations were present. The cartridge equivalent length requirements are specified in the ASTM 432-A3.3.1 and A3.3.2 stating cartridges of chemical grouting material shall contain a sufficient volume to fill the annulus between the bolt and borehole wall. Minimum cartridge volumes required for specified equivalent length designations shall be calculated by using the following equation where the equivalent (EL) is equal to the bolt length:

$$V_g = V_h - V_b$$

where: V_g = required grout volume, cm^3 (in^3),

V_h = borehole volume, cm^3 (in^3),

V_b = bolt volume, cm^3 (in^3).

The borehole volume shall be calculated by using the following:

$$\begin{aligned} \text{Borehole Diameter} &= \text{Nominal Diameter} + 1.09 \text{ mm (0.043 in)} \\ \text{Borehole Length} &= \text{Bolt Length} + 25.4 \text{ mm (1 in)} \end{aligned}$$

The bolt volume shall be based on nominal reinforcing bar cross sections in accordance with Specifications A 615. (5)

The required resin volumes were calculated and the amount of force to install the bolt was measured and recorded. As expected, the amount of force is lower when the annulus area is increased. As shown in figure 5, the effective annulus area was compared to the installation stress on the bolt annulus. As a basis a No. 6 bolt (19 mm (0.75 in diameter)) was installed in a 25.4 mm (1-in) diameter hole. The SRD bolt combinations resulted in much higher installation stresses but the actual installation forces never exceeded 24 kN (5,400 lbs). The force required to install

a No. 7 bolt was about 13 kN (2,900 lb), coupled with the larger annulus, this resulted in the lowest installation stresses. The high installation stresses are a function of the small resin annulus which may be forcing resin into the microfractures, sedimentary separations or partings.

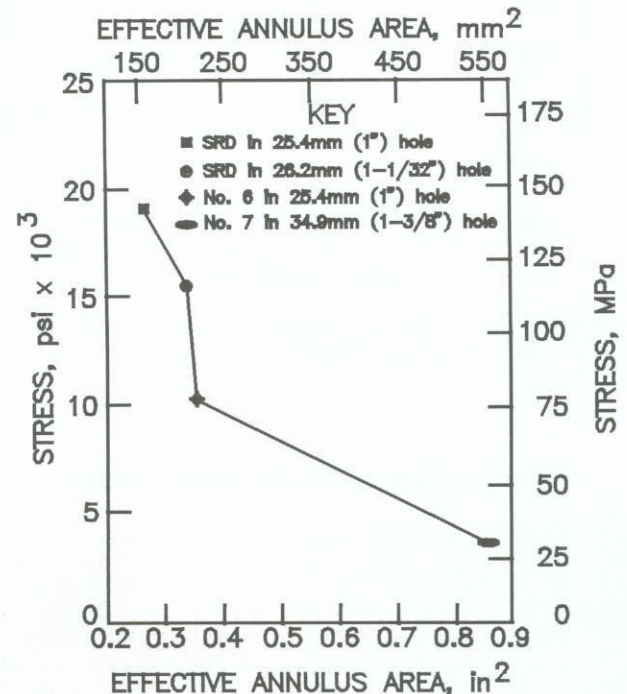


Figure 5. The effective annulus area -vs- installation stresses.

The logical reasons for resin losses can include over-drilling the hole, incorrect resin volumes, and variations in bar volume. These parameters can be easily evaluated and controlled. The parameter that can be evaluated but is difficult to control is the final hole diameter. The volume of the hole can vary with the bit size, concentric rotation of the bit, and the strength of the roof material. When coal bands or weak materials are encountered the hole diameter can increase in diameter. SRD bolts are installed with small volumes of resin and even minor fluctuations in the hole diameter can drastically reduce the final column length. Figure 6 shows the effect that an increase in hole diameter would have on the effective column length of a bolt installed in a 1.5 m (5 ft) long hole with the proper volume of resin. If the hole is drilled just 1.02 mm (0.04 in) oversized, the resin column length is reduced by 25.4 mm (1 in). The chart shows how quickly resin grout length can be reduced as the hole diameter becomes larger.

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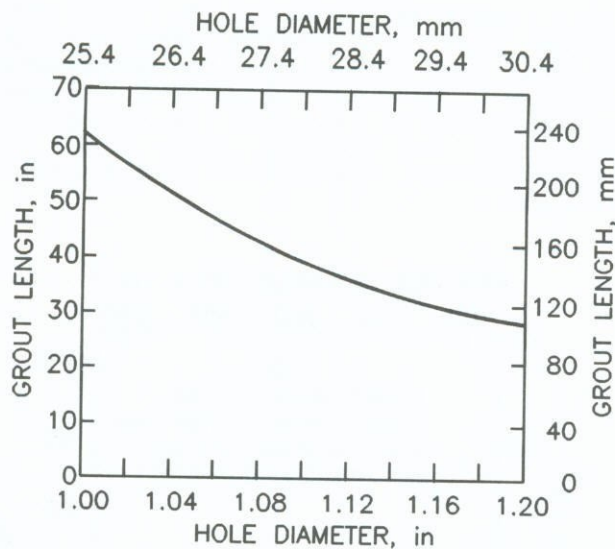


Figure 6. The effects of hole diameter on final grout lengths.

CONCLUSIONS AND RECOMMENDATIONS

Laboratory and field evaluations of the SRD (Special Rolled Dimensions) bolt indicate that the system can be effectively installed in a reduced diameter hole and develop the strength of the bar with 30 cm (12 in) of resin. The system stiffness and shear strength of the reduced annulus bolts were larger in both the laboratory and field experiments. Resin calculation should be performed in accordance with the ASTM 432 requirements and evaluated under field conditions. The installation of the reduced diameter bolts does require higher installation forces, however, these forces are easily obtained with existing bolting machinery. When the grouted length is in competent roof, no resin losses are experienced. The hypothesis for the resin losses experienced during field installations is that the hole diameter is becoming larger when weak roof areas are encountered. This factor, coupled with higher stresses on the resin which forces it into fractures and partings, can account for no resin show at the bottom of the installed bolts. It is important to remember that only 61 cm (2 ft) of resin anchorage develops the strength of this system with a reduced annulus.

The SRD bolt support systems safely reduce resin volumes, provide higher shear strengths and support stiffnesses. These can translate into substantially lower mining costs. The systems evaluation is continuing in large scale support programs for longwall development and subsequent panel extraction. The ultimate goal of this program is to provide engineered support designs that provide safe working conditions under diverse and potentially hazardous ground control conditions.

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Laboratory and In Situ Results of a Slip Nut Yielding Rock Bolt System

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ABSTRACT

Yielding rock bolt systems have been developed to provide ground support while accommodating shifting, creeping, or swelling ground movements. The Dywidag Systems, Inc. (DSI) Slip Nut System is one such system. It has been tested in the laboratory and installed and monitored in the interbedded halite and anhydrite at the Waste Isolation Pilot Plant (WIPP) since 1994.

The Slip Nut System is installed on a point anchored threaded bar or on a threaded bar coupled to a cable bolt. At a pre-engineered load the nut acts as a die and cuts through the thread on which it rests, thereby reducing the load and coming to rest on the next thread where the process repeats.

Two types of laboratory tests were performed using slip nuts on DSI's threaded bars and their Tensionable Cable bolts. Straight pull tests exceeding 0.3m (1 ft.) of displacement were performed on the systems. Also, offset tests were performed on these bolt systems in a test frame designed to simulate WIPP geologic conditions.

A room scale test area consisting of 186 threaded bars began in 1994. Select bolts were instrumented to track bolt load. All bolts were tracked through time for operation. Through three complete years since installation over twelve slips, corresponding to approximately 0.15m (6 in.) of ground movement, have been observed in some areas. Since installation strata offset has exceeded 0.08m (3 in.) at a supported clay layer in the roof. No bolt failures have been observed. Slip loads observed underground generally approximate those observed in the lab. To date, the slip nut systems tested and installed have performed as expected.

INTRODUCTION

The Dywidag Systems International USA, Inc. (DSI) Slip Nut System is a yielding rock bolt support system

developed to accommodate movement in shifting, creeping, or swelling ground found in some mines or tunnels. The system is designed to remain intact despite ground movement and, thus, to enhance mine safety and help contain costs in areas where rebolting of rigid non-yielding systems is typically required.

The Waste Isolation Pilot Plant (WIPP) facility located in southeast New Mexico is a key to the U.S. Department of Energy's national clean-up strategy for transuranic waste generated by defense-related activities. The repository is located 2,150 ft. underground in bedded salt formations which were selected for their inherent ability to creep and eventually seal and entomb the waste emplacements.

Rigid bolt systems installed in such ground have a finite, relatively short life span. The slip nut system is one of several yielding systems being studied to accommodate such ground movement while maintaining ground support. The system responds to load and displacement by yielding at a pre-engineered load. The system is easily monitored by observing slips which occur at the bolt hole collar or the remaining bolt tail length. The results of preliminary laboratory testing and observations of behavior in underground installations are presented.

SYSTEM DESCRIPTION

DSI slip nuts work on a standard No. 7 (22 mm. diameter) grade 60 threadbar. The nut is threaded onto a bar and snugged against its bearing plate (Fig. 1). When load builds to the pre-engineered load level as the result of shifting ground or creeping rock, the nut acts as a die and cuts through the thread, reducing the load and coming to rest on the next thread where the process repeats. Slip nuts may be used on single length, threaded bars or may be installed on DSI's Tensionable Cable Bolts (Fig. 2), a grade 270, seven-strand cable bolt which is coupled to a No. 7 grade 60 threadbar tail. Cables used in these tests were coated with an epoxy resin and had a silica grit impregnated on their outside.

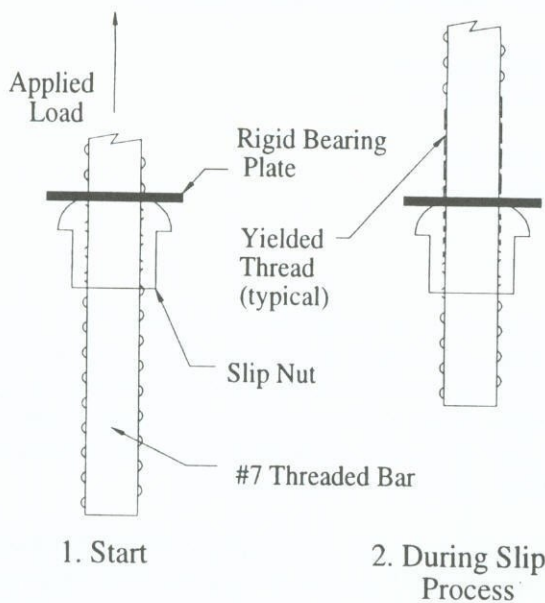


Figure 1: Slip Nut Process

Laboratory tests were performed using threaded bars from two different production facilities, designated Mill #1 and Mill #2 hereafter. Slip characteristics are somewhat different for each. However, results are consistent and repeatable.

Slip nut installations on either the threaded bar or the cable bolt bar require that a tail protrude into the excavation. The amount of yield is a function of the length of the tail. The length protruding into the opening is reduced as the yield process progresses. A full load stop nut can be installed at the end of the tail to transition into a rigid non-yielding system when the slip nut has yielded the length of the tail. If a stop nut is used, the final tail length will be approximately three inches. If no stop nut is installed, the system will lose all bearing capacity when the slip nut cuts the last thread and falls off the end of the bar.

TENSILE TESTS

General description of testing: Bolts were laboratory tested over their prescribed yield range in a 530 kN (120,000 lb.) capacity Baldwin test machine. Tests were in-line, direct tensile tests controlled by displacement. Load and displacement data were collected each second. The data were displayed as a load versus displacement plot in real time and also recorded by the PC based data acquisition system for later analysis. One and a half meter (5 ft.) long bolts were used in the tests. Tests were limited by the approximate 0.2 m (7 in.) stroke of the machine and testing was interrupted as required to reset the test machine.

Results: Testing was initiated and controlled by a displacement rate of approximately 0.013 m/min. (0.5 in/min.). Each of the threaded bars tested were of Mill #1 manufacture and each yielded at or near 89 kN (20 kip). Each cable bolt yielded or slipped at or near the expected 133 kN (30 kip) value; bars used in these tests were manufactured at Mill #2.

Yield is dynamic and sudden. In each test load typically builds to uniform peaks dropping off to zero or near zero load after the slip before reloading. Bolt tails tend to bend slightly below the slip nut during load application. A notable curvature may be exhibited or retained by the bar tail after testing was completed.

A plot of Test DSI#1 data (Fig. 3) is typical for each of the tests performed. Pertinent loads and displacements for cable bolt tests are presented in Table 1.

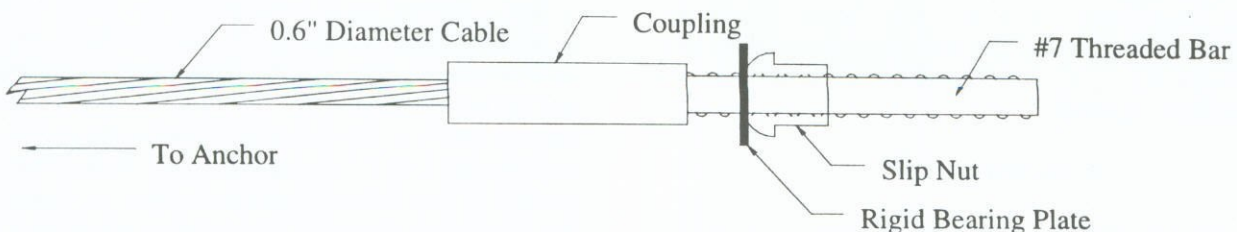


Figure 2: DSI Tensionable Cable Bolt with Slip Nut

Figure 3: DSI Tensionable Cable Bolt with Slip Nut

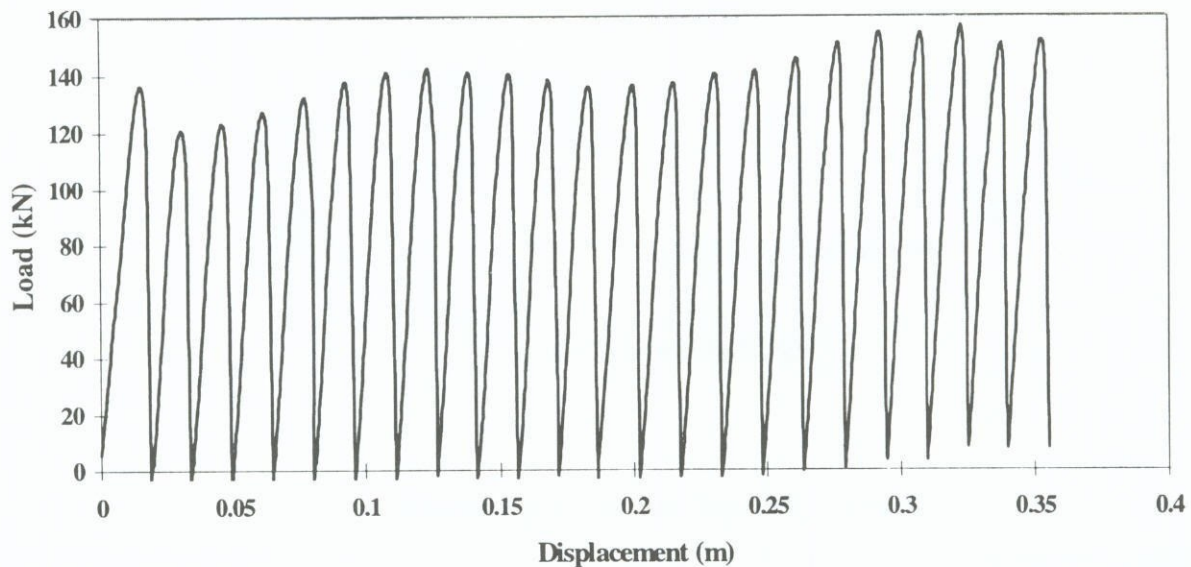


Table 1: Tensile Test Summary Results

Test Type	No. of Tests	Yield Range	Average Yield Peak	Ave. Tested Displacement
Threaded Bar	10	67-102 kN (15-23 kip)	85 kN (19 kip)	0.32 m (13 in)
Cable Bolt	5	120-160 kN (27-36 kip)	142 kN (32 kip)	0.32 m (13 in)

OFFSET TESTS

General description of testing: Bolts were tested in a specially constructed test frame which consisted of an in-line 534 kN (60 ton) hydraulic ram and a 267 kN (30 ton) offset ram capable of loading the bolt normal to the shaft. (Fig. 4). Load and displacement data were manually collected. Calibrated load indicating dial gages were installed on both rams and used to track load. Dial gages were used to determine axial and offset displacement. Each ram was capable of 0.15m (6 in.) of displacement.

Bolts tested were approximately 1.5 m (5 ft.) in length. In the case of the cable bolt, the offset ram contacted the cable portion.

Typically the tests proceeded with axial load being applied such that at least one slip occurred. Then the

offset ram was engaged and the axial ram was maintained. Load was monitored for both rams and displacement data was collected for both the axial (held steady) and offset directions during testing. As with the tension tests, threaded bar tails which were tested were manufactured at Mill #1 while cable bolt bars were manufactured at Mill #2.

Results: Typically tensile load increased to the rated yield point, where the slip nut cut the working thread and load was reduced. Then the process repeated. After initial loading, offset loads were always higher than tensile loads. The peak offset loads tended to increase with each subsequent slip and reload. Figure 5 is a plot of the data from the Cable Bolt #5 offset test and is typical for the tests performed. Each load dip corresponds to a slip of the nut. Table 2 summarizes the test data.

Figure 4: Offset Test Frame

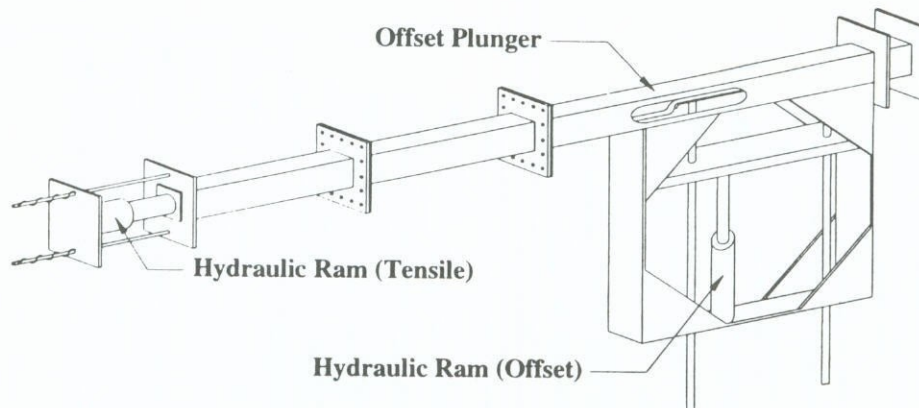
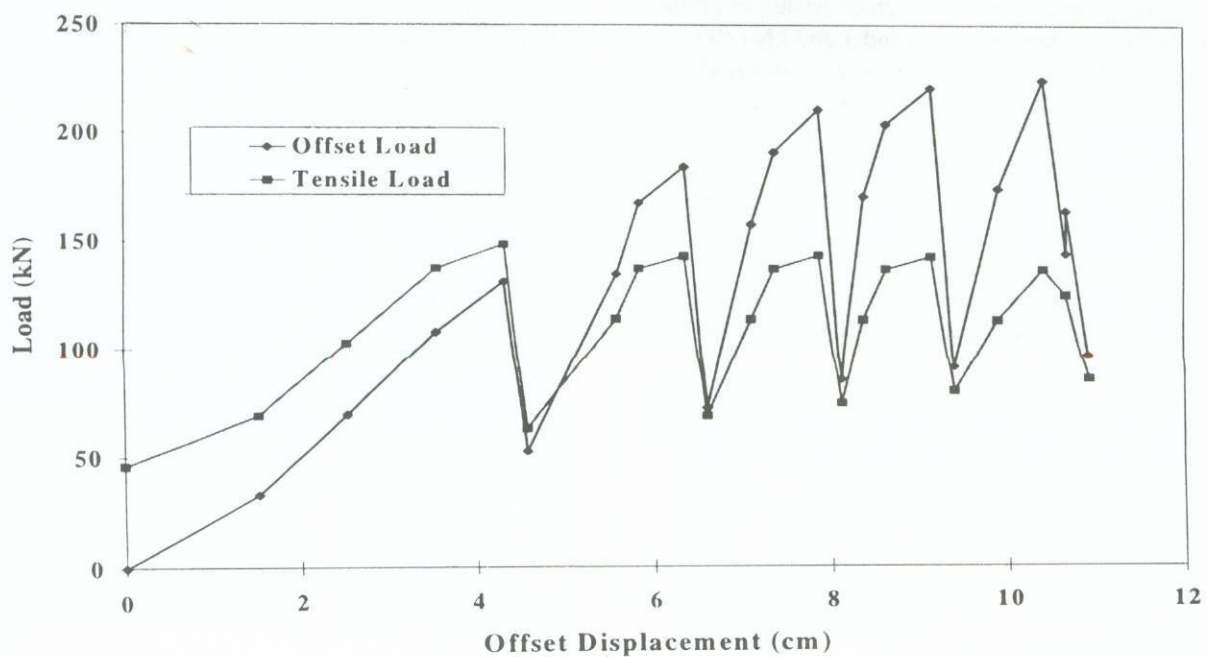


Table 2: Offset Test Summary Results

Test Designator	Peak Tension Load	Peak Offset Load	Offset Displacement	Bolt Disposition
Threadbar #2	103kN (23 kip)	290 kN (65 kip)	0.11m (4.5 in)	Bolt failed
Threadbar #3	92 kN (21 kip)	280 kN (63 kip)	0.11m (4.3 in)	Bolt failed
Threadbar #4	68 kN (15 kip)	247 kN (56 kip)	0.15m (5.8 in)	Intact
Threadbar #5	86 kN (19 kip)	174 kN (39 kip)	0.14m (5.6 in)	Intact
Cable Bolt #2	138kN (31 kip)	204 kN (46 kip)	0.08m (3.1 in)	Failed 4 strands
Cable Bolt #3	138kN (31 kip)	204 kN (46 kip)	0.09m (3.5 in)	Failed 3 strands
Cable Bolt #4	149kN (34 kip)	197 kN (44 kip)	0.07m (2.6 in)	Failed 5 strands
Cable Bolt #5	138kN (31 kip)	224 kN (50 kip)	0.10m (4.1in)	Failed 3 strands

Figure 5: Offset Test





UNDERGROUND INSTALLATIONS

Room scale configuration: A 46m long by 10m wide (150 ft. x 33 ft.) area at the Waste Isolation Pilot Plant (WIPP) was chosen as a test or monitoring area to install threaded bolts and slip nuts. The area is the north half of the middle room of a seven room panel with access drifts at bolt ends. Each room is 91m long by 10m wide by 4m high (300 ft. x 33 ft. x 13 ft.). The room was excavated in 1988.

WIPP strata consists of interbedded halite and anhydrite layers. Typically the anhydrites are underlain with thin clay layers. The roof of the test area consists of approximately 1.5m (5 ft) of relatively pure halite overlain by approximately 0.6m, (2 ft) thick argillaceous halite with a thin clay layer then a several cm. (in.) thick anhydrite layer overlaying the halite layers. Strata offsetting is commonly observed at the clay/anhydrite interface. Above the anhydrite lies another 2m (6 ft.) thick halite bed typically containing a few thin, discontinuous anhydrite stringers. At the time the bolts were installed, the roof beam, which is that portion of the roof from the surface up to the clay layer, had significant fracturing observable on the surface and in boreholes.

In August 1994, 186 threaded bars, 4m (13 ft.) long, were installed in 3.6m (11-3/4 ft.) long boreholes leaving a 0.3m (1 ft.) long bolt tail for the slip nuts to operate. The bolts are resin anchored in the deepest 1.1m (3-1/2 ft) of the boreholes leaving approximately 2.5m. (8-1/4 ft) of free length. The anchorage is located immediately above a clay layer noted above. Bolts are installed on a 1.2m by 1.2m (4 ft. x 4 ft.) square pattern. Threaded bar used in these installations was produced at Mill #1. Select bolts were instrumented with load cells or electric resistance strain gages to track bolt load through time. The room had been previously bolted with 3m (10 ft.) long mechanically anchored bolts many of which had failed.

To facilitate tracking of slips, each threaded bar is outfitted with a strip of color coded reflective tape installed immediately below the slip nut. When a nut slips, the nut covers the tape then personnel document the slip and install a new, differently colored tape strip. A quick assessment of the area may be obtained by sweeping a cap light over the installations; the more active areas of the test area have become quite colorful over time and a bolt with an absence of reflected color indicates a recent slip. Typically this monitoring is performed on a bi-weekly basis.

Tensionable cable bolt configuration: Four cable bolts are installed in an active area of the mine where strata offsetting is prevalent. The location is in a 4.6m high by 7.6m wide (15 ft. x 25 ft.) main. The installations are at the same horizon as the room scale bolts except the roof beam is 0.6m (2 ft.) thinner. However, anchorage in these installations, like the room scale test installations, is located just above the clay layer. In this area the immediate roof beam, that portion of the roof from the surface up to the clay layer, has extensive low angle fracturing and the beam is acting as a cantilever. On the free end of the cantilever, approximately 0.03 m/yr. (1 in./yr.) of downward movement is historically observed and offset along the clay layer is also in the range of 0.03 m/yr. (1 in./yr.) as seen in nearby boreholes.

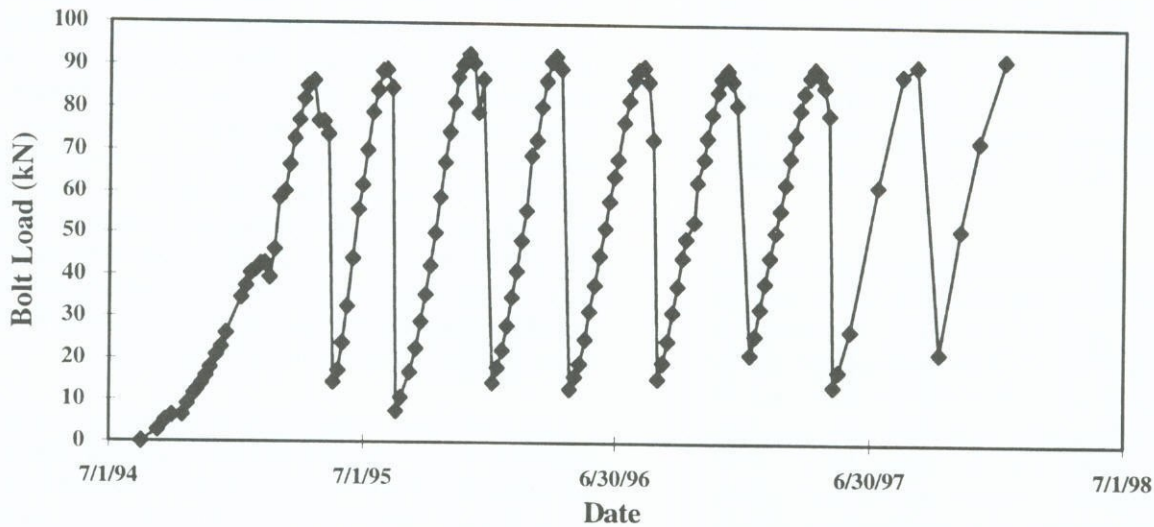
The bolts were installed in 1995 and are installed roughly in a cross-sectional line across the drift. The bolts are nominally 3.0m (10 ft.) long and are installed in 2.7m (9 ft.) deep boreholes with a 1.5m (5ft.) resin anchor. Threaded bar used in these installations was manufactured at Mill #2. Each is outfitted with a rock bolt load cell. Monitoring of the load cells is performed using an electronic data logger on a daily basis.

Observations - Room Scale Monitoring Area: In the three plus years since installation of the bolts in the room scale test area, approximately 80% of the bolts have slipped at least once. In the most active areas bolts have slipped as many as 12 times representing approximately 0.15m (6 in.) of combined roof expansion and strata offset below the anchorage zone.

Data from the rock bolt load cells and load imputed by strain data indicate that slips occur at approximately 89kN (20 kip). This is consistent with peak loads observed in tensile laboratory tests for Mill #1 manufactured bolts. No degradation of load bearing capacity is observed. Figure 6 is a typical plot of load over time for a load cell instrumented bolt installed in the area.

Each of the threaded bars in the monitoring area has withstood the deformation process. Conversely numerous threaded bars with full load nuts installed on a contemporary basis in nearby rooms with the same configuration have failed.

Figure 6: Slip Nut Field Performance



The roof has become increasingly fractured through time. However, data from geomechanical instrumentation installed in the area indicate steady long term ground movement. Roof beam expansion rates, measured by roof extensometers, and convergence rates, measured with tape extensometers are stable, with little or no increase or decrease through time.

application. However, in these especially demanding applications, it shows promise from the standpoint of safety and economics due to its longer life span compared to rigid systems.

The underground slip nut installations will continue to be monitored and evaluated at the WIPP. This is in addition to work with other yielding systems. Further refinements to the slip nut system are currently being investigated and pursued which will engage more than one thread at a time and help to maintain bolt load at or near the peak. It is anticipated that the unique niche requirements for ground control in creeping, shifting, or swelling ground will soon be met by a further developed slip nut system and other yielding support systems now coming onto the market.

Observations - Cable Bolt Monitoring Area: In the approximately three years since installation each of the four bolts has experienced nut slips. In the most active part of the installation, there have been 7 slips on the one bolt. Each of the bars has withstood the deformation process without any indication of failure. Average peak loading prior to slip is approximately 133kN (30 kip). Again this is consistent with lab data collected for Mill #2 manufactured bar.

SUMMARY/CONCLUSION

The DSI slip nut system was tested in the laboratory and later installed in the salt beds of the WIPP. The field installations have accommodated the ground movement to which they have been subjected and have performed as designed without a single bolt failure. This is in direct contrast to rigid, non-yielding bolt systems installed in like ground where numerous bolt failures are observed.

The system has performed in the field as designed and results are consistent with the preliminary lab testing. The system, as with all yielding systems, is unique in its

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Field Monitoring of Rock Bolting Performance in Weak Roof Strata

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ABSTRACT

To understand the reinforcing mechanisms of the fully grouted resin bolt and clarify its performance in weak roof strata, the field monitoring investigation using instrumented bolts has been conducted in a coal mine. The total time of monitoring was around ten months. In the first 8 month period, the rock bolt performance relates to the initial development of the gateroad and subsequent time dependent strata response.

After 8 months, the rock bolt performance relates to the effect of the approaching longwall mining operation, up to the point where the measuring station was destroyed due to the mining operation.

INTRODUCTION

The principal objective of using rock bolting reinforcement in coal mine strata control is to provide improved normal confinement and shear resistance in weak laminated strata. It is generally accepted that the success of ground control using roof bolts is dependent on the three main mechanisms involved in the provision of reinforcement by rock bolts as:

1. Compression: to improve resistance to shear and tension.
2. Forming a composite roof beam support element: to connect several components of the rock layers together, so as to increase the stability of the roof structure.
3. Suspension: anchoring unstable layers into solid ground.

But rock bolts may not always fully provide these functions to effectively support the opening in adverse ground conditions. Particularly, in weak roof strata with characteristics of multi-layers of strata, water sensitivity of roof rock masses, loss of integrity of the overall roof structure can readily occur.

OBJECTIVES

A field monitoring investigate was carried out in Angus Place Colliery, NSW, Australia, by the School of Mining Engineering at UNSW as part of a collaborative ARC research

project including UNSW, Powercoal Pty Ltd, and Springvale Coal Pty Ltd. The objective was to improve the understanding of rock bolting reinforcement using fully encapsulated bolts in weak roof strata, particularly associated with bolt load characteristics and load transfer effectiveness. This included the following particular issues to be investigated:

1. Maximum axial load on the bolts and the load distribution along the bolt and to know the effect of the geological setting of the roof strata on axial load.
2. Variation of axial load with time, to check the time dependent behaviour of rock bolts in weak roof strata.
3. Effect of the mining operation on axial load, to know the performance of rock bolts during the mining process from the time of roadway development and especially when the longwall face was approaching the measurement station.

MONITORING METHODS AND INSTRUMENT LAYOUT

Six strain gauged bolts with lengths of 2.1m were installed in the roof strata. The performance of the fully grouted resin bolts was monitored by placing strain gauges along the bolt. Typically, 18 strain gauges were bonded into grooves on either side of the bolts (9 per side). This allows measurement of the axial force profile developed along the bolt during the different periods.

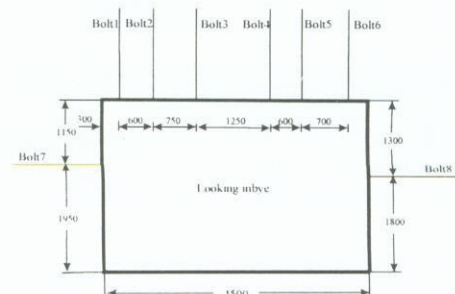


Figure 1. Layout of instrumented bolts in monitored heading

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The measuring station was set up between cut throughs 20 and 21, approximately 20 metres away from 20 cut through (looking in-by), heading 2 (maingate), longwall 22, while the heading was being driven. At the measurement station, a total of eight strain-gauged bolts were installed, including six 2.1m strain-gauge bolts installed in the roof and one 1.5m strain-gauged bolt installed in each side of the ribs, with supporting instrumentation including sonic extensometers and a wire extensometer. Figure 1 shows the layout of the strain-gauged bolts in the measurement station.

GEOLOGICAL FEATURES OF MONITORING SITE

The lithology of weak roof strata plays an important role, particularly with the aspect of reinforcement of the mine roof and its external support. To clarify the geological feature of roof strata, underground logging was carried out. It involved drilling cored roof holes and collecting core samples from four cut throughs located at 3, 7, 12, 17 along the main gate 22, respectively. According to the underground logging, the geological features of roof strata can be summarised as shown in Figure 2.

Figure 3 shows the percentage of rock types within the immediate roof strata. It is noted that, within 8m from roof surface upwards, the main rock types are mudstone, coal and sandstone together with several thin, but very significant claystone bands which were too incompetent to provide geotechnical test data.

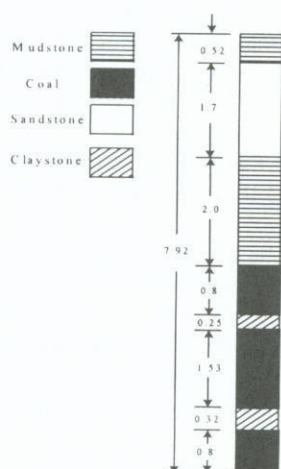


Figure 2. Geological logging on roof strata

Core samples collected from underground logging were tested in the laboratory, the basic mechanical properties of intact mudstone, coal and sandstone are provided in Table 1.

It should be pointed out that all the values provided in Table 1 are subjected to the condition of 0% of water content.

The effect of water content on the mudstone, coal and sandstone has been observed during laboratory testing and

Table 2 gives a brief description of sensitivity of rock types to water content, after the samples were submersed into water for over three hours.

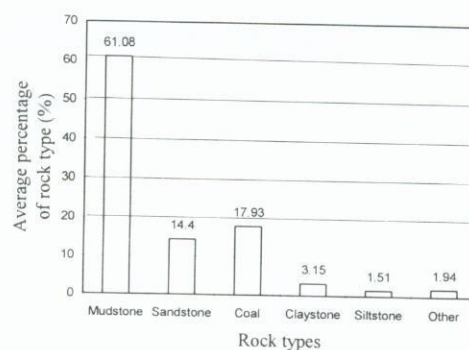


Figure 3. Percentage of rock types in roof strata

Table 1. Mechanical properties of roof rocks

	UCS (MPa)	E (GPa)	σ_t (MPa)
Coal	42.6	2.95	3.41
Mudstone	81.9	5.88	4.96
Sandstone	48.5	5.92	5.02

Table 2. Effect of water on mechanical properties of intact samples.

No.	Rock Type	Description
1	Mudstone	Horizontal fractures and separations appear on the whole samples, and the samples were destroyed to a number of small pieces. There is negligible strength in the horizontal direction.
2	Sandstone	Swell and separation, some slakes to sand, no strength in both horizontal and vertical directions. Core samples have become so soft that the sample can not be picked up by hand.
3	Coal	Fractures start to appear on the samples; not much variation in volume.

It is indicated that the water content significantly influences the mechanical properties of mudstone and sandstone. According to tests conducted on the effect of water content on compressive strength, it noted that the uniaxial compressive



strength of mudstone reduced by up to 46% when the water content changed from 0% to 3%.

REINFORCEMENT SYSTEM USED IN MONITORING SITE

The fully grouted resin bolts, 2.1m in length and 22mm in diameter have been used in Angus Place Colliery. The bolts are a high tensile steel known more commonly as AX bar, with a yield capacity of 24 tonnes and breaking strength in excess of 30 tonnes.

According to reinforcement designation, there were 6 bolts per m² in the roof and one bolt was installed on each side of the ribs with a spacing of one metre.

Considering the characteristics of the weak roof strata, the rock bolts were supplemented with steel mesh.

UNDERGROUND MEASUREMENT PROCEDURE

The measurement and data collection were carried out after the instrumented bolts were installed at the face (within 1m). The data was collected regularly throughout the whole period of the mining process (approximately ten months).

In order to ensure the monitoring covered all stages of the roof deformation, the first reading was taken at two metres away from the face of the heading while the heading was being driven and the last reading was completed at one metre away from the longwall face during the longwall retreat.

Before mining extraction commenced, readings were taken once a month, and after mining commenced, readings were taken at least once per day.

MONITORING RESULTS AND ANALYSIS

It has been known that the capacity of grouted anchorage is a function of many variables which include the intact shear strength of the rock, the shear strength of the grout - rock interface, the bolt-grout interface, the rock mass modulus, Poisson's ratio, and the bolt and borehole diameter (1). But the effective performance of a rock bolting system in roof strata of coal mine roadways can be indicated by:

1. Maximum axial load,
2. Time dependent characteristics of axial load,
3. Axial load distribution along the length of bolt,
4. Effect of mining proximity on axial load.

Consequently, the monitoring results and analysis following those four aspects of the bolting system are provided below.

Maximum Axial Load on Bolts

The transmission of forces in the bolt - grout - rock system is

usually described on the effect of the shear bond strength between rock - grout and grout - bolt, and it is usually represented by the axial load on the bolt. Also it is the most direct parameter that indicates the working state of the rock bolt in the field.

According to the principles of instrumented bolts, it has been known that the instrumented bolt used in monitoring is not a direct load measuring device, so the data collection involved measurement of strain along the bolt and then converted to axial load. It is possible to determine not only the magnitude of axial load, but also the position which corresponds to the load.

From the monitoring data, it was noted that the maximum axial load measured by six instrumented bolts installed on the roof strata varied from 91 to 227kN during development (before mining extraction) and from 210 to 256kN as the longwall approached.

Table 3 summarises maximum and average axial loads on the bolts before and after mining extraction has commenced and the position corresponding to those loads.

From Figure 4, it can be seen that the variation of axial load for each bolt after longwalling is relatively smaller than the variation after development, which was significantly non-symmetrical across the heading. Also, as expected, the axial load of the bolts increased with decreasing distance between bolt and the faceline of longwall 22. It implies that, before mining, the stress induced by previous panel mining played an important role, together with possible geological variation and possible horizontal stress directional effects adversely influencing the right hand side of the heading. It resulted in a non-symmetric loading condition on the roof strata. After longwalling commenced, the bolts worked in a similar condition across the full width of the heading.

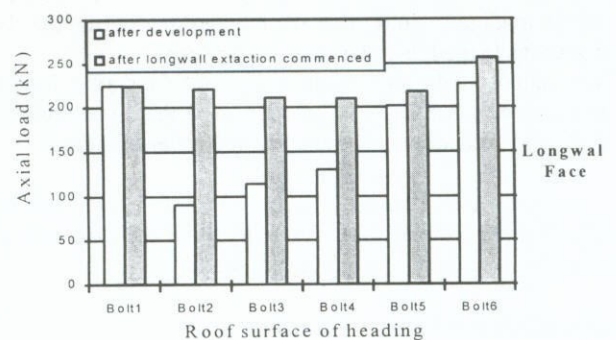


Figure 4. Axial load distribution across the heading

From Table 3, it is noted that the position corresponding to the maximum axial loading of the bolts was mainly concentrated at and above 1.625m depth into the roof. Comparing with the geological logging of the roof strata

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Table 3. Maximum tensile load on the bolts

Bolt No.	Max.Load(kN) (before mining extraction)	Ave.Load(kN) (before mining extraction)	Max.Load(kN) (after mining extraction)	Ave.Load(kN) (after mining extraction)	Position (m)
1	224.7	135.7	224.4	200.2	1.225-1.425
2	90.8	65.1	220.6	200.6	1.625-1.825(bef M.) 1.025-1.225(aft M.)
3	114.1	76.4	210.8	177.5	1.425-1.625(bef. M.) 1.425(aft M.)
4	129.9	77.3	209.6	183.2	1.425
5	201.4	174.4	217.4	208.9	1.825(bef M.) 0.625&1.825(aft M.)
6	226.7	140.4	256.4	236.0	0.825(bef M.) 1.425-1.625(aft M.)

as shown in Figure 2, it is known that maximum load occurs in the area of dull coal, interbedded with carbonaceous mudstone which is one of the weakest areas in the roof strata. Higher locations in the roof where load build-ups occurred also corresponded to the claystone band horizons.

Time Dependant Behaviour of Average Axial Load

The time dependent behaviour of average axial load was monitored totally over 200 days in the longwall maingate heading. Figure 5 shows the maximum average axial loads on the bolts relative to time, up until mining extraction reached the instrumentation site. It is noted:

1. The average axial loads increased initially as the heading was driven.
2. The rates of variation of average axial load carried by each bolt across the heading were different. They may be influenced by the geological features of the roof strata and /or the loading condition on the roof strata, that is, the roof strata was loaded in a non-uniform manner.
3. After initial load build-up (maximum 60 days after heading development), variation of average axial load was relatively small, until the longwall mining started (after 200 days).

In general, apart from mining induced load variation (development and extraction), the variation of the load on each bolt does not significantly change with time.

Axial Load Distribution

The axial load distribution in a fully grouted rock bolt is caused by longitudinal and shear displacements in the rock mass, generating load transfer into the bolt. Figure 6 provides typical load distributions along the length of the bolts at different times recorded. Here the first four graphs represent axial load distribution after development and the last four graphs represent distribution of axial load after longwall mining has approached the site.

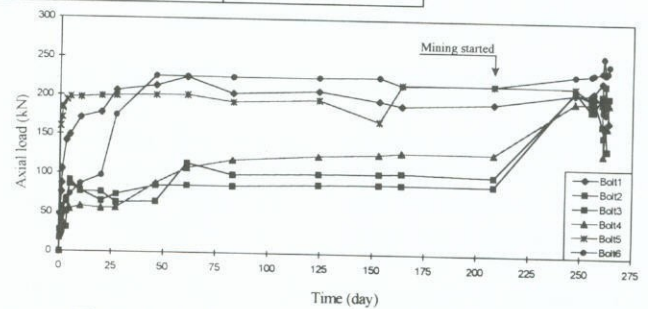


Figure 5. Variation of average axial load with time

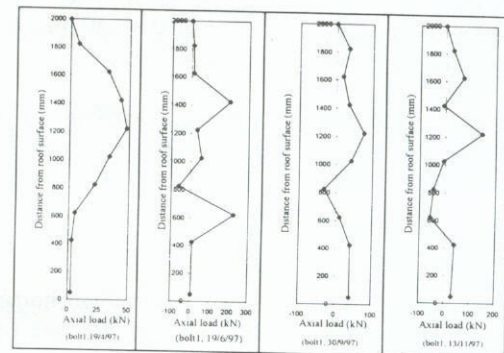


Figure 6. Axial load distribution on the bolts (a) Before mining

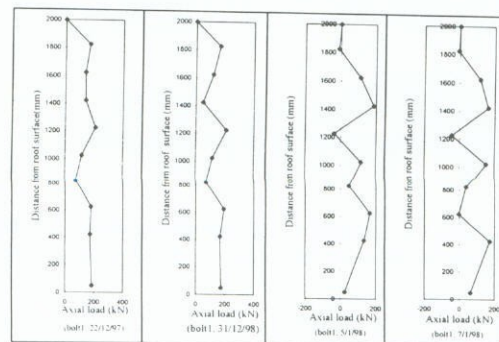


Figure 6. Axial load distribution on the bolts (b) After mining

The mechanisms of axial load distribution may be more complicated, but according to the field monitoring and observation, three reasons may mainly contribute to the axial load distribution along the length of bolt as:

1. Geological feature of roof strata,
2. Stress re-distribution
3. Time dependent behaviour of roof strata.

Axial load distribution is influenced by the characteristics of strata geology. Tao and Chen(2) indicated that the strain distribution of bolts with a single peak can be found when the rock structure is comparatively uniform, and multiple peaks along the length of bolt are a result of the heterogeneous of the rock structure with multiple partings generating load transfer.

According to the geological logging, the roof structure of the heading was extremely heterogenous within 2m from the roof surface. It includes dull coal and claystone with interbedded mudstone. It is believed that this is one of the reasons that multiple peaks of axial load were recorded along the length of bolts during the monitoring, as different layers with various mechanical properties behave differently when load is applied on the roof strata.

Stress re-distribution induced by the heading excavation may be another factor which influences the distribution pattern of axial load. After this heading was driven, the roof bolts were installed immediately, meanwhile, the readings were taken and it was noted that a comparatively smooth distribution of axial load was obtained in the initial results. Then the multiple peaks of load distribution along the length of bolt were recorded due to subsequent roof movement induced by stress re-distribution while the heading was being driven. As separation of roof layers occurred, multiple peaks of axial load can be observed. After stress re-distribution, the axial load on the bolt is mainly influenced by the creep characteristics of rock mass induced by the weathering and water around the excavation. As indicated in the geology feature of roof strata, the mudstone and sandstone are very sensitive to the water content. With time increases, the roof movement and separation has taken place gradually at different levels of roof strata, affected the long term loading pattern and distribution of axial load on the bolts.

Effect of Mining Proximity on Axial Load of Bolt.

From Figure 4, it was seen that when longwall mining started, the axial loads increased significantly compared to the maximum loads after development only. To present more details of variation of axial load after longwalling commenced, the maximum bolt load relative to distance between the measuring station and the longwall face is shown in Figure 7.

It can be seen from Figure 7 that the axial load along each part of the bolt increases with similar speed to the maximum load when the mining operation started. Then, after the 55m

distance between the longwall face and the measuring station, the axial loads along the length of each bolts remained at a certain level without significant change. It is noted that the axial load reduced and changed to compressive load at some measuring points on the bolt when longwall face was very close to the measuring station.

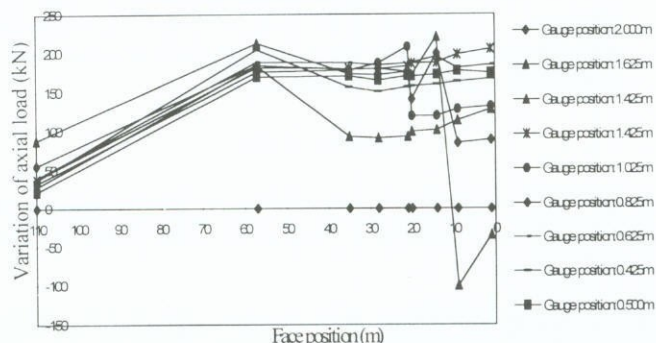


Figure 7. Effect of mining operation on axial load along the length of bolt #3)

Comparing the different stages including heading excavation, and subsequent time dependent stages during the monitoring process, the mining proximity is a major influence on rock bolt performance due to induced stresses.

Initially, the axial load of the bolts increases to the maximum value with reducing the distance between the longwall face and the measuring station. It was also observed that the value of axial load was significantly reduced in some parts of the bolts when the longwall face was very close to the measuring station. This suggests that the cohesion between bolt-resin-rock may have been damaged due to movement of roof strata induced by the mining operation.

CONCLUSIONS

Generally, the rock bolt supporting system in longwall headings of coal mines is influenced by a) initially, the development of heading and mining induced stress, b) time dependent characteristics of roof strata, and c) effect of mining operation. According to the monitoring results, the conclusions can be drawn as:

1. The performance of the roof bolts was significantly influenced by the longwall mining operation, as monitoring results indicated that the ratio of maximum load after development is only 36% of maximum load after longwall approach.
2. Before mining, the performance of the roof bolts in the gateroad was dominated by the previous panel mining-induced stress re-distribution and heading excavation.
3. After initial settlement and before the longwall mining operation, the variation of axial load along the bolt with time depends on the physical properties of rock at different layers, the more stable the roof rock, the less variation of



axial load and vice versa.

4. The axial load distribution along the length of the bolts presented a multiple peak pattern due to the heterogeneous nature of layered roof structure and potential delamination of bedding planes.
5. When the longwall face was very close to the measuring station, significant reduction of axial load on the bolt was observed. It indicated that the bolt may no longer be providing the full function as the interface between rock - resin - bolt is destroyed by mining.

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A Case Study of Bolt Performance in a Two-entry Gateroad

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ABSTRACT

This paper presents the results of a case study conducted in a two-entry gateroad in a coal mine where excessive roof deformation and bolt loading resulted in failure of many roof supports. The instruments consisted of 16 fully grouted, strain-gaged resin bolts, load cells on both full and partially grouted cable bolts, and vertical deflection multipoint extensometers. These instruments were installed on-cycle to measure loading on bolts in an existing support pattern and to determine if cable bolts could be used to improve roof stability.

Results showed significant amounts of movement within the bolted zone. In some cases, these movements were enough to load the supports past their ultimate strength. Geology appeared to be a significant factor in localized roof degradation; that is, a very weak rock layer in the immediate roof overlain by a very strong rock layer contributed to the development of shear planes.

Only one instrumented bolt had a maximum strain of less than 2,000 microstrain, which was the yield point of the steel. The average maximum strain on all bolts was 20,000 microstrain. However, electrical continuity to many gages was lost, and a pattern was noted that would indicate possible bolt failure. Wire mesh and concrete cans installed as secondary support performed very effectively.

INTRODUCTION

Millions of roof bolts are installed in U.S. mines each year. Many of these supports fail, resulting in more than 400 injuries and 10 fatalities a year. Despite the importance of entry stability, there are no adequate methods to relate roof bolt loads, diameter, length, and spacing to mining conditions such as geology, geometry, and in situ stress fields. The Spokane Research Laboratory (SRL), a division of the National Institute for Occupational Safety and Health (NIOSH), is conducting research on roof bolt loading to reduce support failure and subsequent ground falls.

Fully grouted roof bolts have become the primary support in underground coal mines. This type of support responds to rock movements, which results in a nonlinear distribution of load along the length of the bolt. Previous laboratory and field studies (1-10) have shown that strain gages can be used to measure both axial and bending loads on each bolt in the pattern, where loads are being developed, and how much anchorage length is available to provide support. Such information can be very useful in establishing safe support patterns for various mining conditions.

Excessive roof deformation and bolt loading were occurring in several gateroad entries at the Trail Mountain Mine near Orangeville, UT. The problem areas were located in a two-entry development where entry widths were approximately 6 m (20 ft). Pillars were spaced on 15- by 31-m (50- by 100-ft) centers. The coal seam was 3 to 3.4 m (10 to 11 ft) thick and was mined to a height of approximately 2.4 to 2.7 m (8 to 9 ft). Overburden thickness was 427 to 487 m (1,400 to 1,600 ft). The primary roof support was 2.1-m (7-ft), No. 6, grade-60 fully grouted bolts that were spaced approximately 1.5 m (5 ft) across the entry and 1.5 m (5 ft) along the entry. Wire mesh and roof mats were also installed. Secondary support consisted of two rows of 0.9-m (3-ft) diameter can cribs spaced approximately 2 m (7 ft) on center down the entry with a 1.2- to 2.4-m (4- to 6-ft) wide walkway down the middle. The cribs were installed approximately 30 m (100 ft) ahead of the longwall face.

Geology may have been a significant factor in the localized roof degradation. Several 2.7- to 4.6-m (9- to 15-ft) deep core-holes were drilled, and AX core was removed and tested to assist in a geological assessment of the roof. The immediate roof in the bad areas had layers of very weak mudstone, carbonaceous mudstone, and coal. Immediately above these weak layers was a very strong siltstone-sandstone member that had an average compressive strength of 179 MPa (24,600 psi). The mudstone was too weak to allow recovery of a sample large enough to test. The core log near crosscut 59.5 is shown in figure 1.

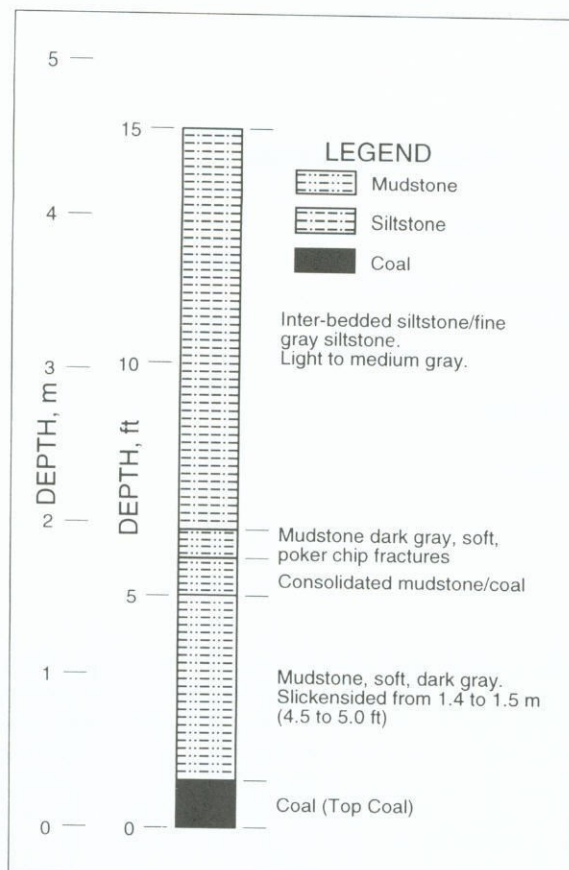


Figure 1.—Core log near crosscut 59

Sixteen instrumented, fully grouted bolts, as well as sagmeters and pressure pads, were installed on-cycle in three test areas in the 10th Right gateroad to measure loading on bolts in the existing support pattern and to determine if cable bolts could be used to improve roof stability. This gateroad was the last in a sequence of longwall panels (figure 2).

TEST AREAS

In the first test area, a combination of fully grouted cable bolts and resin bolts was used (table 1). Two rows of instruments were located approximately 12 m (40 ft) outby from the center of crosscut 62. The resin bolts and cable bolts were alternated as shown in figure 3A.

In the second test area, a combination of partly grouted cable bolts and resin bolts was used (table 1). Two rows of instruments were located approximately 8 m (27 ft) inby from the center of crosscut 62 (figure 3B).

Two rows of instrumented No. 7, grade-60, 6-ft-long resin bolts were installed in the third test area as the primary support. The first row was 4.9 m (16 ft) inby the center of crosscut 56, and the second row was 19.8 m (65 ft) inby the center of crosscut 56 (figure 3C-D). The connector to an instrumented bolt in this test area was severed during installation. The uninstrumented resin bolts were 2.1-m (7-ft) long and were No. 6, grade 60. Entry development and longwall pass of the test rows are shown in table 2. This gateroad had only one longwall panel pass.

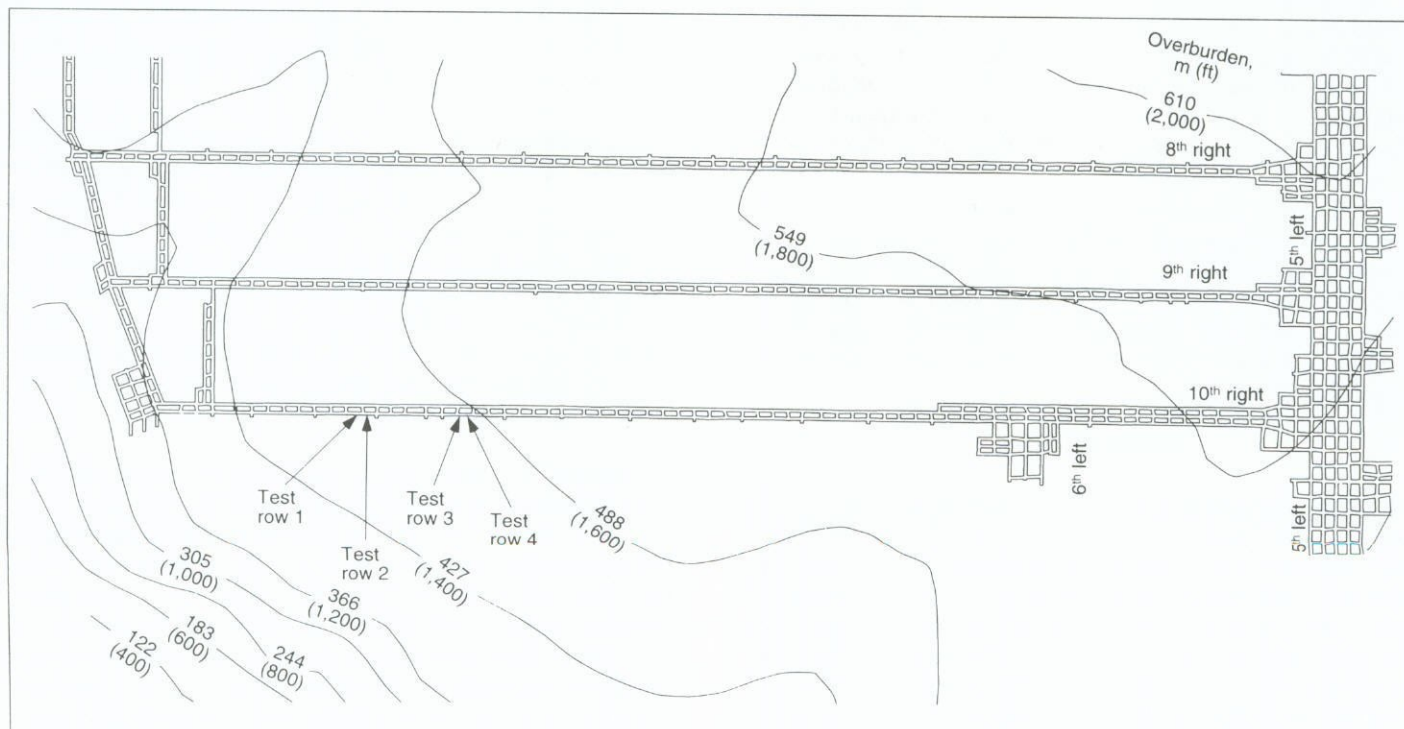


Figure 2.—Plan view of test areas

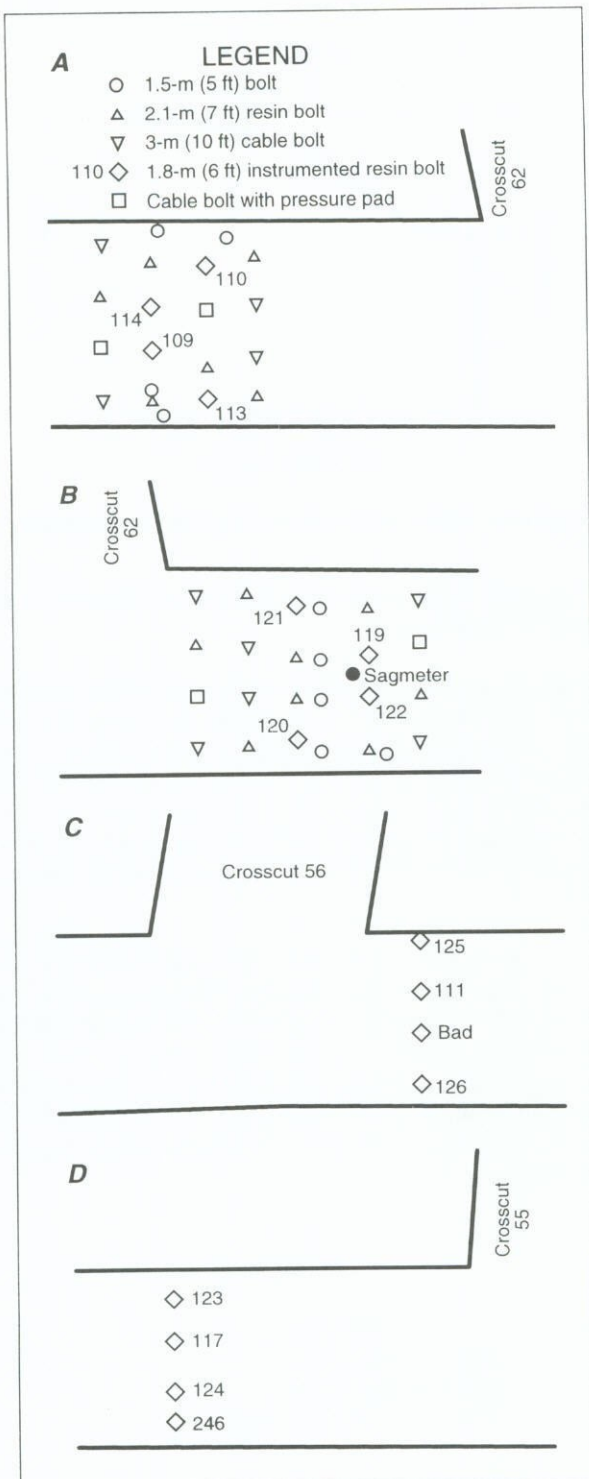


Figure 3.—Test rows. A, Fully grouted bolts; B, partly grouted bolts; C, resin bolts, row 3; D, resin bolts, row 4.

Table 1.—Description of cable bolts in test rows 1 and 2

	Length	Resin length	Diameter	Ultimate strength
Test row 1:	3 m	2.7 m	18 mm	356 kN
	10 ft	9 ft	0.7 in	80,000 lb
Test row 2:	3 m	1.5	15 mm	267 kN
	10 ft	5 ft	0.6 in	60,000 lb

Table 2.—Significant dates of test rows.

Test row	Developed	Cribs installed	Longwall pass
1	12/10/97	4/16/98	4/21/98
2	12/9/97	4/18/98	4/22/98
3	12/1/97	5/17/98	5/19/98
4	12/1/97	5/18/98	5/20/98

INSTRUMENTS

The resin bolts were modified by milling a slot 6.4 mm (1/4 in) wide and 3.2 mm (1/8 in) deep. Five strain gages were attached on both sides, as shown in figure 4. Because these bolts were used for primary support, No. 7 bolts were used in place of the No. 6 bolts. Table 3 describes the properties of the two bolts. Note that the No. 7 bolt has an increased capacity of approximately 28%.

Table 3.—Properties of bolts

Bolt type	Yield load	Ultimate load	Cross-sectional area
No. 6	107 kN	176 kN	2.58 cm ²
	24,000 lb	39,500 lb	0.40 in ²
No. 7, slotted	149 kN	249 kN	3.61 cm ²
	33,500 lb	55,900 lb	0.56 in ²

An Omnidata Polyrecorder data acquisition system (DAS) was used to measure load on the instrumented bolts. Readings were taken immediately after installation and then periodically as mining-induced stresses changed. The strain gages on the bolts were wired into a Wheatstone bridge configuration using 350-ohm resistors and a 5-V excitation. The accuracy of the system is ± 0.445 kN (100 lb).

Loads on the cable bolts were measured on a 64.5 cm² (10-in²) bearing plate with hydraulic pressure cells. Vertical roof movements were measured with multipoint extensometers with anchors set at depths of 4.3, 3, 1.8, and 0.9 m (14, 10, 6, and 3 ft) from the roofline.

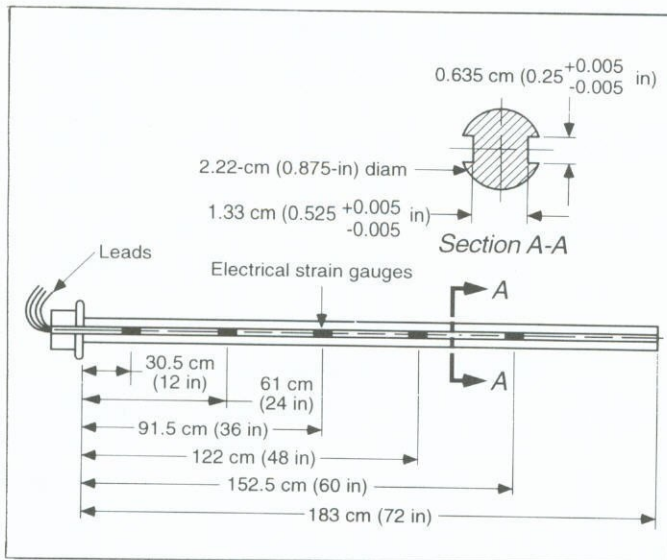


Figure 4.—Instrumented bolt

TYPES OF BOLT LOADS

Bolt loads can be axial, bending, and/or shear. Axial loading is generally the primary force on a steel bolt, although under some situations, combinations of axial, bending, and/or shear forces can cause bolt failure. Shear loads are impossible to estimate with this type of instrument because of the nature of the loading mechanisms and uncertainties in determining the point of loading. However, when joint movement is present, shear loading can be critical in the design of bolt systems, and additional research is required for a better understanding of this loading mechanism.

A design engineer should consider several factors when calculating bending moments measured by instrumented bolts. Maximum bending moments may be localized and not measured accurately. Bending is measured in only one plane, but can take place in other directions, especially if high horizontal stress fields are present. (At our test sites, bolts were oriented during installation to measure the highest estimated plane of bending.) Bending moments can also be caused by joint movement, large-block rotations, and/or differential loading in mats and meshes.

Another, equally important, aspect of selecting bolts for roof support is the evaluation of strain levels in the bolt (figure 5). Typical engineering design limits strain to a percentage of the yield point. Previous evaluations of strain measured on roof bolts show that, in many cases, the yield point of the steel bolt is exceeded where the roof remains stable. Fully grouted roof bolts are a stiff support system in which loads increase quickly as the bolted strata move. Rebar bolts are made from a ductile steel that can reach strain levels of 100,000 to 160,000 microstrain at ultimate load. Yielding occurs at approximately 2,000 microstrain on grade 60 rebar. Immediately after yielding, the bolt will continue to stretch, with very little increase in load until the steel begins to work harden. Thus, design strain limits should take bolt loading mechanisms (i.e., axial, bending, and/or shear) into consideration. Bolts loaded axially with little bending or shear

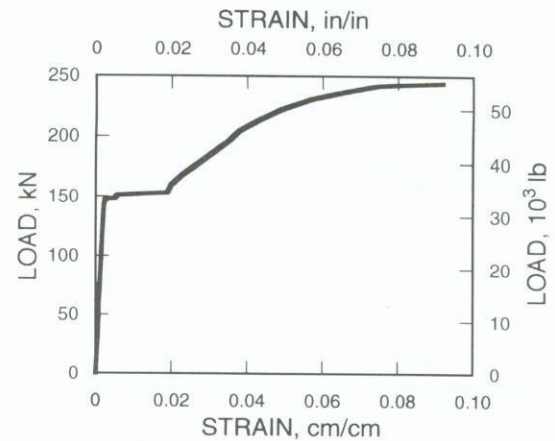


Figure 5.—Load strain plot for No. 7, grade 60, slotted bolt

load can reach higher strain levels than bolts subjected to high shear and bending forces.

Data from each instrumented bolt were evaluated in several different ways. Each bolt was calibrated in a uniaxial testing machine to correlate voltage change to load change. This calibration factor was used below the steel yield point to convert voltage data to load at each of the 10 strain gages. If a bolt section exceeded the steel yield point, the strain was calculated and load estimated from the stress-strain relationship shown in figure 5.

Axial loading was calculated by averaging the load on each side of the bolt at each gage location. Bending moment was calculated with standard beam equations. The section modulus used to calculate bending was determined both experimentally and mathematically. If a strain gage fails, then neither axial load nor bending moment can be calculated for that bolt location. Strain was calculated directly from the voltage readings based on the equation—

$$\epsilon = \frac{4 \Delta V}{(GF)(EV)}$$

where ϵ = strain, in/in,
 ΔV = change, V,
GF = gage factor,
and EV = excitation voltage.

RESULTS

Bolt loading is shown in figures 6 through 22 as a cross-sectional view. Each test row is shown at various time intervals. Figures 6 through 13 show the development of axial loading in the row of fully grouted cable bolts. On December 12, gage 4 on bolt 114 had reached 43,000 microstrain, well into the strain-hardening phase of the steel. When that bolt was read again 4 days later, that gage had an open reading and axial loading on the adjacent bolts had increased. Three weeks later, the strain gage

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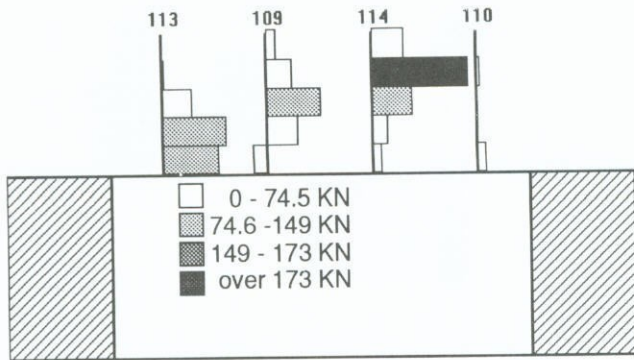


Figure 6.—Axial loads on test row 1 on 12/12/97

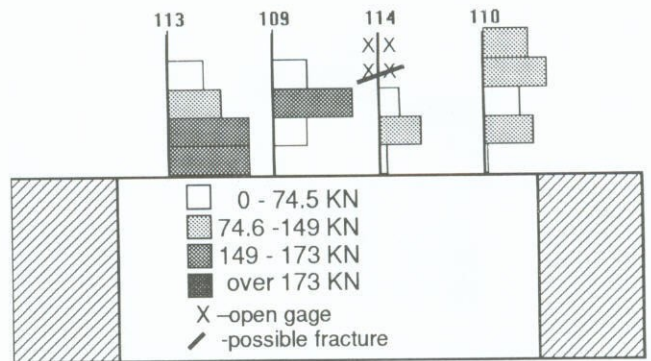


Figure 10.—Axial loads on test row 1 on 4/19/98

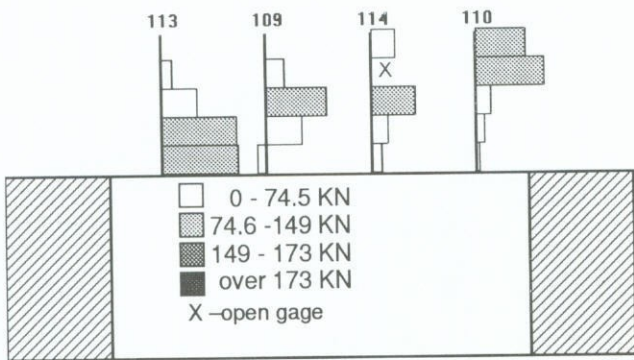


Figure 7.—Axial loads on test row 1 on 12/16/97

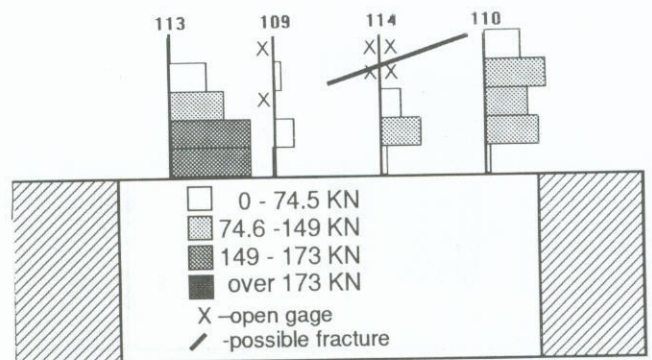


Figure 11.—Axial loads on test row 1 on 4/21/98

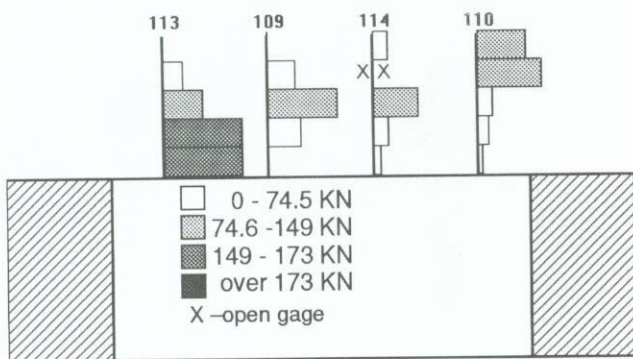


Figure 8.—Axial loads on test row 1 on 1/9/98

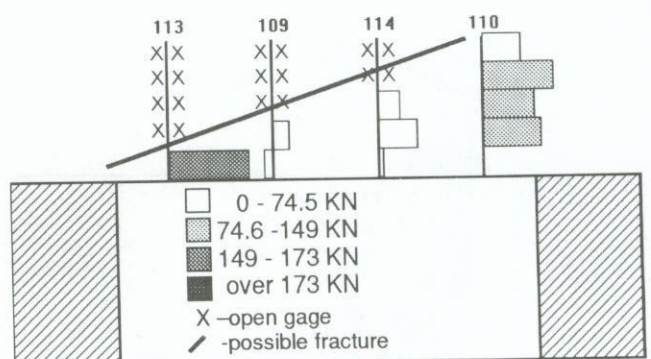


Figure 12.—Axial loads on test row 1 on 4/23/98

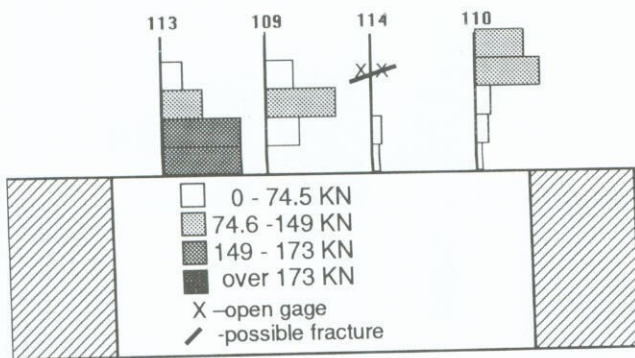


Figure 9.—Axial loads on test row 1 on 1/15/98

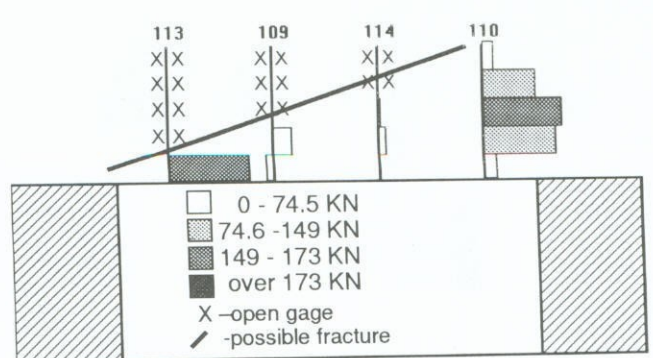


Figure 13.—Axial loads on test row 1 on 6/2/98

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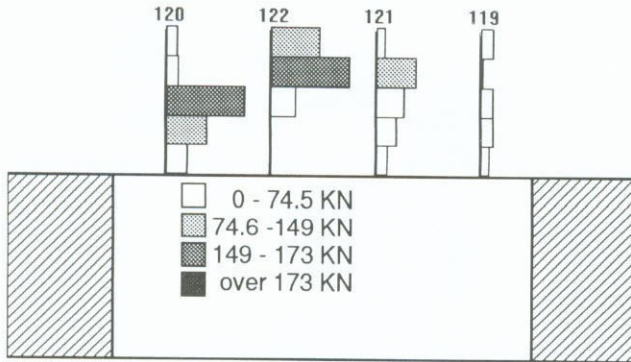


Figure 14.—Axial loads on test row 2 on 12/16/97

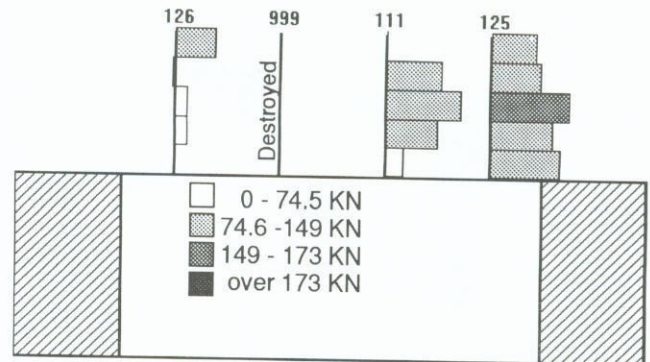


Figure 18.—Axial loads on test row 3 on 4/10/98

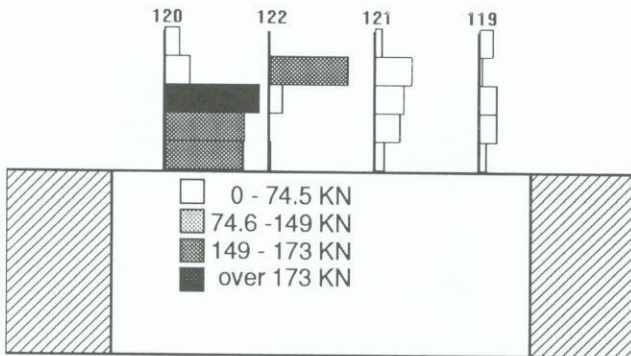


Figure 15.—Axial loads on test row 2 on 4/21/98

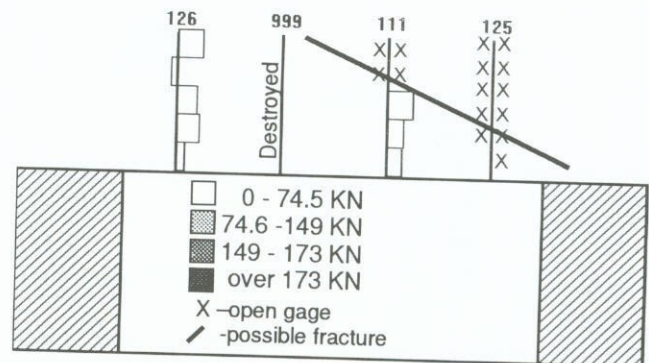


Figure 19.—Axial loads on test row 3 on 6/2/98

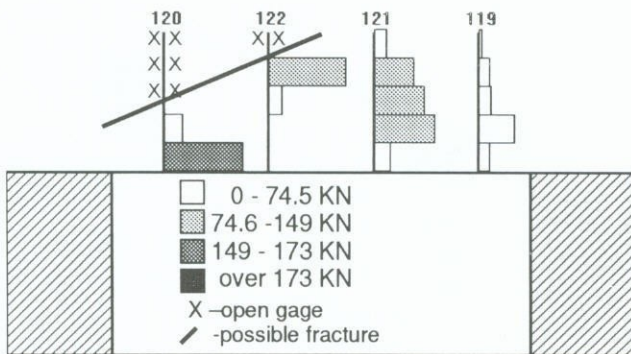


Figure 16.—Axial loads on test row 2 on 6/2/98

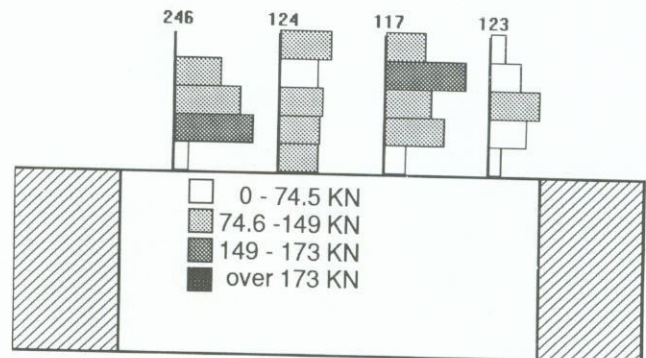


Figure 20.—Axial loads on test row 4 on 12/16/98

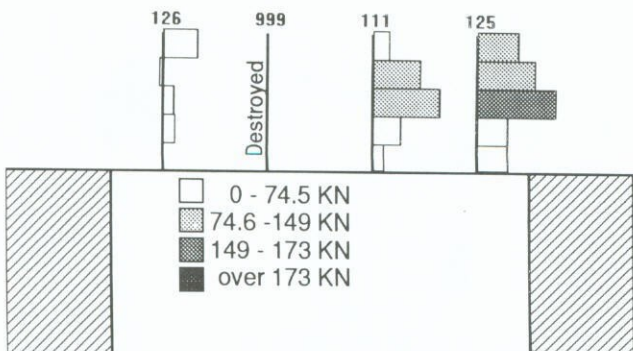


Figure 17.—Axial loads on test row 3 on 12/16/97

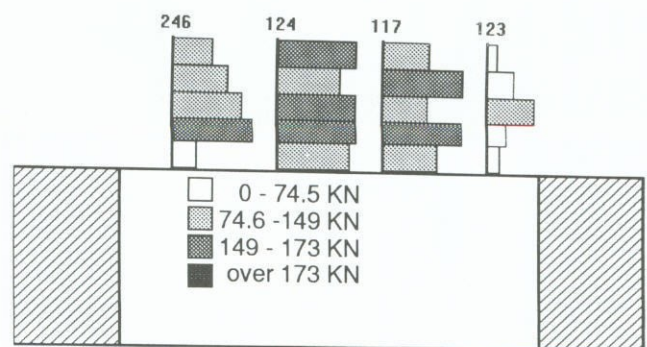


Figure 21.—Axial loads on test row 4 on 5/6/98

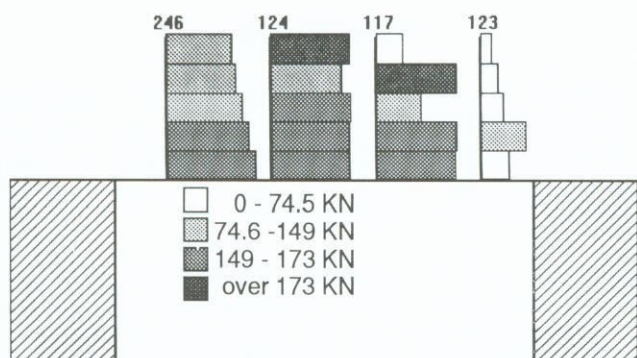


Figure 22.—Axial loads on test row 4 on 6/2/98

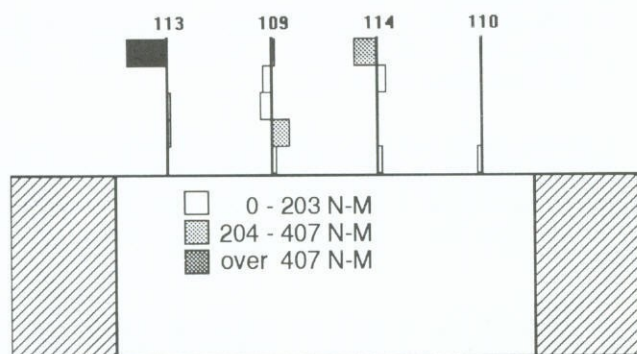


Figure 23.—Bending moments on test row 1 on 12/12/97

opposite gage 4 had an open reading. Six days later, the load level in bolt 114 dropped significantly, which may indicate that the bolt was beginning to break. On April 19, the last four gages were open, and loading increased in the adjacent bolts. Two days later, loading on adjacent bolt 109 dropped significantly and two open gages were recorded. Then on April 23, many of the strain gages had an open reading, which could have been caused by shear movement along a possible fracture plane (figure 13).

This sequence was repeated at test row 2, but only three time shots are shown. At this test site, the possible fracture plane was higher in the roof and affected only two instrumented bolts. A similar sequence was repeated at test row 3, but this time on the other side of the entry. In this test row, bolt 125 broke and was recovered on the mine floor. Test row 4 did not show any strain gage failures, and loading was more evenly distributed along the length of the bolts.

When a strain gage indicated an open reading, axial loading is not shown at that location because axial loading is an average of load on each side of the bolt. Open strain gages can be a result of either excessive loading or wire failure. Under axial loading, most lead wires maintain continuity until bolt failure. Recent shear tests on instrumented bolts show that lead wires are severed well in advance of bolt failure, and bending moments dissipate quickly from a shear joint. There are several indications that a bolt has broken: (1) high loads prior to failure, (2) loss of continuity to strain gages past the break, and (3) drop offs in

loading in the bolt section before the break. Using these criteria, the data indicate that progressive bolt failure may have taken place in three of the four test rows.

Only one instrumented bolt had a maximum strain less than 2,000 microstrain, the yield point of the steel. The maximum strain of all of the bolts was 43,000 microstrain. The average maximum strain of each test area is shown in table 4.

Table 4.—Strain gage information

Row	Ave. max. strain, microstrain	Percentage of open gages	Percentage of broken bolts
1	18,600	45	75
2	20,500	20	50
3	22,100	46	66
4	18,800	0	0

Microstrain is not a good measure of support effectiveness in each test area because so many of the bolts failed. A better measure is the percentage of strain gages and bolts that failed. The highest percentage of gage failures was in test row 1 and test row 3. The lowest percentage of gage failures was in test row 4.

When comparing load values among test areas it is important to remember that fully grouted cables have a higher ultimate strength than resin bolts. On the other hand, fully grouted cables have less ductility. Also, fully grouted cable bolts are larger in diameter than partly grouted cables and have a higher load capacity. The instrumented resin bolts had more cross-sectional area than the unslotted resin bolts. This means that the ordinary resin bolts probably failed before the instrumented bolts.

A typical plot of the bending moments is shown in figure 23. If shear forces caused by joint planes were the primary means of bolt failure, then the bending moments would be very localized and might not be detected because of the large spacing of the strain gages. Resin bolts will not fail under pure bending because of the high ductility of the steel. For this reason, axial loads are a better indicator of bolt failure.

Loading on the cable bolts in test rows 1 and 2 is shown in figure 24. Loading was fairly low until the abutment stress from the longwall panel approached the test areas, at which time the load increased rapidly and the pressure pads failed to work on all but cable 3. Several uninstrumented cables broke in both test areas along with instrumented cable 4.

The amount of deflection measured in test row 2 is shown in table 5. By December 12, 5 mm (0.197 in) of movement had taken place within the upper end of the bolted zone. This was enough movement to load a fully grouted bolt to the yield point of the steel (bolt 122 located next to the sagmeter hole). By June 2, 111 mm (4.37 in) of movement had occurred in this same zone, which was enough movement to cause the bolt to break. Very little movement was recorded in any other zones in this sagmeter hole.

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Table 5.—Amount of roof deflections in test row 2, millimeters

Distance from roofline, m	Date		
	12/16/97	4/19/98	6/2/98
3 to 4.3	1	4	3
1.8 to 3	0	2	1
0.9 to 1.8	5	13	111
0 to 0.9	0	2	-5

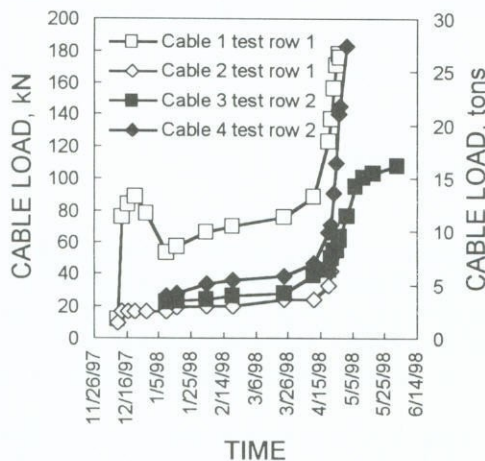


Figure 24.—Cable bolt loads

DISCUSSION

All data showed that enough movement was taking place within the bolted zone to load the supports past ultimate strength. Behavior of the strain gages indicated that shear forces added to axial loading caused bolt failure. The weak mudstone layer and the very strong siltstone-sandstone layer were significant factors because of their great difference in rock strength. By itself, the weight of the material was not enough to produce the bolt loads measured. However, weak material could dilate over the pillars and cause horizontal forces that would exceed the strength of the weaker material. This could have produced tensile failures in the rock that created shear forces on the supports.

Another possibility is that when the yield pillar deformed, the weaker mudstone developed shear cracks similar to those seen in short concrete beam failures. The true cause would be a combination or none of these factors. Observations in this area uncovered more problems at mid-pillar locations than at intersections. This could be caused by the dilation effect of yielding pillars. The wire mesh and concrete cans installed as secondary support were very effective, given the roof conditions.

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Automated Temporary Roof Support Systems: An Update

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ABSTRACT

It has been ten years since the Mine Safety and Health Administration (MSHA) promulgated a safety standard requiring the use of Automated Temporary Roof Support (ATRS) systems in underground coal mines. The use of ATRS systems with roof bolting machines and continuous miners equipped with integral bolters has all but eliminated fatal roof fall accidents associated with manually setting temporary roof support. The last ten years have seen an evolution of the ATRS systems themselves and of MSHA's policy regarding ATRS systems. These changes are a response to operational and safety issues regarding ATRS systems often brought about by the ever-changing coal mining environment. As a result, MSHA's Technical Support personnel receive numerous inquiries from all segments of the coal mining industry on various ATRS topics. These requests for technical assistance range from an interpretation of a facet of the ATRS standard to an in-depth evaluation of a prototype ATRS system. This paper will address the aforementioned evolution of ATRS systems and will also summarize the factors MSHA Technical Support uses to evaluate these systems in the field. In addition, some of the more common misconceptions regarding the design, use, and regulatory requirements regarding ATRS systems will be covered.

INTRODUCTION

Automated Temporary Roof Support (ATRS) systems were first conceived and introduced to the mining industry in the early 1970's. Although long recognized as one of the most important life-saving devices in coal mining, these simple devices are often taken for granted today.

The value of an ATRS system is apparent though when the details of a fatal roof control accident that occurred on March 1, 1998 are examined. A miner was fatally injured by a roof fall when manually installing temporary support. This accident occurred in a location where an overcast was being developed and the ATRS system on the roof bolting machine could not reach the mine roof due to the higher than normal height.

This accident serves as a reminder of the numerous accidents that routinely occurred in past

years as miners were manually setting and removing temporary supports.

It has now been ten years since the Mine Safety and Health Administration (MSHA) promulgated a safety standard requiring the use of ATRS systems on roof bolting machines in underground coal mines. Defined in the regulations simply as "a device to provide temporary roof support from a location where the equipment operator is protected from roof falls", ATRS systems have virtually eliminated accidents attributed to manually setting and removing temporary supports. Since the federal ATRS regulation was implemented in 1988, there has been a continuing evolution in systems themselves, as well as in MSHA's policy regarding ATRS systems. As a result, MSHA's Technical Support personnel receive numerous inquiries on various ATRS topics, ranging from requests for interpretation of some facet of the regulation, to requests for an in-depth evaluation of a prototype ATRS system. This paper will briefly review the aforementioned evolution of ATRS systems, and will also summarize the factors MSHA Technical Support uses to evaluate these systems. In addition, some common misconceptions regarding the design, use, and regulatory requirements regarding ATRS systems will be addressed.

ATRS SYSTEM EVOLUTION

Although the basic designs of a bar-type ATRS system or a safety-arm style ATRS unit (Figure 1) are essentially the same as when they were first introduced to the mining community, there have been some refinements to these systems as well as continued development of new systems. Over the years, ATRS bars have increased in length, as well as in the sophistication of their roof contact pads. While a ten-foot long ATRS bar was the maximum many years ago, it is certainly not uncommon to see bars that can extend up to 14 feet in length today. The roof contact pads have evolved from fixed circular pads, to pivoting rocker pads that can be manually re-positioned, to pads that can be re-positioned hydraulically, and to pads that operate in conjunction with an articulating bar (Figure 2). J.H. Fletcher and Company has also recently introduced an auxiliary roof contact pad that provides additional roof support in by the original ATRS bar. The lifting mechanism of the bars has also evolved from the

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simple T-style jack, to scissors style arrangements, to mechanisms that incorporate lemniscate linkages.

Safety-arm style ATRS systems have also changed somewhat over the years, from the dual arms joined with a ring-style contact device, to independent arms capped with rocker pads to better compensate for uneven roof conditions. Also, single safety arm ATRS systems have been implemented on narrow bolting machines designed specifically for longwall recovery roof support applications.

New style ATRS systems, primarily in the form of "jack"-type systems designed for retrofitting older machines, have emerged as a result of the 1988 federal ATRS regulations. These jack style systems have filled a necessary role in the niche of retrofitting many older machines, as well as machines used extensively in low coal.

ATRS EVALUATION FACTORS

In their early role as evaluators of ATRS systems to be accepted in lieu of drill station canopies, MSHA Technical Support developed approval evaluation factors. Many of these evaluation factors went on to become the basic tenets of the various state ATRS regulations promulgated in the early 1980's, and then the federal regulations in 1988. There are many factors that MSHA Technical Support personnel use to evaluate ATRS systems in the field. Although a majority of these factors are apparent in Part 75.209 of the regulations, some are not immediately obvious from a review of the ATRS standard. The following list of factors can be used to gain insight into the suitability of a specific ATRS system for use at a given mine. These considerations are a combination of regulations, policy, and Technical Support guidelines, and are followed by relevant comments.

ATRS/Positioning Controls

The controls that position and set the ATRS system shall be-- (I) Operable from under permanently supported roof; or (ii) Located in a compartment which includes a deck, that provides the equipment operator with overhead and lateral protection, and has the structural capacity to elastically support a deadweight load of at least 18,000 pounds. [75.209 (e) (2)]

In theory, the distance from the drill head to the ATRS controls is dictated by the roof control plan (the distance between bolt rows). In practice, most bolting machines have their ATRS controls at least five feet outby the drill head. If the ATRS and the positioning controls are not operable from under permanent support, the controls must be located in a suitable compartment which includes a deck. This compartment is not considered a cab, but must provide the equipment operator with overhead and lateral protection. The compartment must also elastically support a deadweight load of at least 18,000 pounds. (This differs from a "canopy" which allows the lesser of 15 psi over the plan view area or 18,000 pounds).

Except for the main tram controls, tram controls for positioning the equipment to set the ATRS system shall limit the speed of the equipment to a maximum of 80 feet-per-minute. [75.209 (e) (4)]

This requirement is necessary to ensure the safety of the machine operator as the machine is being trammed from a "walk-along" position.

MSHA has determined that certain controls on roof bolting machines are not subject to the requirements of 30 CFR 75.1710-1(a).... Controls that position and set the Automated Temporary Roof Support (ATRS) system (including "inch tram" controls) are not required to be located under a canopy. [MSHA Policy]

This provision was incorporated into the MSHA Program Policy Manual to clarify several controls not required to be equipped with overhead canopies or cabs.

Check Valves

All jacks affecting the capacity of the ATRS system and compartment shall have check valves or equivalent devices that will prevent rapid collapse in the event of a system failure. [75.209 (e) (3)]

This requirement appears straightforward, but still generates questions occasionally. First, it should be emphasized that the wording: "all jacks affecting the capacity" means that stab jacks that are part of the load transfer mechanism from mine floor to roof must also have a check valve. Secondly, it should be recognized that the check valve prevents rapid collapse of the ATRS and is not intended to prevent system bleed off. System bleed off which occurs due to leaking hydraulic cylinders, and/or the system settling into soft bottom, is another issue (a maintenance issue or a mining condition issue).

ATRS Load Requirements

The ATRS system shall elastically support a deadweight load measured in pounds of at least 450 times each square foot of roof intended to be supported, but in no case less than 11,250 pounds. [75.209 (e) (1)]

In examining this requirement, two points need to be addressed: 1) why 450 pounds for each square foot to be supported, and 2) what is "the roof intended to be supported"? First, the load requirement of 450 pounds for each square foot of roof is derived from data gathered by Technical Support personnel when working on the canopy regulations. Roof fall statistics for a ten year period revealed that about 70% of all roof falls were two feet thick or less. This was used in determining canopy load requirements, but when working on the requirements for ATRS systems to be accepted in place of canopies the decision was made to go to the next benchmark of roof falls, three feet thick or less. Using this three foot thickness of rock, and an assumed density of 150 pounds per cubic foot, 450 pounds per square foot of roof is determined.

In looking at the second question of the "area intended to be supported" one of the first steps is to determine how many roof bolts are being installed from one ATRS setting. If only one bolt is being installed, then the area to be supported is 5 feet by 5 feet, derived from the maximum allowable support spacing. If more than one bolt is being installed, then the physical dimensions of the ATRS must be evaluated. An imaginary support envelope is constructed around the ATRS bar to determine the area to be supported. Essentially, the length of this support envelope is the length of the bar plus five feet, while the width of the support envelope is simply five feet (regardless of the actual width of the ATRS roof contact pads). Whenever discussing this imaginary support envelope

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concept it must be emphasized that this is only for theoretical purposes in determining the required load capacity of the system, and under most circumstances the area inby the ATRS system should not be considered safe in which to travel. The only exception is when the ATRS system is set within five feet of the face.

ATRS Certification Issues

The support capacity of each ATRS system and the structural capacity of each compartment shall be certified by a registered engineer as meeting the applicable requirements of paragraphs (e)(1) and (e)(2) of Section 75.209. The certification shall be made available to an authorized representative of the Secretary and representative of the miners. [75.209 (f)]

The ATRS certification should be consistent with regard to the specified load requirement, and the contents of the letter should identify the subject machine. The certification should also account for any modifications to the original design.

A common misunderstanding regarding this requirement is the reference to "all applicable requirements of paragraphs (e)(1) and (e)(2)". The applicable requirement of paragraph (e)(2) is referring to part(ii), concerning certification of the compartment's load capacity. This provision does not require that a bolting machine's control location be certified.

Evaluation Factors for Bar-Type ATRS Systems

The following six evaluation factors apply to a bar-type ATRS system (as shown in Figure 3). Some of these factors relate to mine specific conditions.

The distance between the rocker pads should be five feet or less (dimension B on Figure 3). [Technical Support guideline]

In the majority of instances this parameter is built into the systems. Occasionally an older style machine or one with mismatched components from another system is found where this five foot spacing is exceeded.

The length of the bar should be compatible with all planned mining widths. [Technical Support guideline]

The length of the ATRS bar should not just be considered for the "typical" entry width, but rather for all planned mining widths, such as in combined belt and track entries or longwall setup entries.

When an ATRS system is used, the maximum distance that an ATRS system is to be set beyond the last row of permanent support shall be included in the roof control plan. [75.221 (6)]

Not only should this distance be specified in the roof control plan, but also the distance that is specified should be reasonable, typically the bolt row spacing plus 12 inches. In some cases on certain mast feed style or older machines, it is necessary to increase this distance to the bolt spacing plus 18 inches.

The ATRS system should reach the mine roof in all anticipated mining heights, including instances where draw rock is routinely mined or when bolting overcasts. [Technical Support guideline]

Again, the "typical" mined height is not the only consideration when evaluating the necessary extension capability of the ATRS system. While recognizing that not all conditions can be planned for, for example, roof falls, this should not minimize the need to plan for the normal, anticipated, higher-than-typical mined heights. This might mean equipping one bolter with a higher-reach ATRS system as the dedicated high-place bolter. An additional point to stress here is that if extensions are going to be used to reach these high places, they must also be certified.

The boom jack/canopy post height (designated D in figure 3) should be compatible with the mine height. [Technical Support guideline]

In deciding the suitability of a certain machine for a specific mined height, consideration should also be given to the drill station canopy extension capability (or boom mast safety jack if an ATRS in-lieu-of-canopy approval has been granted). The canopy should extend to such a height so as to not confine the bolter operator to an awkward and unsafe position.

If the ATRS is approved in-lieu-of-canopy, the boom jack must be pressurized against the mine roof. [Technical Support guideline]

Although not desirable if a drill station canopy is in place, pressurizing the boom mast safety jack against the mine roof is required as a condition of the ATRS in-lieu-of-canopy approval. By pressurizing this pad against the mine roof, the unsupported roof span between the last row of permanent supports and the ATRS bar is reduced.

The bolting sequence when installing 5 bolts per row should be compatible with the ATRS bar capability. [Technical Support guideline]

When installing 5 bolts per row with a bar-type ATRS system, special consideration should be given to the bolting sequence. Often a five bolt pattern requires a bolting machine to be positioned closer to one rib. Typically this results in a distance from the ATRS bar to the other rib in excess of 5 feet.

Evaluation Factors for Single-boom, Squirmer-type Machines

The following four evaluation factors apply to single-boom, squirmer-type roof bolting machines:

The drilling controls should be streamlined and located a minimum of 30 inches outby the drill head (dimension B on Figure 4). [Technical Support guideline]

This provision is probably the most important single factor in ensuring adequate ATRS protection for the operator of a squirmer-type bolting machine. By streamlining and relocating the drilling controls a minimum of 30 inches outby the drill head centerline, a safe work position outby unsupported roof is provided. Without this provision, the angled positions of the squirmer bolter for several of the bolt installations in the bolt row may force the operator inby supported roof.

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The ATRS pads should extend inby the drill head a minimum of 12-15 inches (dimensions A on Figure 4). [Technical Support guideline]

By extending the ATRS pad inby the drill head center line 12-15 inches, a "break line" is established inby the bolter operator. If the bolting machine possesses drill boom sump capability, this measurement should be taken with the drill head in the center position of any front to back sump distance.

The ATRS pads should extend toward the operator and toward the "off-side" at least 12 inches. [Technical Support guideline]

By extending an ATRS pad toward the off-side of the bolting machine, a "break line" is established away from the bolter operator. Extending an ATRS pad toward the operator results in better support in the area of the operating controls.

The roof bolter operator should not go inby the row of bolts being installed. [Technical Support guideline]

By adhering to the stipulation of the operator not going inby the row of bolts being installed (instead of simply not going inby temporary support), an extra measure of safety is provided. Once again, this is especially relevant when examining the angled machine position of the squirmer style machine in relation to bolting pattern and sequence.

While the previous four evaluation factors were originally developed for evaluating ATRS systems for in-lieu-of-canopy approvals, they can be applied to almost any single-boom squirmer-type roof bolting machine. However, some squirmer roof bolting machines operate in extremely low mining heights, 20-30 inches, that can impose limitations in addressing these factors.

ATRS SYSTEM MISCONCEPTIONS

Despite their long-time presence in coal mines, there are still several common misconceptions about ATRS systems and their use that seem to persist. Some of the more common ones are:

- *If a roof bolting machine does not need a canopy, then it does not need an ATRS system.*

ATRS and canopy requirements differ in two important aspects: the mined height in which they are required and the area of the mine where they are required. First, ATRS systems are required on all bolting machines operated in mining heights of 30 inches or more. The use of systems in mining heights below 30 inches is addressed in the roof control plan. (MSHA District Managers routinely require the use of ATRS systems in mining heights below 30 inches). Canopies, on the other hand, are required on all self-propelled face equipment where the bottom to top (mine floor to mine roof) measurements are 42 inches or greater. (Guidelines for this determination are provided in MSHA's Program Policy Manual, Part 75.1710-1).

In addition to different height requirements for ATRS systems and canopies, there are also differences in the requirements regarding the areas in a mine where use of these safety devices is mandatory. The ATRS regulations state that ATRS

systems shall be used with roof bolting machines operated in a working section, while the canopy regulations are commonly interpreted as requiring canopies on self-propelled face equipment operated in or inby the last open crosscut.

- *An ATRS system is not needed when installing supplemental roof support in areas where primary permanent roof support is already in place.*

Regarding this exception to the required use of an ATRS system, MSHA's Program Policy Manual only addresses the installation of truss supports and does not include other types of supplemental support.

- *The maximum distance that an ATRS system can be set beyond the last row of permanent support is five feet.*

This distance is specified in the approved roof control plan and will vary depending upon the particular roof bolting machine/ATRS combination used. Usually this distance is the distance between bolt rows, plus 12 inches.

- *An ATRS system should be set near its rated load capacity.*

The setting force of an ATRS system should be no more than a few thousand pounds.

- *The controls that position and set the ATRS system must be operable from under permanent support.*

Some machines have ATRS controls in a location where the operator will be inby support when they are activated. In this case, the controls must be located in a compartment (with a deck and some lateral protection).

- *An ATRS system must have a certification tag or label.*

Unlike canopies, ATRS systems are not required to have a tag or label as evidence of certification. The only time an ATRS system is required to have an attached certification tag is if the system has been approved in lieu of a drill station canopy. The reason a certification tag is required in this instance is that the ATRS system has been accepted in place of a canopy which requires an attached tag.

- *Inch tram controls and controls that set the ATRS system must be located under a canopy.*

As stated in the MSHA Program Policy Manual, these controls are not required to be under a canopy.

SUMMARY

When discussing the topic of ATRS systems one aspect becomes readily apparent. Since their introduction to the mining industry approximately 25 years ago much has changed. The much-improved ATRS systems of today are the result of a continual evolution which did not attempt to alter the basic concept - a safer means to provide temporary roof support. This evolution focused on refinements to existing ATRS systems and introduction of new systems to expand ATRS usage. Another change that affected ATRS systems was the introduction of

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federal regulations requiring their use. This paper has been an attempt to summarize this evolution and to provide a resource for those with questions concerning the design, use, evaluation factors, and federal regulatory requirements regarding ATRS systems.

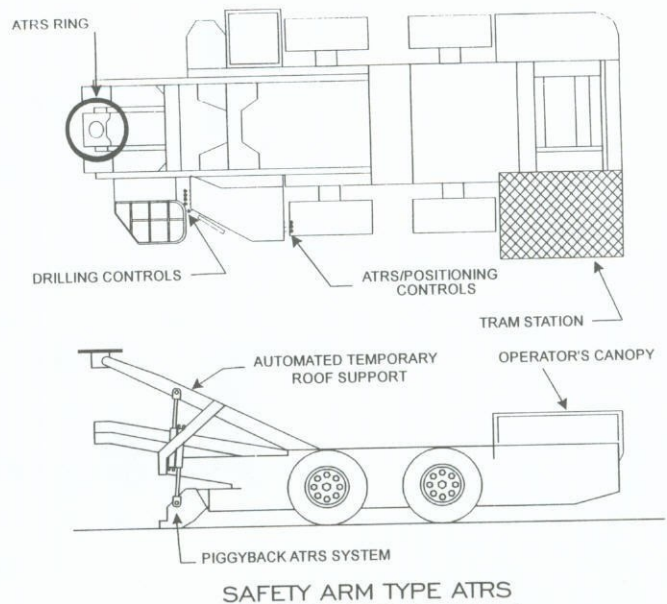


FIGURE 1

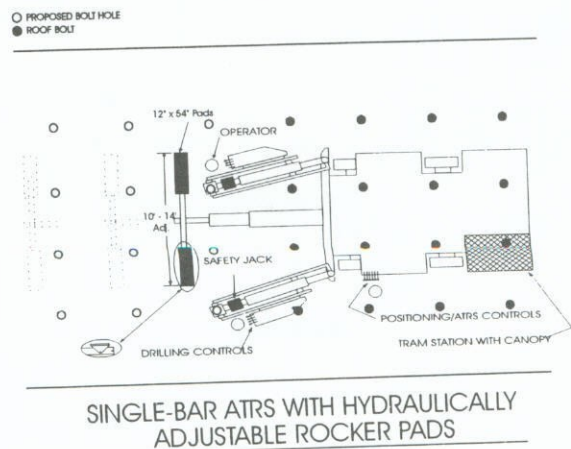
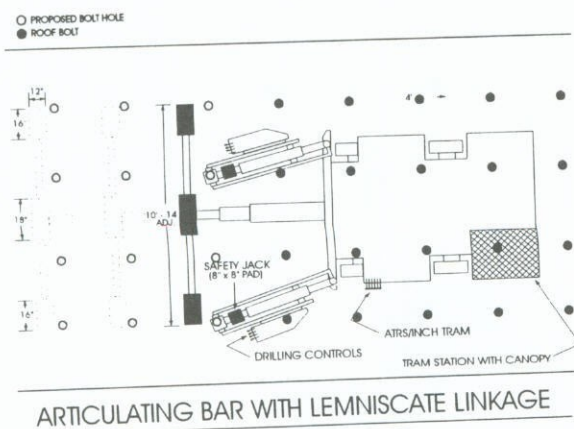
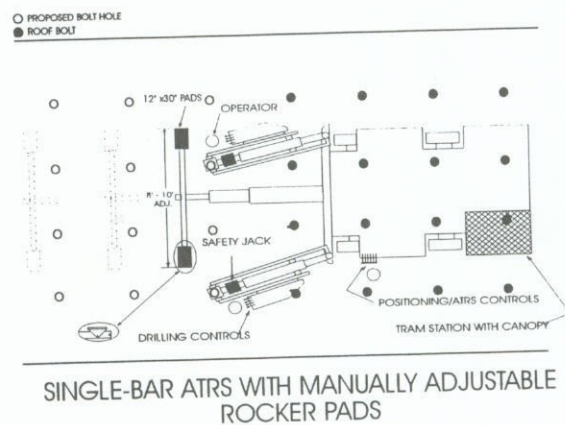
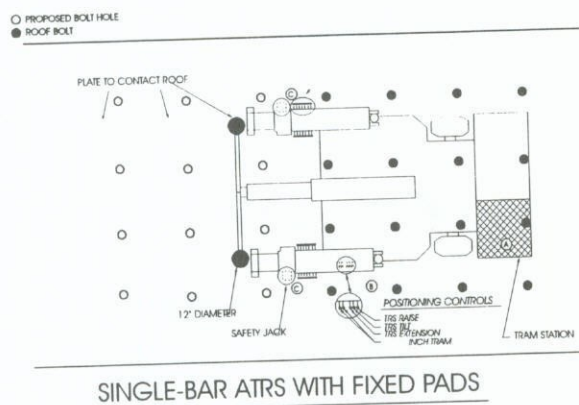


FIGURE 2

X Proposed Bolt Hole
O Roof Bolt

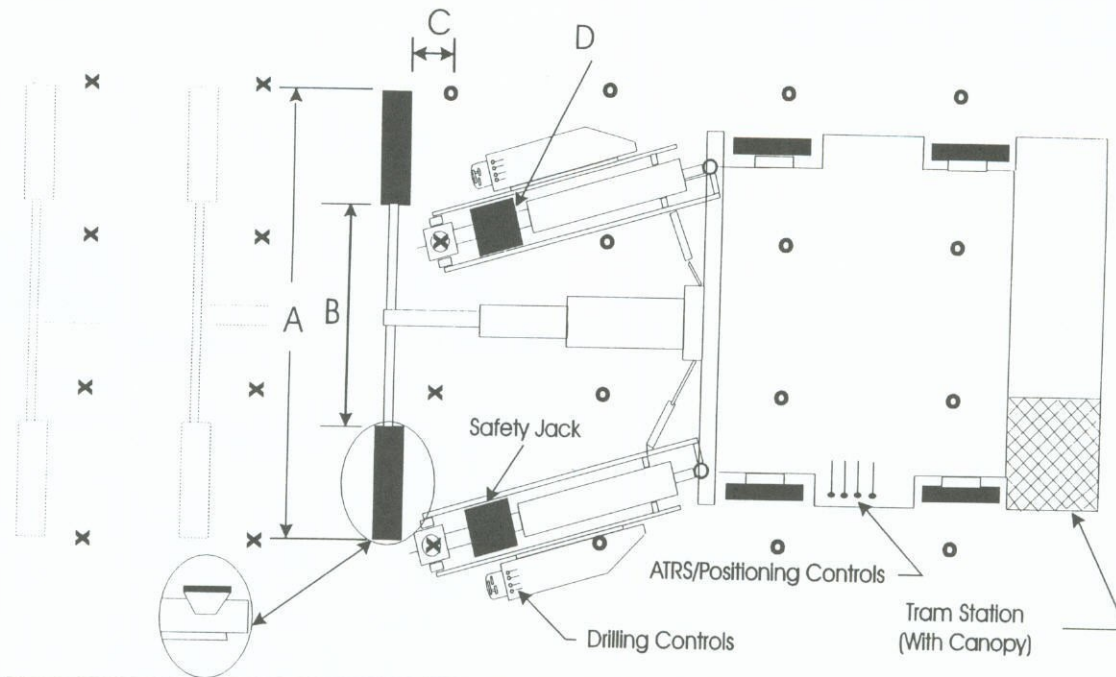


FIGURE 3

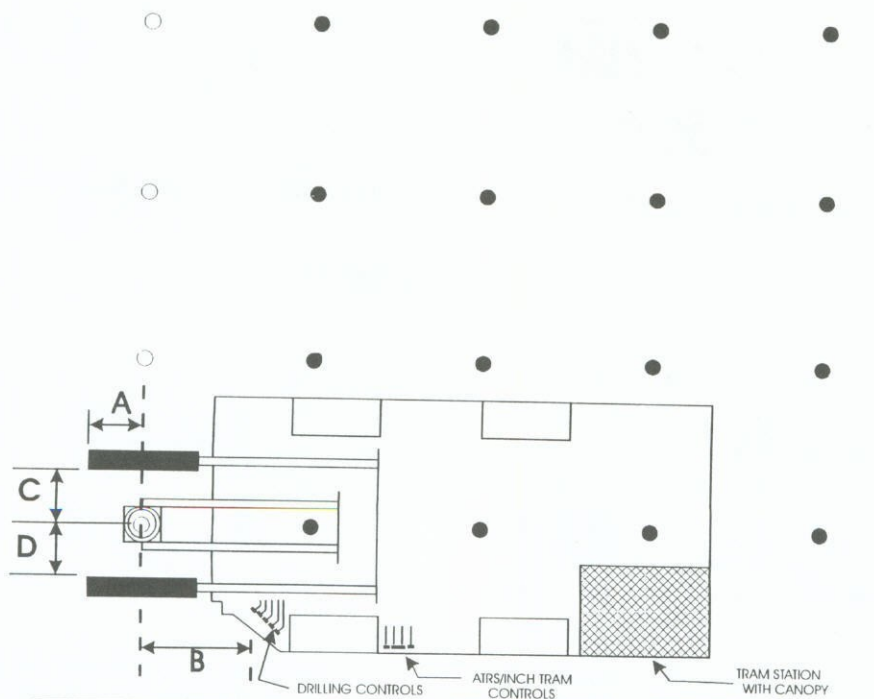


FIGURE 4



Safety and Productivity Innovations in Mechanized Bolting

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ABSTRACT

Underground mine bolting machines have evolved since inception in the 1940's due to the need for increased safety and productivity. However, major design innovations historically came from changes in the mining plan and the mine environment. During the last ten years, productivity needs have outpaced the evolution of existing designs. J. H. Fletcher & Co. has responded with a commitment to research and development which has fostered several new designs. The most recent machine built is a walk through crawler frame powering four drilling units simultaneously with semiautomatic computer control using two operators. This machine utilizes technology to create a safer yet more productive roofbolter for continuous mining sections utilizing place changing methods.

INTRODUCTION

Safety and productivity have been tied together in the design and development of roof bolters since inception. Roof bolting first began to gain acceptance as a means of roof support around 1950. It was win-win-win technology, which spread rapidly. It lowered the cost of roof support by using four bolts instead of a cross bar timber or rail with timber legs, increased productivity by providing a room without timbers encroaching into the mobile equipment travelways and increased safety since there were no timbers to handle, set, and reset. J. H. Fletcher & Co.'s new four head semiautomatic roof bolter also has win-win-win characteristics. It is more productive installing roof bolts, provides a drilling system where the operator is removed from rotating components and can reduce drill bit and steel cost with closed loop computer control. The progression in design of past machines helps us to understand the many advantages of this new roofbolter.

The first mechanized bolters were auger drills mounted on existing equipment (we have some customers who still want us to do this). This greatly reduced labor and fatigue from using hand held drills. Next, a dust collection system was developed to improve the operator's environment. The additional dust

collection system, the full time need for roof bolters, and the constraints presented by various types of underground mine environment soon led to a mobile machine specifically designed for roof bolting. J. H. Fletcher & Co. developed "through the drill steel" dust collection and even manufactured the hollow drill rods for a period of time since there was not a ready source for this product. This was a much more effective dust control system, which has not changed in principle to this day. Dry vacuum drilling provides for a better operator environment and, by not adding water may prevent the rapid deterioration of some types of roof and floor. Water mixed with drill cuttings also creates an abrasive slurry which accelerates the wear of machine components.

By 1960, through the drill rod dust collecting had been adopted by all manufacturers and the new focus was on rapid repositioning of the drill unit to reduce the non-drilling time spent in the place. Drills where the mast could slide from one bolt position to another such as the model DB and drills with two sliding heads like the DDJ which could bolt a complete row without repositioning the chassis were developed. The dual drill concept became popular quickly in mines required to have two operators on a machine due to their labor contract. The Model DM and DDM machines took drill repositioning to the extreme. With up to nine feet of boom extend, the dual drill DDM could bolt a 20-foot cut from only two chassis positions. Even a four drill version of the laterally sliding "hydro-slide" machine and four drill units combining two drills on extending booms and two on lateral slides were produced by the mid 60's. The four drill unit concept was not pursued at the time due to the exposed position of the drill operators between the machine and the face and the close proximity of operators to the powerful drilling mechanisms.

Injury from falling rock had been the leading cause of accidents in United States Coal mines for decades and these injuries were particularly common to roof bolters due to the definition of their task. The late 60's and early 70's saw the development of integral roof support and overhead protection on roof bolters. Hydraulic cylinders which went to the roof adjacent to the operator (safety post) were installed and plates

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(canopies) were added to help shield the operator from smaller pieces. Temporary roof support "TRS" systems were added to the machine in by the drills to prevent larger roof falls from initiating near or riding into the operators area. In 1973, federal regulations required overhead canopy protection. This was phased in over several years from high to low mining heights. The safety advantages of canopies almost always outweigh any disadvantage the canopy may present. Today, Fletcher will not build or ship any standard roof bolters without a drill operator's canopy. Similarly, the remote controlled temporary roof support system mounted to the in by end of the drill and controlled from under permanently supported roof eliminates the roof bolter operator's need to work or travel in by supported roof. West Virginia first enacted remote controlled or "automated" TRS regulations followed by other states and federal requirements. The enhancement of TRS systems and canopies have been continuous since inception but these alone are not always enough.

Due to potential brow and rib falls, particularly in higher mines, efforts concentrated on positioning the operator to reduce exposure. The drill mechanism and boom can offer protection when the operator is positioned on opposite side of these components from the adjacent rib. Versions of almost every model dual boom bolters were developed with drilling controls between the booms. High seams present the problem of providing a means for the operator to safely drill and install roof support without standing on the floor. The model HDDR with its lifting man platform, inside controls, TRS and canopies helped to keep the operator in as safe a position as possible, by putting the man in a basket on the boom, these machines were as safe as remotely controlled drills, but with higher availability and more versatility. In 1983, a customer having severe rib roll problems requested J. H. Fletcher & Co. to develop a roof bolter which would provide passage from the rear (outby end) to the operator's station on the inside. The purpose was to avoid risk of traveling between the side of the machine and the rib. This "walk-thru" concept rapidly gained acceptance and has been designed into all higher seam dual boom machines. The most familiar being the Model HDDR. The latest version of this drill offers a wider walkway and rear ramp to facilitate loading materials, ergonomically designed operators platform, and safety devices like 2-hand fast feed and hydraulic disconnect control.

Machines with lifting platforms were also developed for high seam applications particularly in the salt and limestone industries but they too, like the early coal bolters, had to be repositioned for each bolt installation or pair of bolts for dual drill machines. Some non-coal mines with long distances between working faces wanted the productivity gains that a drill with a swinging and extending boom offers on a higher speed chassis. In these conditions, roof support material flexibility was not as necessary, so a remote controlled drill was the logical answer. However, this did require the development of a dependable resin inserter. The first ARRD was developed in 1988 to answer this need. A roof referenced drill system to maintain the resin and bolt systems in position relative to the drilled hole was developed. This stabilized the bolter's position

relative to the drilled hole for resin and bolt insertion.

With the success of the ARR-D bolter, it was recognized that the dual boom HDDR could be fitted with remote control drills capable of installing resin bolts and one man could feasibly bolt a place faster than using a conventional HDDR without being in close proximity of the drill and bolt mechanism. A joint development contract to pursue this concept was undertaken between J. H. Fletcher & Co. and Marrowbone Development Co., then owned by Shell Corporation. Due to having one operator controlling more than one drill unit, a PLC was employed to automate several of the functions. This machine was built and tested in 1989 but never made it to a production section before the contract ended. Several valuable lessons were learned from the ARRD and this drill regarding remote control and automation.

We decided that there was tremendous potential for computer controlled drilling on the "Marrowbone" drill and we needed to do more development in this area. The auto bolter module was designed in 1992 to create a computer controlled drilling and bolting unit that could be fitted to different machine platforms depending on the mine type and conditions. The module was tested at A. T. Massey's Martin County Coal Mine in Kentucky and FMC's Trona Mine in Wyoming. Foreseeable uses included not only the HDDR platform like the Marrowbone drill for place changing, but also on miners, surge cars, continuous haulage, and on the diesel articulated chassis for non-coal applications. The auto bolter module demonstrated the feasibility of such a unit and provided exciting drill performance. The module would drill in fairly hard sandstone at rates as high as 17' per minute without breaking 7/8" diameter drill steels or burning up drill bits. In fact, drill bit life was improved over conventional operator controlled HDDR roofbolters. This brings us to our four head semiautomatic computer controlled bolter now undergoing test underground.

The new machine's basic concept was first outlined in February 1997. It had to be safer and more productive than the current Model HDDR. No configuration of two drill units could provide the productivity required, therefore, it became necessary to revisit running four drills in parallel, but preferably with only one or two operators. We knew we could not use conventional drill systems with control position detents since these have proven to be so dangerous that all roofbolter manufacturer's voluntarily agreed to stop using detent controls in the late 1970's. All detent drilling controls were recalled in 1985. We turned to the automated unit as a way to drill the hole without the operator handling the drill rod. We positioned the controls to keep the operator close enough to the drills so as to give him good visibility to locate the drill units yet far enough away to be beyond incidental contact. With the automated drill system on board we next considered the question of installing the roof support hardware. The ARR-D had carosel and rack systems for carrying the hardware but these took more time to load than the bolter operators have available in highly productive mining systems. The automated drill provided time for the operator while it was drilling unattended. This time was sufficient for an operator to prepare

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bolts and resin for two holes or more depending upon drilling conditions. Therefore the machine is designed to have a storage device for each drill which will hold a drill rod, a resin cartridge, and a bolt. In operation the drill rod is handled totally by the machine and the operator loads a resin cartridge and bolt for each unit while they are drilling, optimizing the use of the operator's time and improving the productivity potential of the machine.

With increased safety and productivity without sacrificing availability as the four head project goals we set off to develop reliable systems and hardware. The lessons learned from the ARRD, the Marrowbone project as well as the remote drill module project were applied to the newly designed four head bolter.

Recent innovations on production machines also greatly shaped the four head bolter. The machine uses a center walk way and a powered rear ramp. These features provide the same safety and convenience as found on a HDDR. The machine has bolt on crawler assemblies similar to those on production machines. The ATRS resembles the production "T" style ATRS. The hydraulic system uses variable volume piston pumps similar to those found on the Mobile Roof Supports.

The four head bolter uses modular construction allowing easier replacement of machine components. The drill rigs are constructed from sub-assemblies that bolt together. The roof reference drill mast has a machined pad that accept the mount for a manipulator arm assembly and a material carousel. Two drill rigs are fitted to a "T" style ATRS. The two "T" style ATRS's hold the four drill rigs solidly against the mine top. This reduces the tendency for the drill rig to move relative to the mine top during high thrust. The two "T" style ATRS have answered the flex problem found on the original remote drill project. In the event that these assemblies are damaged they can be replaced with available replacement assemblies. Individual parts in the assemblies can also be replaced on a part basis.

The machine uses the manipulator arm assemble to position and control the drill steel, the resin delivery system, and the bolt. The manipulator arm assemble is made up of an upper and lower centerlizer. The upper centerlizer acts as a guide while the lower centerlizer acts as both a guide and clamp. The manipulator arms move the drill steel, resin delivery system and bolt from the material carousel to the drill chuck. The manipulator arms also return the drill steel and resin delivery system to the material carousel. In this way the drilling materials can be clamped to allow accurate positioning and then guided to provide improved control. This is accomplished with the operator never touching any drilling material while it is in the drill motor. The hardware used in these assemblies is proving to be reliable but will certainly continue to evolve.

The machine has a total of 180 HP. Two dual piston pumps provide the hydraulic power; each set driven with 75 HP. Horsepower consumption is a concern during only part of the bolting cycle and potentially during severe tramming situations.

The 75 HP motors are providing excellent tram speed (120 f/m) for the 70,000 lb. Machine. To eliminate heat and overload concerns the pumps are also fitted with a load sense and horse power limiting control. The four vacuum blowers are powered by two 15 HP electric motors. Each 3600 RPM dual shaft motor accepts two direct coupled blowers. The blowers motors are individually controlled and only operate during drilling. The blower on demand reduces the operators exposure to noise and lowers the over all machine horsepower draw.

To this point the discussion has targeted physical hardware and touched on some of the operators environment features. The next step is to consider the operation of the machine. The four drill rigs are run by two operators. Each operator has the responsibility of loading the material carousels on his or her respective side of the machine. Material loading takes place as the hole is being drilled. Since all four holes are being drilled at once and material handling is in process during drilling, there is significant potential to reduce the time required to bolt a place. Additionally the resin delivery and bolt installation cycles can run concurrently on all four drills, further increasing productivity.

Bolting consistency is also addressed. The machine will repeat the same feed rate, spin or mix time, and hold or thrust time on each bolt in accordance with the resin manufactures guidelines.

The machine is obviously not controlled by the traditional feed, rotation, and fast feed manual valves. The operator has a control station with toggle switches, rocker switches and selector switches. A given switch may initiate various solenoids in accordance to the program loaded into the PLC. Various sensors feed information back to the PLC which controls the out puts to the solenoid valves. These are not detented controls but are controls sensitive to feed forces, feed rates, drill torque, drill RPM, drill position and vacuum conditions. Monitoring and reacting to various parameters, having two centerlizers per drill motor, and using the material carousel as a physical barrier shielding the operator from the drill string allows the operators to turn attentions to material handling. The operator needs to initiate the three parts of the bolting cycle. A toggle switch with an enable switch are activated to initiate drilling, or resin insertion, or bolt insertion.

To date some worthy benchmarks have been reached. The machine has drilled, and bolted holes in under two minute cycles. This includes a drill time of less than 30 seconds for a five foot hole and a 15 second hold time for the resin. The machine does install each bolt the same providing improved consistency. The solenoid test function and on board diagnostics have proven themselves. The bit life looks good but shows more potential for improvement. Using the PLC to record drill parameters and using this information to improve the drill program is proving to be practical and efficient. This feature was first used on the remote drill project. Another application of the recording feature is to "look at" what is being drilled. The changing drilling parameters allow estimating roof strata.

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What's next? Each of these items as well as the entire machine will continue to evolve in accordance with the coal industry. The mine height is a critical limitation of the current design. The drill steel length plus approximately 20" is required to use the drill module. The possibility of multiple segmented drill rods will ease mine height vs. bolt length considerations. The shape and structure of the machine will be changed to adapt to varying mining conditions. Continued hazard analysis and evolution will provide information to further improve safety and productivity.



Factors Influencing Intersection Stability in U.S. Coal Mines

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ABSTRACT

Groundfalls are much more likely to occur in coal mine intersections than in entries. NIOSH is using the experience of U.S. coal mines to determine the factors which influence intersection instability and provide guidelines for the safe excavation and support of intersections. Detailed field investigations have resulted in a database of U.S. coal mines containing 12 mines and 639 roof falls so far. By using the roof fall rate as the outcome variable, correlations between roof geology (CMRR), intersection span, and roof support have been established. Case studies have indicated that replacing 3-way intersections with 4-way intersections may not reduce the total number of roof falls. Additionally, the size of intersection spans tend to decrease with lower (weaker) CMRR. Protocols have been established for the collections of roof bolt parameters. The performance of individual roof bolts can now be tracked with roof fall rate.

BACKGROUND

In underground coal mines, tens of thousands of intersections are driven each year. Intersections represent exposed ground which can span 25-50 ft, well over the normal width of an entry. When pillar corners are rounded to make travel easier, and when pillar spall (especially in high seams) exposes or undermines more roof, the larger spans can easily become hazardous. In 1996, there were a total of 2,824 unplanned roof falls from 892 underground coal mines. These falls resulted in 711 injuries and 8 fatalities. Of the roof falls with known locations, approximately 71% occurred in intersections. Considering that intersections account for only 20-25 % of the total drilage, on a foot by foot basis, roof falls are 8-10 times more likely to occur in an intersection than an equivalent length of entry. An estimate of number of intersections driven per year reveals the exposure hazard to miners. At a rate of 1,500 intersections driven per million tons mined and 230 million

development tons in 1995, approximately 350,000 intersections are driven per year in U.S. coal mines.

INTRODUCTION

The increased likelihood of failure in intersections has been documented in the past. These studies have primarily focused on specific localized problems leading to failure, including horizontal stress and geologic discontinuities (transition zones related to paleochannels, faults) (1,2,3). Numerical modeling studies have confirmed that intersections are less stable than entries, and that 4-way intersections are less stable than 3-way intersections. Intersection instability was found to be dependent on rock quality and the ratio of horizontal stress to vertical stress (4,5). Considering that intersections are at much more risk of failure than entries, it is surprising that more attention is not given to engineering special support for intersections. Even mines with demonstrated intersection problems often do not install more support during development for these areas, preferring to rely on supplemental support when problems occur. Other secondary factors contributing to intersection failure include the quality of bolt installation (long unsupported times before bolting, oversized holes, fingergloving, etc.), and turnout frequency and location (2,6).

Oversized intersection span has been identified as a cause of failure (6,7,8). Suggestions for controlling overspanned intersections and turnouts include longer bolts in the intersection corners near the ribline (8), limiting turnouts to corners not at critical angles to the principal horizontal stress direction, and sequencing crosscuts to avoid turnouts altogether.

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For any given geology, there are spans which will induce entry failure. The key is defining the allowable span based on roof strength and roof reinforcement, and not to exceed it. Individual roof control plans specify maximum spans which can vary considerably between and within MSHA districts. Some roof control plans limit the sum-of-the-diagonals to 60-70 ft (Figure 1), specify the number, location, and size of turnouts in the intersection, or restrict turnouts to specific entries.

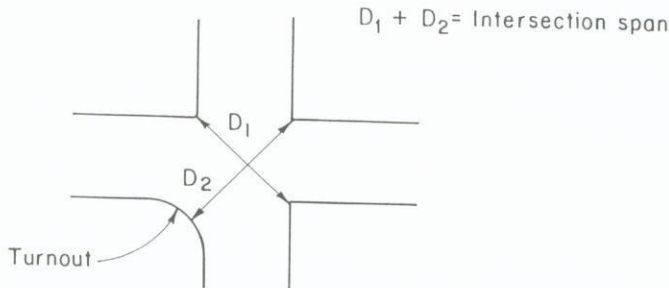


Figure 1. Sum-of-the diagonals intersection span measurement.

Rock Load and Roof Stress

Intersections may easily become overspanned if care is not taken when mining. When the roof rock is strong, small increases in span probably will not compromise the stability, but where the roof is weak, small increases in span can add significant load to the bolted interval above the intersection. If the geology is uniform, the rock-load height can be expected to be proportional to the span and to the roof rock quality, according to the equation proposed by Unal (9):

$$h_i = \left(\frac{100 - \text{RMR}}{100} \right) w_e \quad (1)$$

where h_i = Rock-load height, ft;
RMR = Rock Mass Rating, which is equivalent to the CMRR (15)
 w_e = Entry width, ft.

This equation implies that the rock-load height above an intersection of competent roof, with an CMRR near 100, would be very small, while an intersection with incompetent roof (CMRR near 0) would have a rock-load height approximately equal to the span. Assuming that K can be substituted for the geologic adjustment factor, $(100 - \text{CMRR} / 100)$, equation 1 can be rewritten as:

$$h_i = (K) w_e \quad (2)$$

thus the rock-load height is proportional to the span.

Assuming that the rock load above the intersection can be represented as a pyramid (fig. 2), the volume of rock and rock load over the intersection can be estimated by the equation:

$$R_L = V_i \gamma = \frac{(w_e)(w_c)(h_i)}{3} \gamma \quad (3)$$

where R_L = Rock load above intersection, lb
 V_i = Volume of rock above intersection, ft³;
 γ = Unit weight of rock, pcf;
 w_e = Entry width, ft; and
 w_c = Crosscut width, ft.

Assuming $w_e = w_c$ in most instances, then the rock load is proportional to the cube of the span, as shown in equation (4) where w_e and $K(w_e)$ have been substituted for w_e and h_i , respectively. If it is assumed that the rock load above an intersection can be represented by a cube (instead of a pyramid), the load is still proportional to the cube of w_e equation (5).

$$R_L = \frac{K(w_e)^3}{3} \gamma \quad (4)$$

$$R_L = K (w_e)^3 \gamma \quad (5)$$

Judging from the shape of roof fall cavities, the truth usually lies somewhere in between these two idealizations. In either

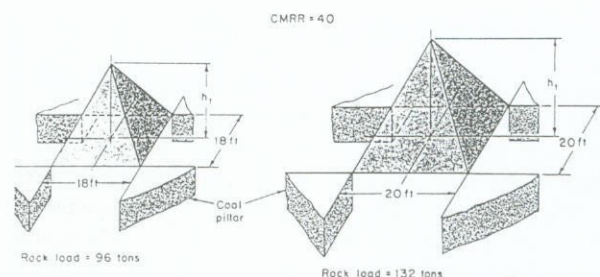


Figure 2. Comparison of rock loads above the intersection.

case, a small increase in entry width (and therefore increase in intersection span) results in a significant increase in the rock load. As figure 2 shows, increasing entry span by just two ft can increase the intersection load by nearly 40%.

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Where the geology is not uniform, and roof falls are truncated by an overlying self-supporting strong bed, the rock load height may be constant regardless of the intersection span. In this case, the rock load increases in proportion with the square of the span.

Roof Monitoring

Roof strain monitoring has been employed to predict imminent intersection failure. Roof monitoring and the use of early warning devices to predict intersection stability is not a widespread practice in U.S. coal mines. Roof monitoring programs have been implemented at a few mines, with generally successful results. One example was the convergence monitoring at over 600 stations in the Virginia mines of Island Creek Coal Company in the 1970's (10). It was found that closure rates in excess of 0.1 in/d indicate instability.

The monitoring program was thought to have prevented "numerous" falls while it was in operation (11). Maleki and McVey (12) monitored roof sag at 139 stations in 2 western U.S. coal mines. At one mine, sag rates less than 0.01 in/d denoted stability, while sag rates of 0.025 in/d was the critical sag rate that usually indicated a roof fall would occur within 8 days. At the other mine, 0.032 in/d was found to be the critical sag rate.

The use of early warning devices is a requirement in UK coal mines. Falls of ground have been dramatically reduced in British coal mines since the advent of roof bolting and monitoring in 1991 (13). "Action levels" of roof strain are established by roof instrumentation, and monitors are placed at maximum intervals of 65 ft (20m) of entry. Inby monitors are read every shift and recordings are made on a weekly or monthly basis. Any movement exceeding 1 in. signals the need for some remedial action, generally the installation of supplemental support. This "action level" will vary with geology and support and should be customized to each mine roof situation. These monitors cost \$25-100 depending on numbers of anchors and ability to be attached to data recording systems.

NIOSH INTERSECTION FAILURE PROJECT

The NIOSH office of Mine Safety and Health at Pittsburgh Research Laboratory has an ongoing project to study the factors which affect the stability of intersections. Once these factors have been identified and evaluated, the goal is to estimate the probability of failure in intersections and provide engineering guidelines for safe design and support of intersections.

The approach is to use roof stability experience of U.S. coal mines in order to determine allowable intersection spans based on geology, and to prescribe support for that span.

In order to compare the relative frequency of roof falls in coal mines, a roof fall rate was calculated for all U.S. underground coal mines using data from 1996 (14). Only non-injury roof falls reported to MSHA were considered in the analysis. The roof fall rate in each of the mines was calculated based on the estimated amount of entry driveage. The entry driveage was estimated from the reported production, adjusted for seam height, and whether the mine employed a longwall. The actual seam height was used in the conversion. As an example, in two mines with an equal number of roof falls and equal production, the mine with the higher seam would have a higher roof fall rate because it would have less driveage and less exposed roof. Development tons in longwall mines were estimated to be 1/4 of total annual tons. Therefore, the fall rate for longwall mines was multiplied by 4. Figure 3 shows the distribution of mines by total roof fall rate. Fifty-six percent of the mines reported no roof falls in 1996. Seventy-five percent of the mines had fall rates of .3 falls/10,000 ft of driveage or less (almost all of these mines had <10 falls in 1996). 48 mines reported 10 falls or more in 1996. While most of the mines had low fall rates, almost 50% of the falls came from mines representing only 5% of the production. This means that a large proportion of the reported falls are coming from relatively few mines. Figure 4 shows the roof fall rate (falls/ft of driveage) for intersection falls only. It mirrors the rate for all falls.

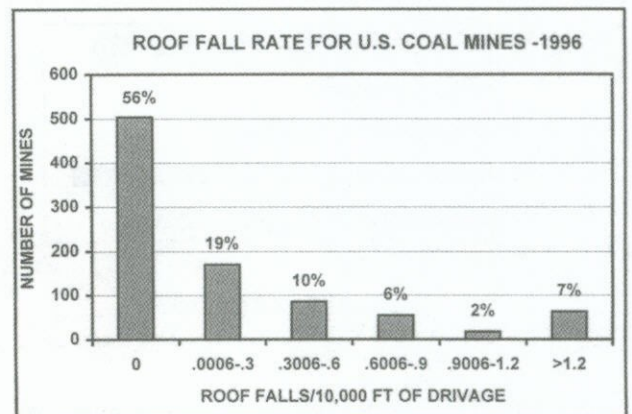


Figure 3. Roof fall rate for U.S. coal mines - 1996.

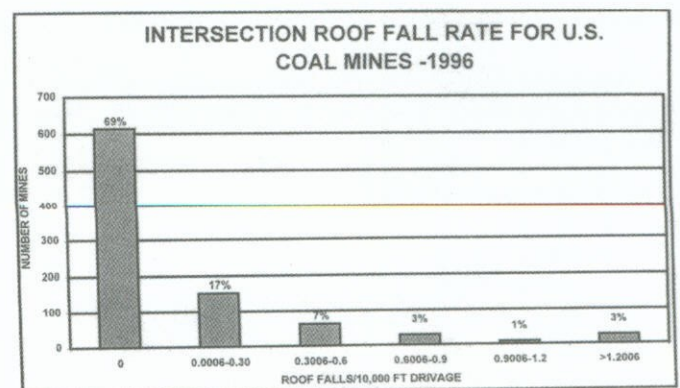


Figure 4. Intersection roof fall rate for U.S. coal mines - 1996.

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Figure 5 shows the fall rate for mines by seam height. The seams with the highest fall rate are >106 in. Figure 6 shows the distribution of roof fall rates by mine size (no. of underground employees). Small mines with less than 20 employees and mines with 51-150 employees had the highest fall rates.

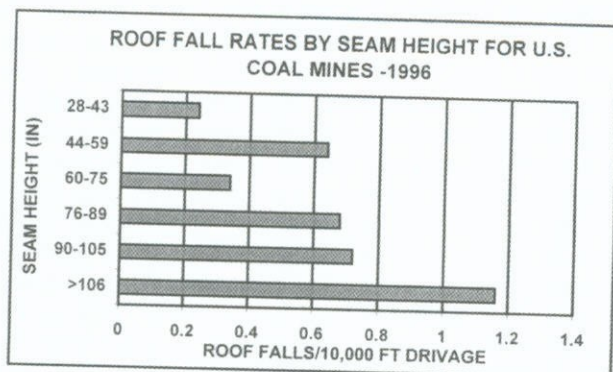


Figure 5. Roof fall rate for U.S. coal mines by seam height - 1996.

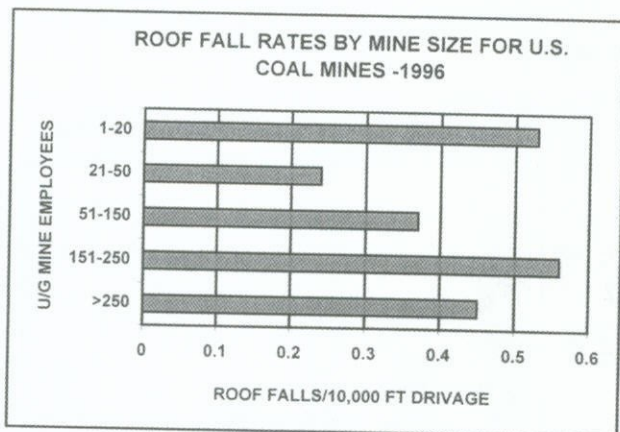


Figure 6. Roof fall rate for U.S. coal mines by mine size - 1996.

FIELD STUDY OF INTERSECTION INSTABILITY

There are a number of factors which affect the stability of coal mine roof in general and, intersections specifically. These factors include:

1. Roof geology (CMRR)
2. Entry span
3. Artificial support
4. Horizontal stress
5. Time between mining and support installation
6. Abutment

In order to study these factors, we have constructed a database of ground control variables from coal mines throughout the U.S. This database currently contains 12 mines and uses roof falls as the failure criteria. When the mine survey is complete the analysis will use a multivariate statistical approach to consider influence factors on roof falls and determine a design equation for intersection stability. Through experience the first three variables listed above have been shown to contribute most to the fall of roof. The other variables are important but are usually more site specific. In this project attention is being focused on roof geology as the key to intersection stability. In the past, empirical studies have been hampered by the large number of geologic variables which were recorded and applied to the model (e.g. uniaxial strength, RQD, thickness of the weakest bed, location of the weakest bed). Generally the influence of geology was divided between all the variables and not one of the variables exhibited a significant influence. The Coal Mine Roof Rating (CMRR) should resolve this problem (15).

In gathering roof stability data for each mine, standard procedures have been established to ensure data integrity. Extensive interviews with operators are conducted to identify the roof problems, establish the roof geology, and document support. From several failure criteria, the roof fall rate has been selected as the outcome variable which will be used to judge the influence of other mining and geologic variables. Non-injury roof falls are defined according to MSHA (16) as meeting any of the following criteria:

- Falls above the roof bolt anchorage.
- Falls which impair ventilation.
- Falls which impede passage of persons.
- Falls which cause miners to be withdrawn from the area affected.
- Falls which disrupt regular mining activities.

Falls of unsupported ground in the face area which do not trap equipment are normally not reported. Areas within the mine that are affected by abutment loading (longwall tailgates, pillar lines, multiple seam interaction zones) are excluded from analysis, as are zones of anomalous geology (eg. major faults). The distribution of standup time of supported ground is being determined, and data base will include only falls occurring less than 18 months after development. This is done so that roof falls that are time dependent and occur in outby areas are not given the same priority as falls near the face which expose more miners to hazard. At one mine near Providence, KY 50% of the falls occurred within one month of development, while only 10% of the falls occurred more than 12 months after development (17).

Roof geology is also broadly sampled by selecting mines for study which represent the full working range of CMRR. Within each mine enough roof exposures are observed to adequately define each of the significant roof types. Most often, this means visiting any and all roof falls that are accessible. CMRR verification is also done from core logs of holes on the property when possible. Roof bolts are identified on the mine

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map by the operator and verified underground by reading the markings on bolt heads. At the same time, immediate roof lithology is verified and intersection spans are measured. Standard procedures are followed for intersection corner location and the measurement of 3-way, turnouts, and odd intersection geometries. It is critical at intersection falls that the intersection span is measured, if possible. Eight intersections around the fall are measured to determine if any trend existed toward overspanned intersections in proximity to the fall. It is also important to measure spans in successfully supported ground in order to contrast these with failed ground.

Mine workings are then partitioned by roof bolt type and length, CMRR, intersection span, and other influencing variables like depth of cover and area beneath stream valleys. Drivages are then measured for these blocked areas in order to normalize their impact (Table 1).

TABLE 1 EXAMPLE OF DATABASE.

Mine	CMRR	Bolt Length	Tension	Grout	Bolt Capacity (Kips)	Bolts/Row	Row Spacing	Entry Width (ft)	Intersection Span (ft)	Depth (Avg.)	Height (ft)	Roof Fall Rate*
1	45	6	yes	None	18	4	5	19	65	350	5.5	5.69
2	41	5	No	Full	18	4	5	19	59	350	5.5	1.74
3	41	6	Yes	Partial	18	4	5	19	59	350	5.5	0.29

*Roof falls per 10,000 ft of drivage

Roof fall rates (falls per 10,000 linear foot of drivage) are then calculated from the reported falls in each area. In the end, each blocked area of the mine which constitutes some combination of the above variables is assigned a roof fall rate (Fig. 7). In this way the contribution of each variable to the roof fall rate can be calculated and converted into the regression equation.

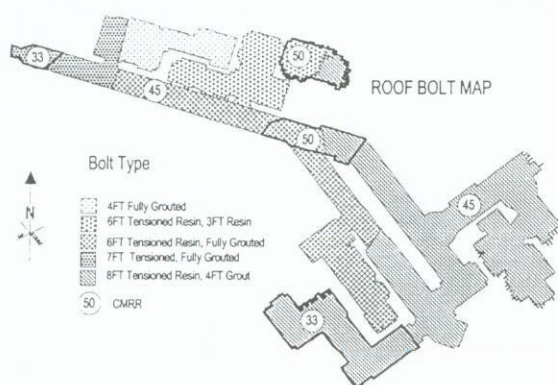


Figure 7. Example of partitioning of mine with overlapping geologic and bolt variables.

Current Database Results

The following are summary data from the current database. From 12 mines, over their entire existence, a total of 639 roof falls were tabulated. Roof fall rates were calculated per 10,000 ft of drivage for each mine (Fig. 8). The fall rate for intersections was also calculated. The mines are located in most of the major coal basins in the country and include small mines, large mines, room and pillar operations, and longwalls. The final data set will contain about 30 mines and will cover a range of poor roof to strong roof.

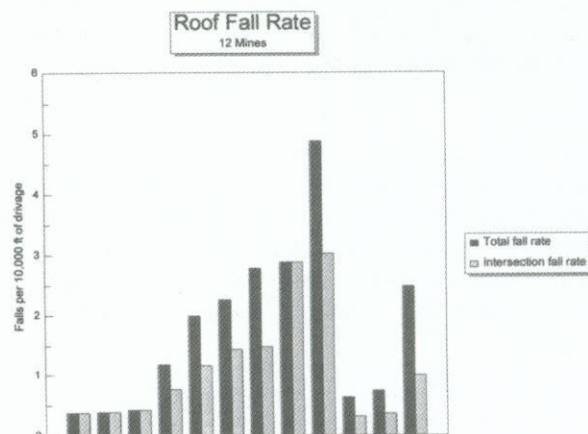


Figure 8. Total and Intersection fall rates for each of the mines in the database.

The roof falls which make up our mine sampling are classified by location in intersections or entry segment (the mined entry between intersections). Of the falls now included in the database, 59% now occur in intersections.

In order to compare the stability of intersections vs. entry segments, the falls were calculated as a percentage of total mined places fallen. Figure 9 shows that intersections were 2.5 times as likely to fall as entries. Individually in the 12 mines, the ratio of failed intersections to failed entries ranged from 1.5 to 3.3.

Coal Mine Roof Rating

Past experience at individual mines demonstrates a correlation between CMRR and roof fall rate. (11). Roof fall rate decreased as the CMRR increased. The CMRR for the database ranges from 32 (extremely weak) to 75 (very competent). Figure 11 shows significant variation in fall rate with individual CMRR, indicating that other factors also influence the fall rate. Individual mines show variable degrees of correlation between CMRR and roof fall rate. At one mine in Mingo Co., WV the roof fall rate for a weak shale (CMRR = 40) was 3.4 per 10,000 ft of drifage, while the roof fall rate for a firm shale or stackrock (CMRR = 50) was 1.0 per 10,000 ft of drifage. Both rock types were bolted with 48 in fully grouted bolts.

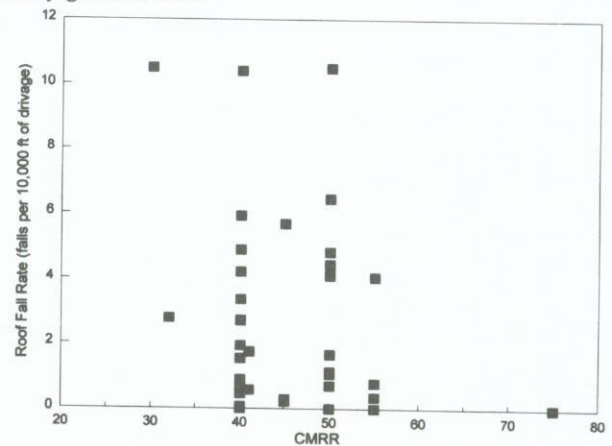


Figure 11. Relationship of CMRR to roof fall rate.

A relationship was found between CMRR and intersection span (Fig. 12). Generally, as CMRR increased in the 12 mines in the database, the average intersection span (sum-of-the-diagonals) increases. Factors such as roof support and local mining practice influence this relationship. This may indicate that operators know from experience that stronger roof is better able to support a larger span and this is reflected in their mining widths.

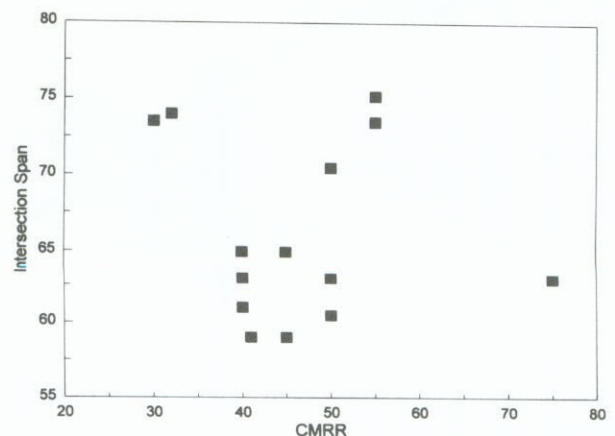


Figure 12. Relationship of CMRR to intersection span.

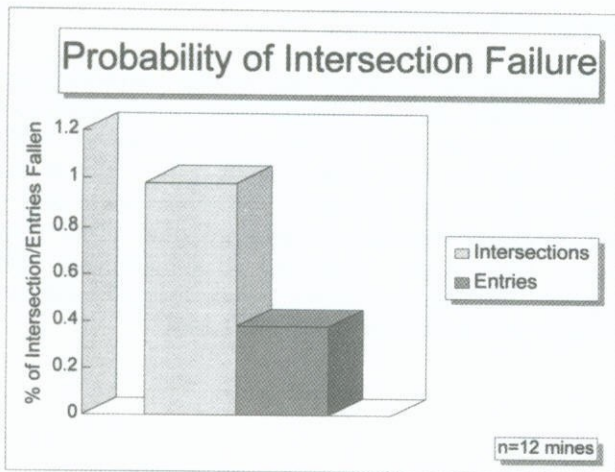


Figure 9. Comparison of intersection and entry failure rates.

The relative stability of the type of intersection (3-way vs. 4-way) was also investigated using the preliminary data from the 12 mine database. The percentage of 3-way and 4-way intersections was tabulated. Then the percentage of 3-way falls and 4-way falls was determined. Figure 10 shows the proportion of 3-way vs. 4-way falls in comparison to the percentage of all of these types of intersections. 42% of all intersections were 3-ways, but only 36% of all falls in intersections were in 3-ways. 58% of all intersections were 4-ways, but 64% of all falls in intersections were in 4-ways. Proportionately there was a higher fall rate in 4-ways than 3-ways. In this data set then, a 4-way intersection is 1.28 times as likely as a 3-way to fall. However, since it would normally take two 3-way intersections to replace one 4-way, the remedy of replacing 4-ways with 3-ways is not likely to reduce the number of roof falls. It would take a fall rate of at least twice as high for 4-ways to consider replacing them with 3-ways. A mine that systematically replaced 4-ways with 3-ways would be expected to see a 56% increase in total number of falls.

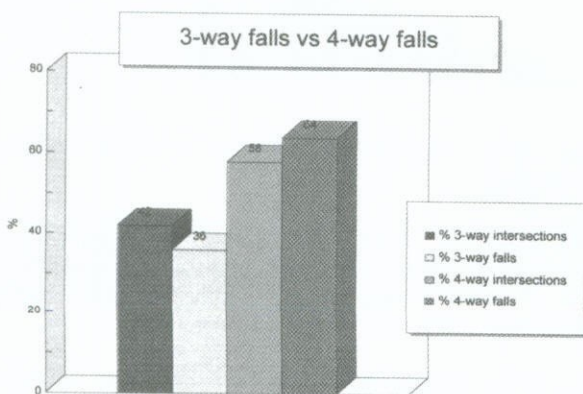


Figure 10. Failure rate of 3-way vs. 4-way intersections.



Intersection Span

The average intersection spans were calculated for each combination of variables comprising each mine partition and represented as one row in the database. Figure 13 shows the correlation between intersection span and roof fall rate for all CMRR values of 45 and below. One mine with a wide intersection span (74 ft) and a lower roof fall rate (2.78) can be explained because heavy steel sets may be supporting failed roof and contributing to a lower roof fall rate. The CMRR values represent the weakest roof. The weakest roof rocks are the most sensitive to large spans and have the highest roof fall rates.

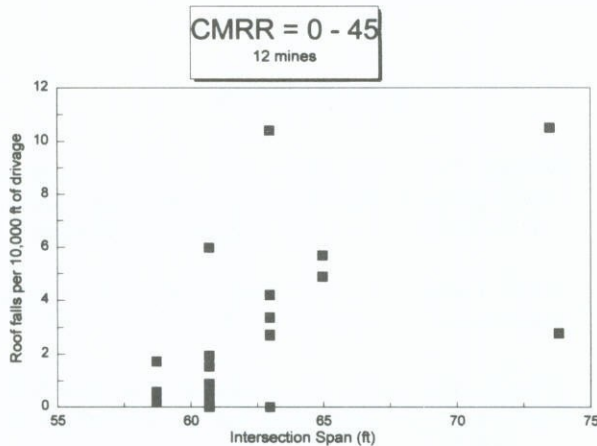


Figure 13. Relationship of intersection span to roof fall rate for weak rocks.

A number of ways to minimize intersection spans have been proposed. In one Pennsylvania mine, roof conditions deteriorated in wide intersection spans (one diagonal measured 52 ft) when a track chute entered a panel and also intersected a crosscut. By dropping one crosscut and staggering another, the span was reduced. Reducing the number and location of turnouts should result in reduced intersection spans. The practice of driving crosscuts straight across the developed entries, where possible, as well as supporting at-risk intersections before they become intersections, will stabilize spans (8).

Roof Bolting

There were numerous bolt types, lengths and spacings used in the twelve mines. These ranged from 4 ft to 12 ft lengths, tensioned and untensioned bolts, and ungrouted, partially grouted, and full-column grout. Bolt spacing varied from 3-5 bolts per row and 4-6 ft row spacing.

Poor roof bolt installation may be able to account for some of the variation in the roof fall rate regression model. This topic includes the following: long lag times before bolting, overdrilled holes, loss of tension, loss of resin due to lack of containment, etc. This type of data can be important but very difficult to obtain. In one mine we found a high rate of tension bleed off (55% of bolts tested had lost tension). Also a significant number of bolts observed in roof falls were "finger-gloved," indicating the resin was not mixed properly. Long bolting lag times are more damaging in laminated rock with a CMRR in the 40's or below. Data shows that 50-80% of the total deformation and load changes occur within a few hours or days after initial excavation (8).

Case History

At one western Pennsylvania mine in the Pittsburgh seam 55 intersection diagonals were measured. The measured data considers intersection spans in fallen areas and non-fallen areas. The falls observed in this mine were cleaned up and accessible. The most significant observation was that for 83% of the roof falls, the sum-of-the-diagonals exceeded 70 ft (Fig. 14). In contrast, none of the non-fallen bolted intersections had a span exceeding 70 ft. As the span increases so does the amount of support. Supplemental support was necessary in 10 intersections where the average span was 67.1 ft. Although none of these were fallen, they required additional support to stabilize them. The mean span for the roof falls at this mine is only 11% greater than the mean for the stable intersections, but the estimated rock load is 43% greater. The CMRR for this mine is 40, typical of the Pittsburgh seam. The correlation of increased span with more roof falls is an indication that a sum-of-the-diagonals of 70 ft or less is necessary for stable roof in this geology.

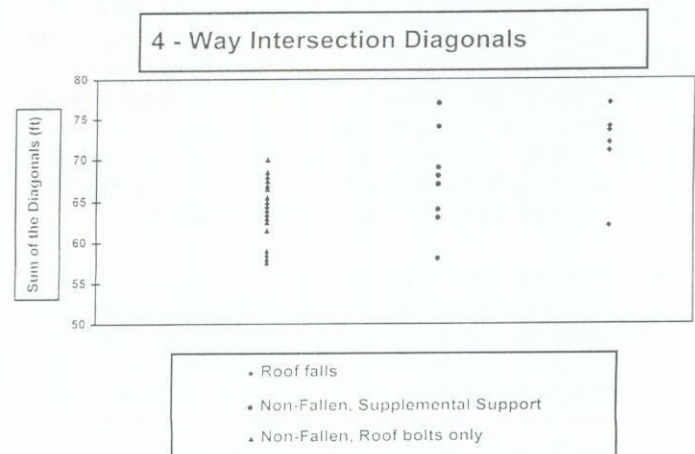


Figure 14. Intersection spans for roof bolts, supplemental support, and fallen roof areas at an eastern coal mine.

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Mining method has also played a role in intersection stability at this mine. Where a miner-bolter was used to drive development sections there was a dramatic improvement in roof conditions. No falls have occurred in development by miner-bolter. If the roof fall rate in these areas were the same as that of areas driven by place-change miners, 10 falls would have been expected. The improvement may have been due to the smaller entry width and intersection span resulting from the miner-bolter, or to quicker roof support installation, or a combination of both.

CONCLUSIONS

By using the roof stability experience of the U.S. coal mining industry, a model of the parameters which influence roof failure is being developed. Through a system of monitoring standup time of roof falls, and sorting out anomalous causes of falls (abutment loading, unusual geology, etc.), roof falls are counted and roof fall rates are assigned to partitioned areas of the mine. Then roof geology is determined, intersection spans are measured, and roof bolts are documented for each partition.

MSHA and NIOSH roof fall data has shown that intersections are, on a foot by foot basis, about 8-10 times more likely to fall than entries. Three variables appear to have the most influence on roof fall rates:

- Coal Mine Roof Rating - For the weakest group of roof rocks (CMRR ≤ 45), roof fall rates increase for increasing sum-of-the-diagonals intersection span, indicating that weak rocks have some limit of stable span. Variability in this correlation is due to increased support in at-risk intersections. It is critical to accurately define the roof fall rates for CMRR in the weakest roof rocks in order to anticipate instability.
- Intersection Span - For the entire range of CMRR in the sample population, intersection span increases with increasing CMRR, confirming that stronger rocks can support bigger spans. It is the transition zones between weak, moderate, and strong CMRR ranges which have to be better defined. Case studies have indicated that replacing 3-way intersections with 4-way intersections may not reduce the total number of roof falls.
- Roof Support - Individual case histories show various roof fall rates attached to different support. At one mine supplemental support was necessary in numerous intersections where the span exceeded 67 ft. At another mine loss of bolt tension in 55% of the tested bolts may have contributed to the extremely high roof fall rate. At another mine, roof falls were dramatically reduced by the use of 8 ft point anchored, resin-assisted bolts. These types of individual results will be more meaningful when analyzed relative to the whole database.

The effects of mining method (miner-bolter vs place change mining and practice (intersection development, poor roof bolt

installation) also have an impact on intersection stability. When complete, the database will contain data sets on 30-35 U.S. coal mines and a multivariate analysis will be used to determine the influence of each variable on the ultimate stability of an intersection.

FUTURE RESEARCH

We will continue to collect field data from coal mines and expand our data base to include 30-35 mines. With that data as input, a multivariate analysis will be applied. The end goal will be to develop a design model for the safe excavation and support of intersections.

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Analysis of the Effect of Rate of Extraction on Strain Development in Interaction Subsidence Data Using a Neural Network

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ABSTRACT

In 1993 an opportunity presented to monitor the effects of undermining an abandoned roadway by a longwall panel in the Sydney, N.S., coal field. Interaction subsidence data was collected by survey leveling to benchmarks in the undermined gateroad, as well as by a standpipe system equipped with 15 vibrating wire piezometers measuring changes in elevation in the gateroad relative to an outby header tank. The piezometers were monitored at 30 minute intervals using a datalogger, while the leveling surveys were conducted weekly over the life of the project.

The monitoring program resulted in the collection of a very large body of data, relating vertical subsidence to the relative position of the underlying face. The most basic analyses derived a travelling subsidence curve from the leveling data only, which hinted at a relationship between rate of extraction in the panel below and the amount of strain developed in the undermined strata.

The complexity of the relationships between the amount of subsidence obtained, the location of the individual monitoring stations, the rate of face advance below and the potential for "periodic" subsidence due to geological conditions has made analysis difficult, and the amount of data has compounded this problem. Recently, an analytical approach using a commercial neural network analysis package has been applied, which appears to have resulted in the ability to draw some meaningful conclusions from the data set.

In this paper, a short history regarding the collection of the data set is followed by a brief description of neural networks. This is followed by a description of the several analyses performed, some of the results obtained, and conclusions drawn.

INTRODUCTION

Until recently the Cape Breton Development Corporation (CBDC) produced coal from three mines in the Sydney Coal Field, which extends northeast under the Gulf of St. Lawrence from the east coast of Cape Breton Island. At present, coal is

produced from two mines, Prince and Phalen Collieries. The Phalen Colliery extracts coal from the Phalen Seam offshore of New Waterford in the central part of the coal field. Longwall panel and pillar retreat mining is practiced, with in-seam gateroads supported by rock bolts. Abandoned and flooded mines flank the present workings in the Phalen Seam.

Until November 1992, the Lingan Colliery was working the Harbour Seam, which lies 138 m above the Phalen Seam. Workings in the Harbour and Phalen Seams were concurrent, but in different areas.

In November 1992, the 5 East Panel at Phalen Colliery was retreating towards the main access deeps of the mine. In doing so it had undermined flooded workings in the Harbour Seam to the east of the overlying Lingan Colliery. As the 5 East faceline began to undermine an abandoned and sealed panel in the Lingan Colliery 138 m above, a flow of water was detected from behind a fire seal on the abandoned panel. A major inflow of water followed, eventually causing the closure of the Lingan Mine, with the loss of about 700 jobs.

From observations of water levels in adjacent abandoned mines and from water chemistry analyses, it was eventually determined that the most likely source of this water was the adjacent abandoned and flooded workings. The mechanism by which the water made its way through a 350 m wide solid inter-colliery pillar between the adjacent flooded workings and Lingan Colliery is still not known for certain. It has been suggested that bed separation at or above the Harbour Seam horizon, caused by longwall workings in the Phalen Seam, provided the hydraulic connection.

INTERACTION MONITORING PROGRAMME

The potential effect of Phalen Seam workings at the Harbour Seam horizon, and concern over the potential for the flow of water into the Phalen Seam workings from flooded workings above became a major concern to CBDC. Also at stake was the significant loss of revenues that would result from restricting the extracted height in the Phalen Seam to reduce tensile strains at the flooded Harbour horizon.

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CBDC enlisted the assistance of the CANMET MMSL Sydney Laboratory (CSL) in Sydney, N.S., to investigate the effects of the extraction of the Phalen Seam at the Harbour Seam horizon. An ideal monitoring opportunity was offered by the extraction of the 1 Centre Panel at Phalen Colliery, which lay directly beneath the abandoned and sealed 5 East Top Level in Lingan Colliery (Figure 1). However, by this time Lingan Colliery had ceased operation and was being allowed to flood. Water pressures monitored behind fire seals in old panels above outby 5 East Top Level were still indicating high water levels, although flow rates into the Lingan Mine appeared to be stable at about 1500 igpm. It was by no means certain that the program could be completed, but CBDC undertook to plan the rehabilitation of the 5 East Top Level, with careful monitoring of the conditions in the abandoned district and the water situation in the panels above it.

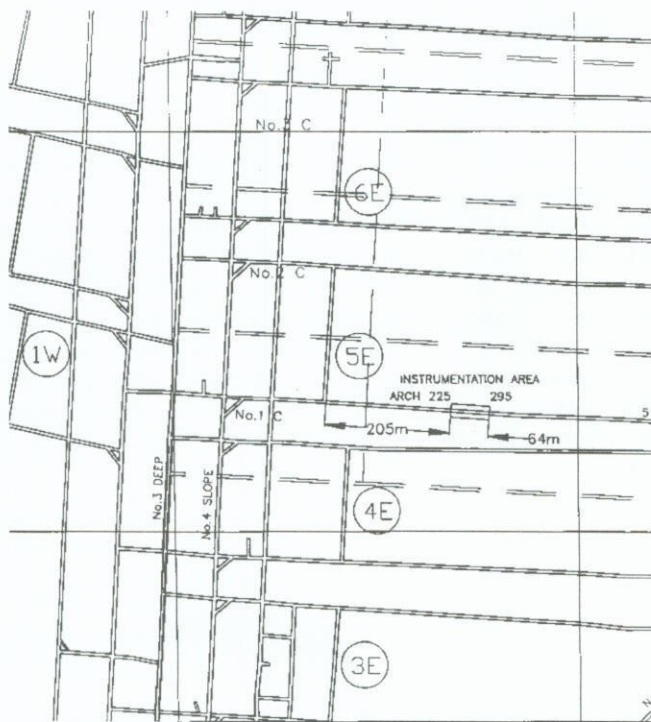


Figure 1: Location of Monitoring Site with Respect to Underlying Workings

CSL undertook to fund a contract for the design, construction and installation of the instrumentation in 5 East Top Level. Jacques, Whitford and Associates Limited (JWA), of Dartmouth, Nova Scotia were retained by the Canadian Federal Department of Supply and Services to provide the following services:

- Design of a suitable monitoring program to accurately measure strains, subsidence and bed separation at or near the Harbour horizon during the mining of the 1 Centre Longwall, in the Phalen Seam, 138 m below.
- Fabrication of the entire instrumentation package, including the purchase of instrumentation and all conduit and

connections.

- Installation of the instrumentation package at the selected site, and all connections to a remote base at No. 4 Slope, Lingan Colliery.

These tasks were undertaken in close consultation with CSL staff, including significant input from them on the selected instrumentation design.

The entire program, including the rehabilitation of the abandoned district, the selection and design of the instrumentation, and installation and monitoring procedures, has been described elsewhere (1), and early results presented (2).

Survey Leveling

The importance of the monitoring program to CBDC, combined with the risks to the program presented above, resulted in the adoption of an instrumentation plan that included conventional survey leveling and electronic distance measurements (EDM) to floor mounted benchmarks in the level, and direct monitoring of subsidence and ground strain by carefully selected instrumentation (2).

Survey benchmarks were established at regular intervals (approximately 4.5 m apart) along the level from the face start line to a position 80 m inbye the monitoring section. The benchmarks consisted of standard 2.4 m x 22 mm rebar rock bolts. They were installed in 2.0 m deep holes drilled within 1 m of the high side of the level, angled at 30° to the high side. The bolts were secured in the hole by one capsule of slow set rock bolting resin, and stabilized with wooden wedges. The wedges maintained the bolts in position during surveying, but would allow any ground movement to occur without disturbing the bolts.

Installation of the monitoring system was carried out between February 22 and 26, 1993 inclusive. On completion of the installation work, the positions of the floor pins, and the positions of the instruments on the pins, were measured and recorded.

Survey leveling began immediately after the bolts were installed. CBDC surveyors began weekly surveys using conventional leveling and electronic distance measuring (EDM) techniques. The first two surveys, which took place on February 15 and 22, 1993, included all the bolts to obtain baseline readings. Subsequent surveys were restricted to the last 26 bolts until movement was noticed, after which an additional 10 stations were monitored each week to keep pace with the face advance in the seam below.

Vibrating Wire Piezometers

After an inspection of 5 East Top Level, a site was selected for detailed monitoring of subsidence. Fifteen additional pins adjacent to the survey benchmarks were installed over a roadway length of about 70 m, spaced about 4.5 m apart.

A vibrating wire piezometer was attached firmly to each pin, and connected to a standpipe which ran from the monitoring site to a tank located outside the area to be undermined. The

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piezometers monitored the change in head pressure between the header tank, kept at a constant level by an overflow, and each monitoring position, allowing the amount of subsidence to be determined.

After initial problems with leakage from the standpipe, the system worked very well. Monitoring was undertaken with a datalogger at 30 minute intervals over the lifetime of the monitoring project (2,3).

RESULTS

Proprietary spreadsheet software allows a number of relationships to be examined prior to further analysis.

The location of the underlying face with respect to each monitoring position was determined from production data supplied by CBDC.

General Data Review

Figures 2 and 3 show the development of the subsidence curve with respect to the position of the underlying face for the leveling and piezometer data respectively. Several features are immediately apparent on both graphs:

- little or no subsidence occurs prior to the passage of the face. The apparent "upsidence" illustrated by the piezometer data is discussed later;
- the maximum curvature, and hence the location of the maximum tensile strain, occurs at or about the edge of the underlying extraction;
- the subsidence curve is steepest, i.e. the rate of subsidence is greatest, at a distance of between 75 and 100 metres behind the faceline (this also corresponds to the point of inflection of the subsidence curve, and the point at which strains go from tensile to compressive);
- the maximum subsidence appears to occur between 300 and 400 metres behind the faceline. The leveling data shows a maximum subsidence of between 1.1 and 1.2 metres, while the piezometer data indicates a maximum subsidence of slightly less than this - 0.9 to 1.1 metres;
- although the subsidence trends are fairly clear, there is definitely some scatter of the data points.

The "upsidence" shown in Figure 3 appears despite careful correction of the raw data for atmospheric pressure variation and temperature compensation of the piezometers. One possible explanation is that a rise in water temperature due to warming from March to July resulted in a decrease in density in the standpipe, and hence apparent upward movement. There is some evidence of a warming trend from temperature data collected at the monitoring site. The travelling subsidence profile after correction for ambient temperature effects on water density is presented in Figure 4. It is very similar to the curve derived from the leveling data, indicating that full subsidence might not have been reached (compare Figure 3) but shows slightly more subsidence - about 0.25 metres more 300 metres or so behind the face.

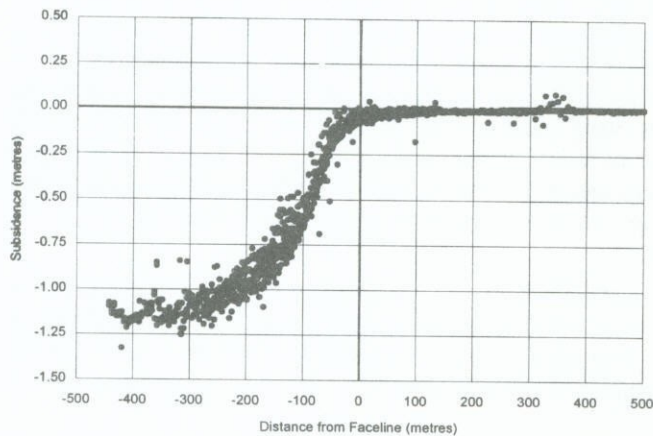


Figure 2: Subsidence, from Leveling Data

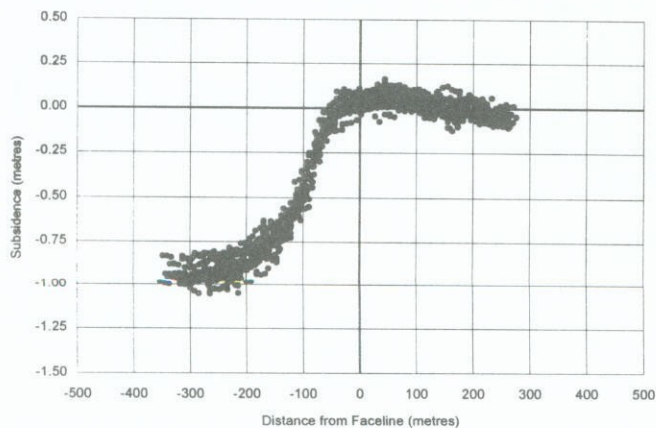


Figure 3: Subsidence, from Piezometer Data

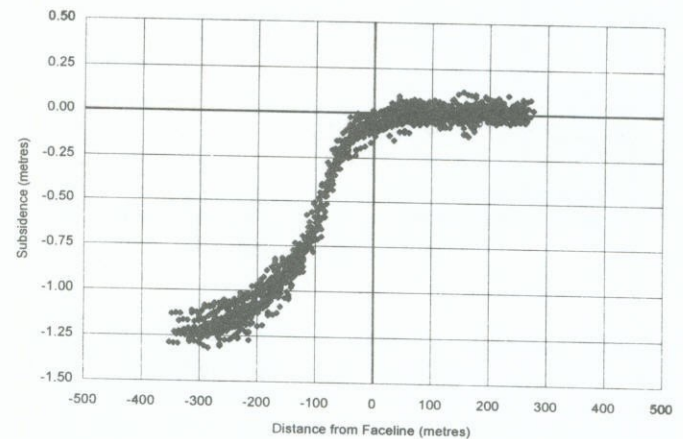


Figure 4: Corrected Subsidence, from Piezometer Data

Causes of Data Scatter

As previously mentioned, the data shows considerable variation in measured subsidence, especially behind the face and at the onset of subsidence. There are a number of possible



explanations for this, apart from the inherent "measurement error", including variable roadway conditions at the monitoring stations (floor heave, etc) and variations in surrounding strata.

Independent observations on the undermining panel indicated the presence of periodic weighting effects. Periodic weighting, and hence caving, would be expected to affect the subsidence of the overlying roadway, possibly showing up as a variation in maximum subsidence obtained. A preliminary analysis of this data set (2) indicated this possibility. The location of measuring stations at 4.5 m intervals over a 70 m length of roadway might therefore be expected to show some evidence of variations in subsidence at individual monitoring stations.

Another possible explanation is the effect of rate of extraction on subsidence. There is unpublished evidence from Gemany that this effect exists, but apart from a cursory analysis of the current data set in this respect (2), there has been no conclusive evidence presented in the literature.

The remainder of this paper describes the analytical method used to process the data, and some of the results obtained.

NEURAL NETWORKS

The following information on neural networks is adapted from the Qnet users manual (4).

A neural network is best defined as a set of simple, highly interconnected processing elements that are capable of learning information presented to them. The foundation of neural network theory is based on studies of the biological activities of the brain. A neural network's ability to learn and process information classifies it as a form of artificial intelligence (AI).

Neural networks can be effectively applied to a vast array of problems, many of which have been thought to be too complex or lacking in sophisticated theoretical models. Neural networks are responsible for making significant advances in the traditional AI fields of speech and visual recognition. Investment managers are creating investment models to better manage money and improve profits. Scientists and engineers use them to model and predict complex phenomena. Marketing professionals are employing neural networks to accurately target products to potential customers. Geologists can increase their probability of finding oil. Lenders use neural networks to determine the credit risk of loan applicants. Complete neuro-computing hardware systems are being built for use in everything from automobiles to manufacturing systems. The variety of problems that can be solved effectively by neural networks is virtually endless.

A Brief Introduction to Neural Networks

A human brain continually receives input signals from many sources and processes them to create the appropriate output response. Our brains have billions of neurons that interconnect to create elaborate "neural networks". These networks execute the millions of necessary functions needed to sustain normal life.

For some years now, researchers have been developing models, both in hardware and software, that mimic a brain's cerebral activity in an effort to produce an ultimate form of

artificial intelligence. Many theoretical models (termed paradigms), dating as far back as the 1950's, have been developed. Most have had limited real-world application potential, and thus, neural networks have remained in relative obscurity for decades.

The back-propagation paradigm, however, is largely responsible for changing this trend. It is an extremely effective learning tool that can be applied to a wide variety of problems. Back-propagation related paradigms require supervised training. This means they must be taught using a set of training data where known solutions are supplied.

Back-propagation type neural networks process information in interconnecting processing elements (often termed neurons, units or nodes). These "nodes" are organized into groups termed layers.

There are three distinct types of layers in a back-propagation neural network: the input layer, the hidden layer(s) and the output layer. A network consists of one input layer, one or more hidden layers and one output layer. Connections exist between the nodes of adjacent layers to relay the output signals from one layer to the next. Fully connected networks occur when all nodes in each layer receive connections from all nodes in each preceding layer.

Information enters a network through the nodes of the input layer. The input layer nodes are unique in that their sole purpose is to distribute the input information to the next processing layer (i.e., the first hidden layer). The hidden and output layer nodes process all incoming signals by applying factors to them (termed weights).

Each layer also has an additional element called a bias node. Bias nodes simply output a bias signal to the nodes of the current layer. Some applications, including the Qnet application described below, handle these bias nodes automatically. They do not need to be included or specified by the user.

All inputs to a node are weighted, combined and then processed through a transfer function that controls the strength of the signal relayed through the node's output connections.

The transfer function serves to normalize a node's output signal strength between 0 and 1. Some common transfer functions are: the sigmoid, gaussian, hyperbolic tangent and hyperbolic secant functions. Network processing continues through each layer until the network's response is obtained at the output layer.

When a network is used in recall mode, processing ends at the output layer. During training, the network's response at the output layer is compared to a supplied set of known answers (training targets). The errors are determined and back-propagated through the network in an attempt to improve the network's response. The nodal weight factors are adjusted by amounts determined by the training algorithm. The iterative procedure of processing inputs through the network, determining the errors and back-propagating the errors through the network to adjust the weights constitutes the learning process. One training iteration is complete when all supplied training cases

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have been processed through the network.

Qnet

Qnet is a commercial neural modeling system that is designed to exploit today's more powerful PC hardware and operating systems. Qnet executes in 32-bit protected mode of 386, 486 or Pentium™ class CPU's. Network sizes are limited only by the computer system's memory and disk capacity. Qnet offers advanced network design features for creating complex networks that learn using highly optimized backpropagation training algorithms. Qnet features include:

- Speeds over 700,000 connections per second on Pentium based PC's.
- On-line help.
- Fast, easy network design.
- Easy data interfacing
- Complete interactive analysis of the training process.
- AutoSave to automatically store the network model during training.
- Learn Rate Control to automate network training.
- Multiple training algorithms.
- A network recall mode
- Example problems.

Qnet was selected and purchased to conduct the data analysis after a review of literature on available software. The use of Qnet by the author does not imply a recommendation, although it was found to meet the claims made by the manufacturers and proved very easy to use.

NEURAL NETWORK ANALYSIS OF SUBSIDENCE DATA

Neural networks have shown their ability to "learn" about the relationships between a number of independent variables, for example, household, demographic and income/expenditure statistics of loan applicants, and their effect on single or multiple dependant variables, for example, loan risk.

In this application, there were only three independent variables considered;

- Location of the faceline with respect to the monitoring station.
- Location of the monitoring station in the gateroad, and
- Rate of extraction.

A number of other independent variables might also cause variations in subsidence or subsidence rate. For example, height of extraction in the undermining panel, variations in the stress on the pillar above 5 East Top Level and variations in local geology.

The height of extraction was virtually constant over the whole of 1 Centre Panel, therefore this was not considered as having an effect on the measured subsidence values.

The only evidence available regarding pillar stresses suggested that there was little variation in stress over the length of the monitoring section. The evidence is indirect, being the results of roadway height surveys conducted in 1983 and 1993. The results of the surveys, and the pillar width along 5 East Top Level, are shown in Figure 5. The location of the monitoring site (piezometers) is shown. The survey pins extended towards the vertical axis.

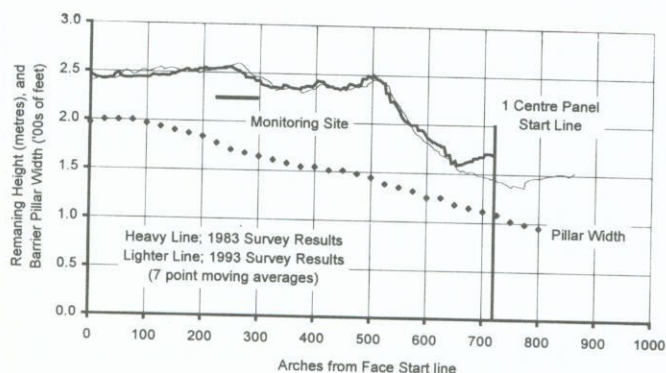


Figure 5: Pillar Widths Adjacent to the Monitoring Site

The absence of any significant reduction in roadway height over the length of the monitored roadway suggests that there was insignificant effect of increased pillar stresses on the roadway, although closer to the 1 Centre face start line there is a significant actual and time dependent reduction in roadway heights associated with reduced pillar widths.

The dependant variable in each analysis was either the amount of subsidence, or the rate of subsidence.

This probably represents an application of neural network analysis that might have been equally successful using traditional statistical methods. However, as the purpose of the analysis was also to demonstrate the potential of neural networks over a wide range of applications, neural network analysis was used exclusively, even when developing simple relationships, such as the travelling profile with respect to the face position.

Subsidence Related to Periodic Caving

One of the potential factors to affect the amount of subsidence developed at each monument was the effect of periodic weighting, and hence caving and subsidence development. In order to determine if such an effect might be present, the final subsidence values measured at each survey pin and piezometer were obtained as an average of the last few readings (the last three in the case of leveling pins, the last 4 in

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the case of the piezometers). Figure 6 shows the results, plotted against the original distance from the 1 Centre start line.

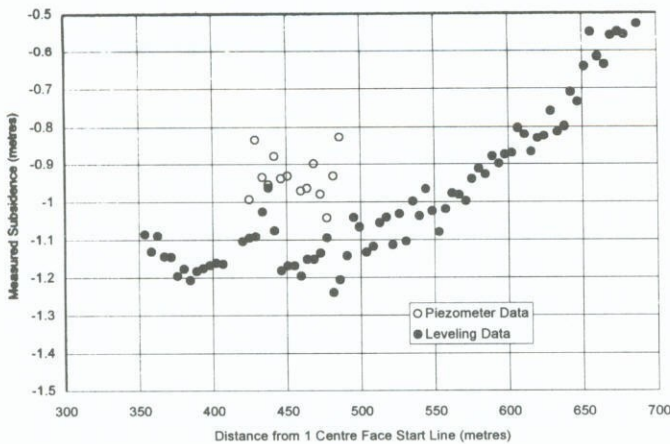


Figure 6: Final Subsidence Values

Data points in the region 500 to 700 metres from the 1 Centre Face start line (i.e. those closest to the final position of the face) show clearly that maximum subsidence had not been obtained when survey leveling stopped.

Data points in the region 350 metres to 500 meters from the 1 Centre Face start line (i.e. those that were undermined first) show some indication that maximum subsidence had been achieved, but there is no apparent indication of cyclic subsidence resulting from periodic weighting and caving in the 1 Centre waste below contradicting the earlier indications presented by Cain et al., (1996). As a result of this finding, no further analysis was done to explore this effect.

Subsidence Related to Rate of Face Advance

Early analyses of this data set (2) purported to show that the rate of subsidence increased with the rate of extraction (Figure 7). The data set this relationship was derived from was the leveling data set used in the current analysis.

Leveling Data

In Figure 7, the subsidence rate is presented in metres per day, plotted against distance from the faceline. This presentation would inevitably show the trend indicated – the greater the amount of material extracted in one day, the greater the amount of subsidence – and so any inferences drawn from it do not provide conclusive proof of the proportionality of tensile strain and advance rate.

In a further stage of analysis, the rate of subsidence was expressed as metres subsidence per meter per day of face advance. If there is a relationship between subsidence and rate of face advance, the rate of subsidence expressed in this way should increase or decrease with rate of face advance, or stay constant.

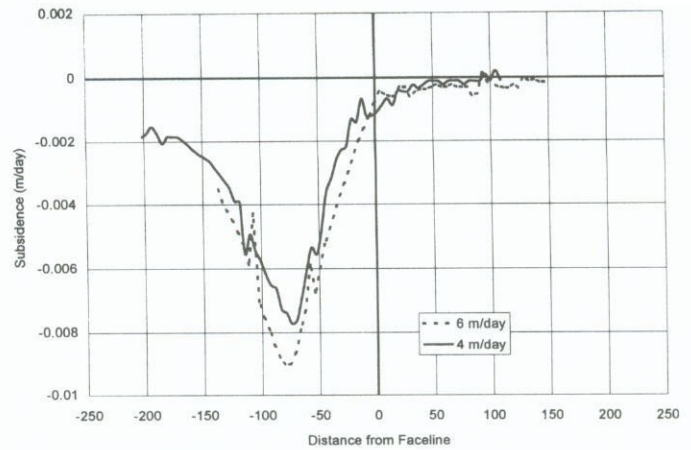


Figure 7: Effect of Rate of Advance on Rate of Subsidence (after (2))

A neural network model was constructed using the Qnet package described. Two input nodes accepted rate face advance and distance from the faceline, and the output was the rate of subsidence expressed as described. To reduce the learning time required, a subset of the data, from 150 metres in front of the face to 150 metres behind the face was used. The model was “trained” on the monitoring data, using about one fifth of the data set as the “test” data.

The model, when trained, had an RMS error of the training and test data sets of 0.04, and correlation coefficients of 0.94.

The trained model was then run in recall mode to calculate subsidence rates for distances of +150 m to -150 m either side of the face line at advance rates of 3, 4, 5, and 6 metres per day. The monitoring results and modeled results of rate of subsidence were in reasonable agreement, exemplified by Figure 8.

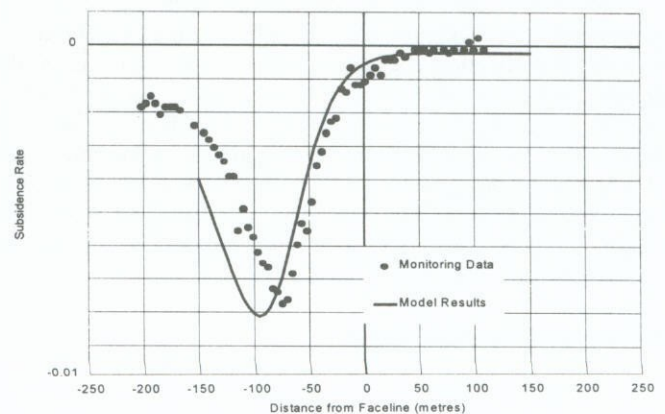


Figure 8: Comparison of Actual and Modelled Rate Data

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The subsidence rates were then converted to subsidence per day, following which a cumulative subsidence curve was calculated, from which strains were developed using the NCB SEH method (5). Figure 9 clearly shows that the model predicts increasing tensile strains as the rate of face advance increases.

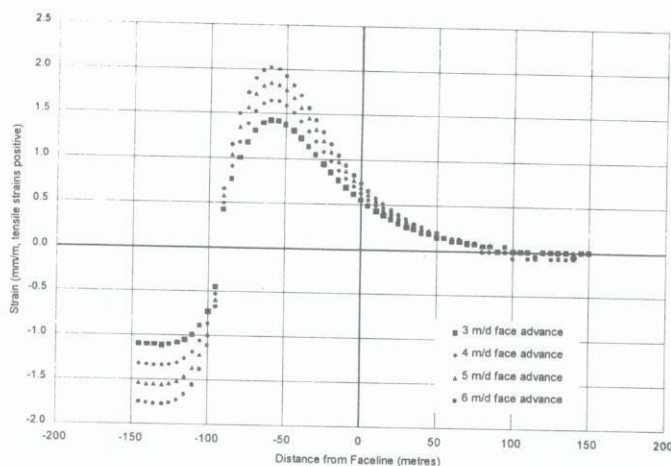


Figure 9: Tensile Strains Predicted From Model Results

Piezometer Data – Weekly Results

The full piezometer data set includes readings taken at 30 minute intervals. Initially, readings corresponding to those obtained from the leveling surveys – i.e. on a weekly frequency over the monitoring time span – were extracted for comparison purposes. The extracted data set was considerably smaller than the leveling data set, consisting of only fifteen individual monitoring stations from which full subsidence was determined (in the case of the leveling data set, more than thirty individual sites were monitored to full, or close to full, subsidence, and almost all of them were fully monitored between +150 m/-150 m each side of the longwall face line.)

The same neural network model was used on the weekly piezometer data. Training was successful, with an RMS error of 0.03 and a correlation coefficient of 0.98. However, the results obtained were not as good as those obtained from the leveling data. Figure 10 shows the subsidence rates predicted for advance rates of 3, 4, 5, and 6 metres per day. Wide variations both ahead of and behind the face line are evident. The small number of monitored stations, compared to the number monitored in the leveling surveys, is thought to be the principal reason for the poor adaptation of the model in this instance.

Piezometer Data – Full Data Set

What the piezometer data set lacks in monitoring stations, it makes up for in frequency of monitoring. While the daily rates of face advance used in the analysis of the leveling data are averages over the time period between surveys, the piezometer data set includes readings at half hour intervals, with accurate face advance data on a shift by shift basis.

However, is the subsidence at a particular point related to extraction that takes place in the shift the subsidence is

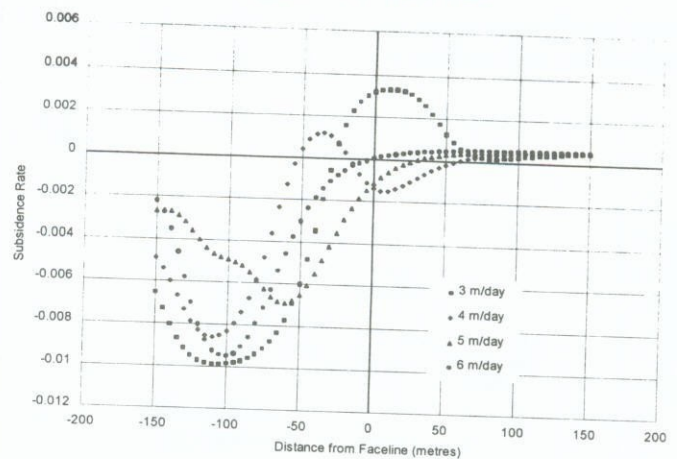


Figure 10: Modeled Rate of Subsidence Based on Piezometer Data.

measured, the shift before that, or the shift a week before that? Some indication of this can be obtained by examining subsidence records obtained before and after periods of face inactivity. Data previously presented (3) shows that strata response was within one shift of the cessation or commencement of mining operations.

Fortunately, the ability of the neural network to “learn” relationships should take this into account as the model develops over the learning process.

A first attempt to model subsidence was made using the full shift by shift data set collected at the piezometer monitoring stations. However it was found that the neural network had great difficulty in producing a reasonable model. Perhaps one reason for this is that the rates were so small that they approached the errors associated with the instrumentation.

In the second attempt, the piezometer data set was reduced to include data on a daily basis, and this time the neural network produced a model with an RMS of about 0.05 and a correlation coefficient of 0.85. The rates of subsidence obtained from 100 m in front of the face to 150 m behind the face were converted back to subsidence, and strains calculated from the subsidence curves as before. Figure 11 illustrates the increase in modeled strains with respect to rate of face advance.

Figure 12 shows the final product of the analysis, the relationship between maximum derived tensile strain and rate of face advance, indicating that as rate of face advance increases, so the maximum derived tensile strain increases.

DISCUSSION

Neural networks are not yet a widely used tool in data analysis, and the work presented above indicates that they might have a role in the modeling of typically complex relationships found in geotechnical monitoring.

The conclusion, that there is a relationship between rate of face advance is not new – as reported above such a relationship

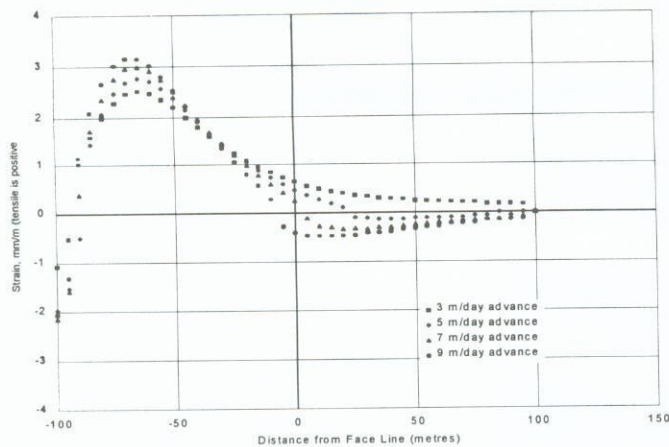


Figure 11: Increase in Strain with respect to Face Advance

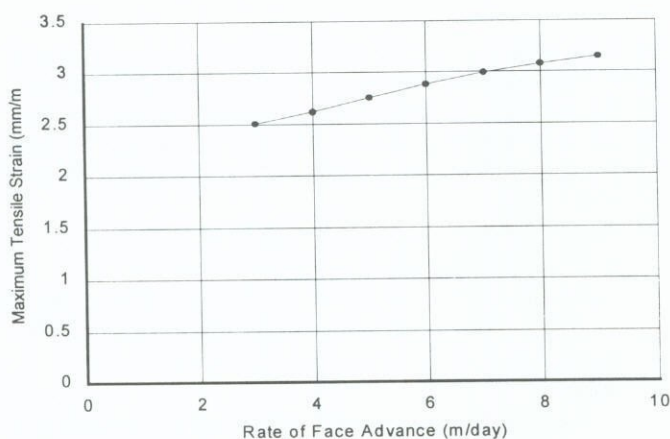


Figure 12: Relationship Between Tensile Strain and Rate of Advance

has been hinted at based on European data. The present analysis quantifies a special case – interaction subsidence measured along a travelling profile – but it could have important ramifications for working modern, rapid retreating faces beneath aquifers or bodies of water.

Unfortunately, translating a travelling profile into a transverse profile is not an easy task – in fact, the two profile shapes and characteristics are very different. – so it is not possible to determine the effects of the rate of advance on tensile strains experienced over the sides of the extraction monitored here. Intuitively, though, it might be expected that the ribside strains would be greater than the travelling strains measured here, and that the change in tensile strain with rate of face advance would be consequently much larger.

CONCLUSIONS

The following two simple conclusions can be drawn:

- Neural networks appear to present an alternate method of analyzing geotechnical monitoring data which presents complex interactions between the independent variables.

- Using neural network analysis, a proportionality between rate of face advance and maximum tensile strain obtained has been determined for the data set considered.

RECOMMENDATIONS

In order for this work to progress, a suitable data set must either be found or obtained by monitoring to allow the effect of rate of face advance on rib side strain development.

This is no easy task. The volume of data required for an accurate analysis would be prohibitively expensive to obtain by traditional survey leveling methods, requiring a large number of individual monuments over a panel edge, monitored on a daily, or shift by shift basis, and tied in to accurate face locations.

ACKNOWLEDGMENTS

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This paper is presented by permission of the Cape Breton Development Corporation. Any opinions expressed by the authors are their own and may not represent the views of CANMET or CBDC.

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Analysis of Panel Stability for Post-Mining Slurry Injection

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ABSTRACT

Slurry injection into mined-out panels is an attractive alternative for disposal of coal preparation plant fines. However, in areas where surface subsidence cannot be tolerated, either by law or necessity to protect surface structures or aquifers, special consideration must be given to the effect of slurry injection on panel stability. Slurry injection can lead to panel failure and unplanned subsidence by degrading roof, pillar, and floor strength. Water degradation is of particular concern where clay is present in the floor.

Recently, unplanned subsidence related to slurry injection occurred at an Indiana room-and-pillar coal mine subject to weak floor conditions. This paper discusses the efforts of the mine operator to determine the causes of the panel failure and to design panels and injection procedures to allow for future economic slurry injection without subsidence. The numerical and empirical/analytical techniques applied give the operator a rational approach to pillar panel design for future injection panels under various cover depths and weak floor conditions.

INTRODUCTION

Underground disposal of refuse from coal preparation plant fines by slurry injection was recently implemented by Solar Sources Underground, LLC, at its Monroe City Mine near Monroe City, Indiana. The Monroe City Mine is a room-and-pillar operation that recovers steam coal from the Danville Seam of the Illinois Coal Basin. No pillar retreat is performed, as the State of Indiana does not allow planned subsidence. Rock partings in the seam, combined with a weak carbonaceous shale roof that is partially mined with the coal, result in an average mining height of 5.5 ft (about 1 ft greater than the composite coal thickness), and a run-of-mine product that must be washed on-site. Included in the coal preparation plant reject is a significant quantity of fines (estimated up to 22% of plant feed) that must either be dewatered and disposed

of on the surface or disposed of underground by injection as a slurry. Underground slurry injection is the preferred method of fines disposal, provided that it can be accomplished without significantly impacting underground mining cost, productivity, reserve recovery, or pillar stability. The advantages of slurry injection include...

- Preparation plant costs and downtime are reduced by eliminating the need to dewater fines refuse.
- Fines refuse can be pumped rather than hauled by trucks to the disposal site, resulting in reduced transportation costs.
- The size and/or number of surface disposal areas are reduced.
- Eliminating the fines from the surface refuse disposal area provides for a significantly more stable landfill.
- Underground disposal of fines refuse is more environmentally sound because the slurry does not negatively impact the groundwater system, and is isolated from the surface environment.

For these reasons, a slurry injection system was designed by Solar Sources to replace the pre-existing fines dewatering and surface disposal system. Mining of the initial slurry injection panel (West Mains #1 North panel, see Figure 1) was completed in early 1997, and slurry injection began soon after. With the exception of some pump problems, the system functioned well for about two months. In late summer, however, the system failed, and surface cracks were found above the northern half of the slurry injection area. In response, Solar Sources decided to suspend slurry injection until they determined (1) the cause of the failure, and (2) if practical modifications could be made to the system to reduce the probability of future failures.

To determine the cause of the failure, Solar Sources retained NSA Engineering Inc., to perform a review of the geotechnical setting, injection practices, and the sequence of events leading to the failure of the injection system. NSA used numerical and empirical/analytical analysis techniques to arrive at modified

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slurry injection practices designed to reduce the likelihood of future system failures.

CAUSES OF THE SLURRY INJECTION SYSTEM FAILURE

After reviewing the geotechnical setting of the Monroe City Mine (particularly the West Mains #1 North injection panel), injection practices, and the sequence of events preceding failure of the injection system, NSA concluded that loss of floor/pillar integrity in the north end of the West Mains #1 North injection panel was the most likely cause of the injection system failure. This bottom/pillar failure was a direct result of saturation of the underclay bottom with the slurry water in the entries and crosscuts, weakening the material and structural properties of the pillar support system to the point it could not support the weight of the overburden. Catastrophic convergence propagated throughout the northern

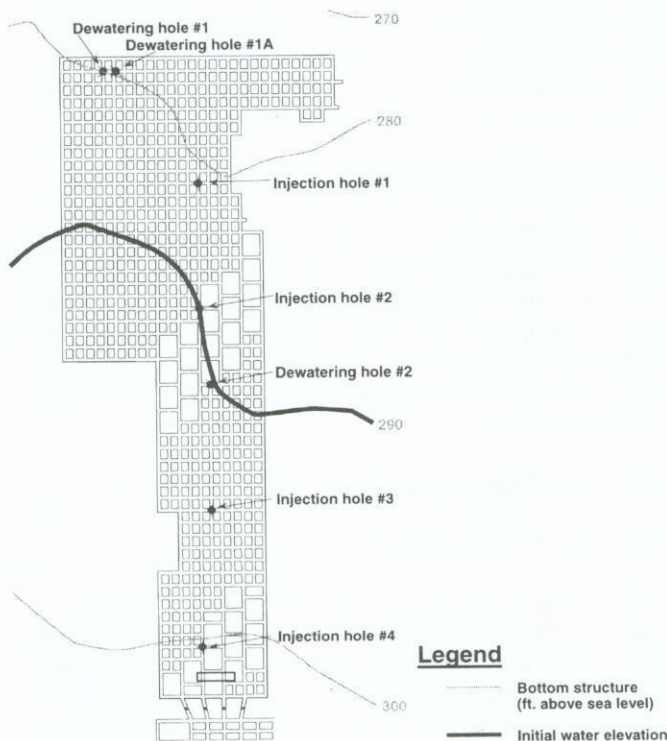


Figure 1. West Mains #1 North initial injection system.

end of the injection panel, stopping at the solid coal boundaries of the panel to the west, north, and east, and at the larger pillars left to support a surface gas line to the south (see Figure 1). The convergence of the roof onto the stowed solids from the slurry blocked the flow of new slurry from the injection wells to the recovery well, causing water to back up and breach the West Mains #1 North seals. This explanation is supported by the following evidence:

- Surface cracks typically found at the perimeter of subsidence areas were found directly above most of the perimeter of the area described above. Initial observation of these cracks occurred just one day before failure of the #1 dewatering hole. The pump in this hole “hung” when retrieval was attempted, indicating ground shifting, which is typical in and near subsided areas.
- When dewatering hole #1 (see Figure 1) was drilled, overburden strata were generally intact. When a replacement dewatering hole (#1A) was drilled, the driller noted the rock strata were broken along their entire length--another indication of subsidence.
- Water elevation data for the original dewatering borehole indicate most of the mine floor at the north end of the injection panel was under water for almost two months prior to the failure of the injection system, allowing the underclay in the entries and crosscuts to become saturated. Slake durability tests performed on the underclay indicated that it degrades significantly when exposed to water, and the original pillar design was based on the physical properties of dry underclay bottom.
- A camera borehole survey conducted following failure of the injection system indicated that slurry (fines) had been forced 4 and 6 ft up into dewatering hole #2 and injection well #2, respectively, indicating a compression of the slurry as would be expected with convergence.

DESIGN OF MODIFIED SLURRY INJECTION METHODS

To determine likely failure mechanisms associated with the West Mains #1 North subsidence event, and to develop a pillar design that minimizes the likelihood of future subsidence, NSA applied numerical and empirical/analytical analysis techniques. The primary tools used in the analyses were the boundary-element numerical modeling code Mulsim/NL, the ARMPS pillar stability program, and Vesic-Speck ultimate bearing capacity (UBC) calculations. The basic steps in the analysis were as follows:

- 1) Develop proper input parameters to Mulsim/NL numerical models to simulate loading and observed ground responses for both the dry (pre-injection) and wet (post-injection) conditions for West Mains #1 North.
- 2) Using the input parameters developed in step 1, determine a suitable pillar size for future injection panels using Mulsim/NL and ARMPS, based on wet conditions and the maximum cover depth.
- 3) Perform UBC calculations and compare with Mulsim/NL pillar loadings to determine floor bearing capacity, anticipated pillar loads, and floor factors of safety for various pillar sizes.

This approach was developed to give Solar Sources a rational means of pillar panel design for future injection panels under various cover depths and weak floor strata conditions.



Analysis Tools

MULSIM/NL

The primary tool used in the analysis was MULSIM/NL, a boundary-element method code developed by the U.S. Bureau of Mines (USBM) (Zipf, 1992). MULSIM/NL is used to model stress, displacement, and yielding within tabular deposits such as coal. The distinguishing feature of MULSIM/NL is its ability to handle **in-seam** materials based on both linear and nonlinear mechanical (stress-strain) behavior. The geometry of the model is variable; thus, incremental model changes can be used to determine optimal mining sequences, as well as the impact of varying pillar sizes and seam interaction effects. MULSIM/NL iteratively solves a set of equations representing the stress-strain state of each element in a grid until equilibrium is reached (loads have stopped transferring).

Following a confined core approach to pillar strength (Karabin, 1994), element properties can be arranged so that the weakest elements are adjacent to the mine opening, with element strengths increasing into the pillar core. Using strain-softening elements with increasing peak and residual strengths toward the core, elastic-plastic behaviors observed in pillar cores are approximated, and correspondence with classical empirical pillar design methods is achieved. The effects of width-to-height ratio are also accurately accounted for.

NSA uses a modified version of the basic MULSIM/NL code. This modified code includes the ability to model the effects of varying overburden, and an element birth capability to allow sequential mining and support introduction (standing supports or backfill) to be modeled more effectively. In addition, NSA has developed a post-processor that presents MULSIM/NL results three-dimensionally. The height of each element in the model gives the stress in that element relative to other elements, and the color of each element shows either (1) how close the element is to yielding, (2) that it is in the process of yielding, or (3) that it has completely yielded to its residual strength. By comparing different areas within a model, and one model to another, the plots give a very clear indication of relative stability.

ARMPS

As a crosscheck with MULSIM/NL results, the ARMPS (Analysis of Retreat Mining Pillar Stability) program was used. Stability factors calculated using ARMPS (Mark, 1995), a USBM pillar design program, are useful because they can be directly related to a case history database that includes over 110 pillar retreat sections. These case histories cover an extensive range of geologic conditions, caveability characteristics, extraction methods, cover depth, and mining geometries.

The ARMPS program performs three basic calculation steps. First, the estimated load-bearing capacities of the pillars within the active mining zone (AMZ) are calculated using the Mark-Bieniawski elongated pillar strength formula. The AMZ extends from the face a distance equal to five times the square root of the mining depth. Secondly, the loading that pillars within the AMZ are subjected to is estimated for all phases of development and, if applicable, retreat. Finally, stability factors are calculated by dividing the load-bearing capacity by the estimated loading. Through back analysis of pillar recovery case histories, it has been found that designs with ARMPS stability factors greater than 1.5 are usually successful (94%), whereas designs with stability factors less than 0.75 are not (92%). Stability factors between 0.75 and 1.5 produce mixed results, where both adequate and inadequate designs can be found.

Through numerous case studies, NSA has found that ARMPS provides a good starting point for evaluating pillar panel stability, with more refined results achieved through MULSIM/NL modeling. By comparing MULSIM/NL and ARMPS results, and correlating those results with underground conditions, mine operators can get a very good feel for the stability of planned developments.

Vesic-Speck UBC Calculations

The Vesic-Speck approach estimates the ultimate bearing capacity of the floor strata by considering the floor to be a two-layer cohesive soil or rock system (Pytel), with the weak layer overlying a stronger foundation, and the angle of internal friction (ϕ) equal to zero for both layers. The cohesion of the weak layer is estimated from an empirical relationship based on the moisture content of the layer.

Analysis Assumptions and Results

West Mains #1 North Failure

West Mains #1 North is divided into an inby and outby portion by 80- by 100-ft pillars left to protect a gas line on the surface (Figure 1). The inby portion is nominally a 17-entry system, with entries on 50-ft centers and crosscuts on 60-ft centers. Entry width is 20 ft, for a total panel width of 820 ft. The outby portion is nominally a 10-entry system with the same pillar dimensions, for a total panel width of 520 ft. In addition, the mining height in the inby portion of the panel is 6.5 ft due to an increased parting thickness inby (compared to the outby mining height of 5.5 ft).

The massive panel failure in the inby portions of the panel was confirmed by the surface subsidence cracks coinciding with the inby panel periphery and the slurry injection operation record. Prior to the failure, inby portions of West Mains #1 North were completely submerged by slurry water. The most likely scenario is that the slurry water caused significant weakening of the coal pillars and the underclay floor, leading

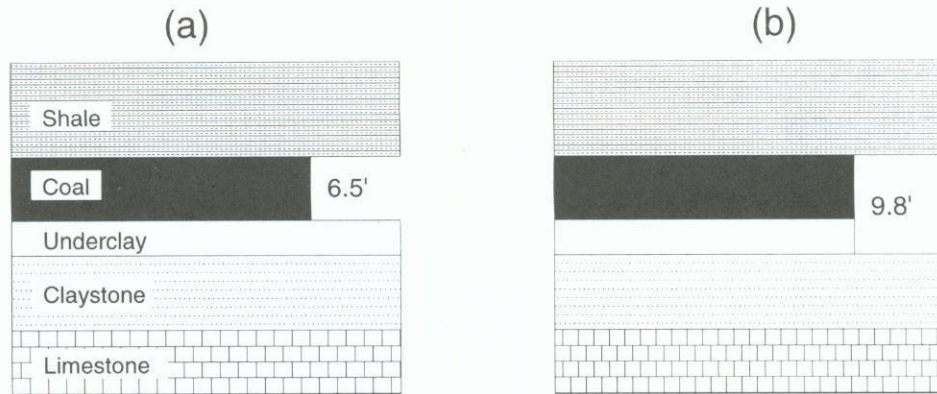


Figure 2. Hypothesized pillar geometry prior to (a) and following (b) slurry injection.

to panel failure and surface subsidence. After the failure, the slurry water in the inby portion of the panel was forced into outby portions of the panel. However, there has been no evidence to date of failure in the outby portion of the panel. This implies that inby portions of the panel failed due to a combination of (1) slightly higher mining height (and therefore lower width-to-height ratio for the pillars), (2) slightly greater cover depth, and (3) weakening of the coal pillars and the underclay floor by slurry water infiltration.

In any ground stability analysis, it is advantageous to be able to compare conditions that led to a failure with those for which no failure has occurred. In West Mains #1 North, the failure of inby portions of the panel versus the stability of outby portions gives a reference point to which numerical modeling can be calibrated. The goal of the MULSIM/NL analysis, therefore, was to determine the input properties for saturated conditions that result in failure inby and stability outby.

NSA approached the problem by assuming that the slurry injection water weakened the underclay in the entries to an extent that they no longer provided confinement to the pillars. This lowers the support capacity of the pillars in two ways (Figure 2). First, loss of floor confinement increases the effective pillar height by the underclay thickness. What was originally a 6.5-ft pillar behaves as a 9.8-ft pillar due to the underclay becoming, in effect, part of the pillar. Secondly, the physical strength of the pillar is diminished, because it is now composed of both stronger coal and weaker underclay. To simulate these effects in the MULSIM/NL analysis, the mining height was increased by 3.3 ft (the average underclay thickness), and a proper composite pillar strength was determined by running several models with progressively weaker pillar strengths until the model results simulated panel failure approximating the known failure of West Mains #1 North.

In the West Mains #1 North analysis, MULSIM/NL models were created using a 150 by 150 element grid and 10-ft elements for a modeled area of 1,500 ft wide by 1,500 ft long. To model the entire panel, separate models for inby and outby were run, and the results were seamed together. The actual

cover depth (170 to 220 ft) for the modeled area was accounted for. First, dry conditions were simulated. The geomechanical parameters for this model are shown in Table 1:

Table 1. MULSIM/NL input parameters prior to slurry injection.

	<u>Inby</u>	<u>Outby</u>
Mining height, ft	6.5	5.5
Pillar centers, ft	50 by 60	50 by 60
UCS of coal, psi	2,000	2,000
Elastic modulus of coal, psi	280,000	280,000
Poisson's ratio of coal	0.35	0.35
Elastic modulus of roof/floor, psi	600,000	600,000
Poisson's ratio of roof/floor	0.25	0.25

MULSIM/NL results showing loading conditions before slurry injection are presented in Figure 3. The results show that stability is maintained throughout inby and outby portions of the panel. All pillars are loaded to less than 80% of their strength. These results correspond to observed conditions, i.e., the West Mains #1 North panel was stable prior to slurry injection.

The next step was to modify input to the model to simulate conditions after slurry injection. As previously described, the mining height was increased by the average underclay thickness (3.3 ft.) over the modeled area, and the UCS of coal was stepped down until model results corresponded with the observed West Mains #1 North panel failure. Input parameters to these models are given in Table 2.

Table 2. MULSIM/NL input parameters after slurry injection.

	<u>Inby</u>	<u>Outby</u>
Mining height, ft	9.8	8.8
Pillar centers, ft	50 by 60	50 by 60
UCS of coal, psi	1,350	1,350
Elastic modulus of coal, psi	280,000	280,000
Poisson's ratio of coal	0.35	0.35
Elastic modulus of roof/floor	600,000	600,000
Poisson's ratio of roof/floor	0.25	0.25

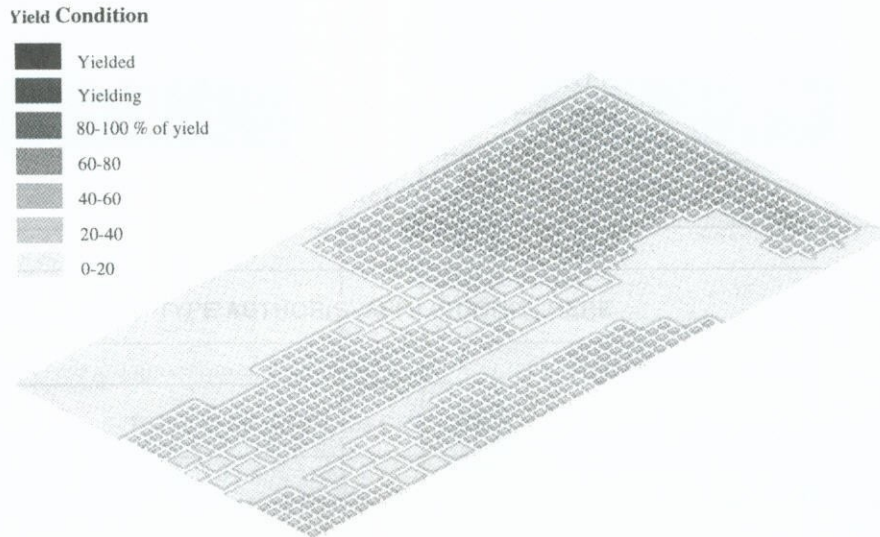


Figure 3. MULSIM/NL results showing loading conditions in West Mains #1 North prior to slurry injection.

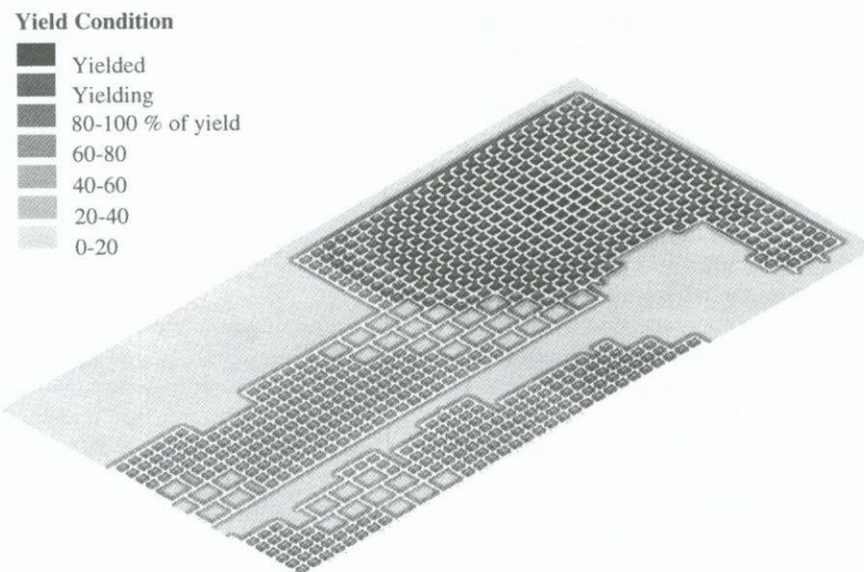


Figure 4. MULSIM/NL results showing loading conditions in West Mains #1 North after slurry injection.

Modeled conditions after slurry injection are shown in Figure 4. The result shows massive pillar failure in inby portions of the panel, and stable pillars in outby portions (outby the large pillars left to protect the gas line). These results match observed conditions, namely, failure of the inby portions of the panel as indicated by surface subsidence cracks, and stability of

the outby portions of the panel as indicated by borehole camera observations. With the MULSIM/NL models properly calibrated (using the input parameters given in Table 2 above), a parametric analysis of pillar size was performed.

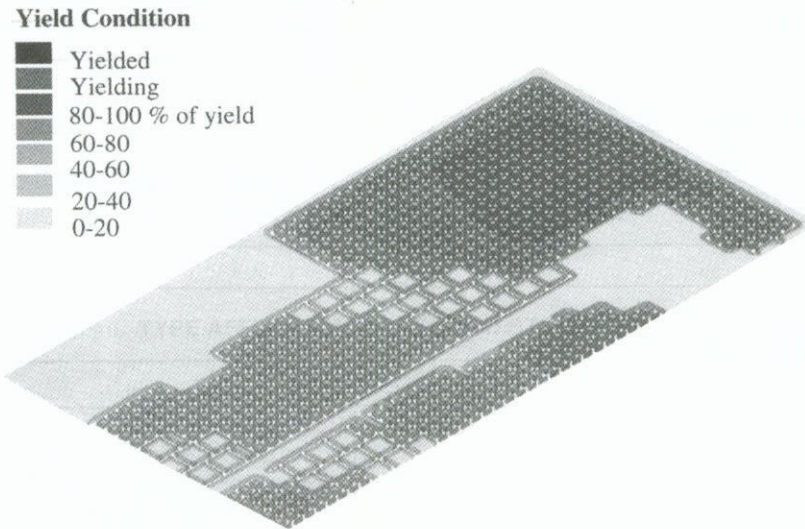


Figure 5. Mulsim/NL results showing loading conditions in West Mains #1 North as if it had been developed with 40- by 40-ft pillars, and after injection.

West Mains #1 North with 40- by 40-ft Pillars

To study the effects of increased pillar size on panel stability, West Mains #1 North was modeled as if it had been developed using 40- by 40-ft pillars (60- by 60-ft centers), using the saturated condition input parameters given in Table 2. The result is presented in Figure 5. Although some rib yielding has occurred in the inby portions of the panel under the deepest cover, all pillar cores remain stable, loaded to less than 60% of their strength. These results indicate that by increasing pillar size, future panels can be designed to remain stable after slurry injection, with pillar size primarily dependent upon maximum cover depth (assuming mining height remains constant).

Pillar Design for Future Injection Panels

The next step in the design process involved simulating the stability of various pillar layouts under a constant maximum (worst case) cover depth for future slurry injection panels adjacent to West Mains #1 North. Four different pillar layouts were considered: 30- by 40-ft, 40- by 40-ft, 40- by 50-ft, and 40- by 60-ft pillars in a nine-entry system, all at a constant depth of 210 ft. ARMPS stability factors (SF) were calculated for each configuration to establish a relationship between the Mulsim/NL and ARMPS results and to determine a minimum ARMPS stability criterion for future design work. For added detail, 5-ft elements were used in the Mulsim/NL analysis using a 140- by 95- element grid for a modeled area of

700 ft wide by 475 ft long. Input parameters after slurry injection are the same as those given in Table 2.

Results of the Mulsim/NL and ARMPS analyses are summarized in Table 3.

Table 3. Mulsim/NL and ARMPS results for various pillar sizes at 210 ft of cover.

	Mulsim/NL	ARMPS SF	
	Stability (wet)	(wet)	(dry)
30- x 40-ft Pillar	Unstable	1.09	2.27
40- by 40-ft Pillar	Stable	1.36	2.89
40- by 50-ft Pillar	Stable	1.56	3.34
40- by 60-ft Pillar	Stable	1.72	3.68

Based on the Mulsim/NL and ARMPS results, a 40- by 40-ft pillar was recommended to Solar Sources for future injection panels adjacent to West Mains #1 North, under cover depths of less than 210 ft. Comparing Mulsim/NL and ARMPS results, NSA recommended that future injection panels be designed to maintain an ARMPS stability factor of at least 1.3, using an increased mining height to account for the thickness of the underclay, and an in situ coal strength of 320 psi to account for the weakened pillar composed of both coal and underclay.

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ARMPS stability factors were then calculated for each pillar size to determine the maximum depth appropriate for each configuration when slurry injection is practiced. Results are shown in Table 4.

Table 4. ARMPS stability factors at various depths for the pillar sizes studied¹.

	180 ft	220 ft	250 ft	280 ft
30- x 40-ft Pillar	1.27	1.04	0.91	0.82
40- x 40-ft Pillar	1.59	1.30	1.14	1.02
40- x 50-ft Pillar	1.82	1.49	1.31	1.17
40- x 60-ft Pillar	2.00	1.64	1.44	1.29

¹Nine-entry, wet.

Ultimate Bearing Capacity Calculations

As a check of the MULSIM/NL/ARMPS design approach, ultimate bearing capacity (UBC) calculations were performed using the Vesic-Speck approach. The immediate floor properties used in the analysis (based on information provided by Solar Sources) are given in Table 5:

Table 5.- Input parameters to UBC calculations.

Strata	Thickness, ft	UCS, psi	Moisture, %
Underclay	3.3	153	8.11
Claystone	7.4	579	6.43
Limestone	6.4	3776	5.05

In the Vesic-Speck approach, the strength of the weak floor member is related empirically to its inherent moisture content. For this analysis, both the underclay and the claystone were considered weak layers lying on top of the more competent limestone. Underclay and claystone were combined, therefore, with a total thickness of 10.7 ft and a weighted moisture content of 6.95%. Ultimate bearing capacity is affected by the thickness of the weak floor members, inherent moisture content of the weak strata, and pillar dimensions (square pillars giving the highest bearing capacity). Figure 6 shows the variation in UBC with each of these parameters. Because inherent moisture content and weak strata thickness have the most dramatic effect on UBC, NSA recommended that Solar Sources monitor these parameters as future slurry injection panels are developed to expected.

As an example of this procedure, bearing capacities were calculated for the floor conditions given in Table 5 for the various pillar sizes considered. Bearing capacities were then divided by average core stresses determined from MULSIM/NL analyses at 210 ft of cover to arrive at a bearing capacity safety factor. Results are summarized in Table 6.

Table 6.- Bearing capacity safety factors for various pillar sizes at 210 ft of cover and average floor conditions.

Pillar size, ft	UBC, psi	Average core stress, psi	Bearing capacity safety factor
30 by 40	784	893	0.88
40 by 40	834	621	1.34
40 by 50	817	542	1.51
40 by 60	807	495	1.63

The bearing capacity calculations show the inadequacy of the 30- by 40-ft design at 210 ft of depth, whereas 40- by 40-ft and larger pillars should remain stable.

Operational Considerations and Procedures for Slurry Injection

In addition to modifying pillar design in slurry injection areas, NSA recommended operational procedures to Solar Sources to develop a slurry injection system that maximizes the

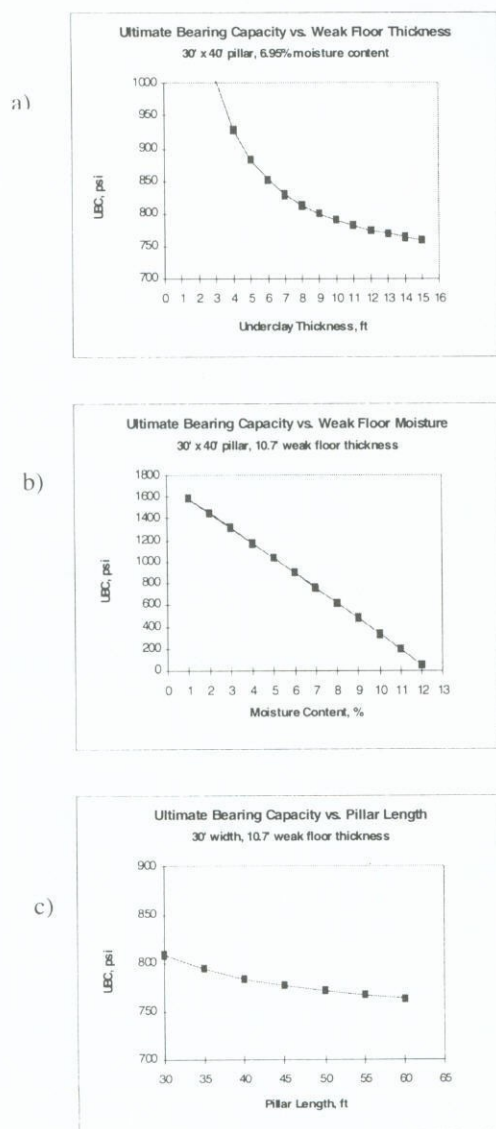


Figure 6. Variation in ultimate bearing capacity with a) floor strata thickness, b) floor strata moisture, and c) pillar length.

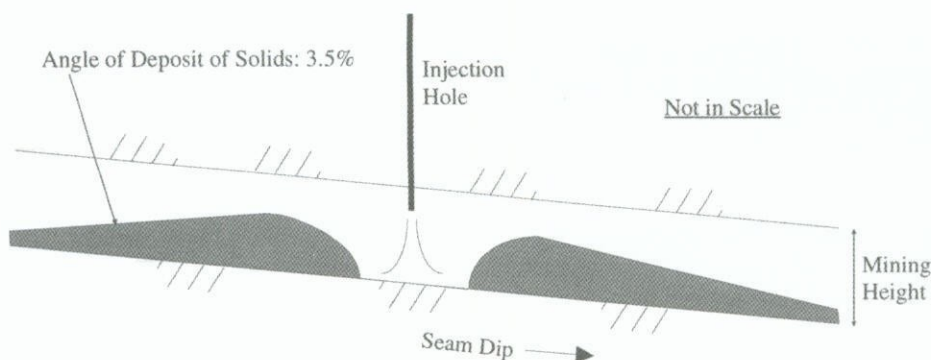


Figure 7. Typical profile of solids deposited at the bottom of a slurry injection borehole.

amount of slurry that can be stowed using a minimum number of injection holes. To understand the components of this system, an understanding of how the slurry behaves after injection into the mine opening is helpful.

Most slurries generated by coal preparation plant thickeners have a 25% to 30% solids-to-water ratio. Due to the pumping velocity, the water and solids remain reasonably mixed while being piped to the injection area. Once the slurry reaches the injection area, the velocity of the slurry is significantly reduced, and the solids start to separate from the water, forming a delta deposit. The coarse fraction of the solids separate first, and the finer fractions are carried farther from the injection hole. The area immediately surrounding the injection hole remains clear of solids due to the flushing mechanisms created by the high velocity of the slurry entering the injection area. A typical profile is shown in Figure 7. The life of a slurry injection hole is therefore a function of the distance of the slurry injection hole from the water recovery hole; the dip of the seam between the injection and recovery holes, the angle of deposit of the solid material, and the size fraction distribution of the solids that make up the slurry.

To facilitate successful slurry injection, a slurry injection area should have the following characteristics:

- The area should be sealed, and all areas down dip from the seals should be separated from active workings by a solid coal barrier to avoid mine inundation.
- To maximize the life of slurry injection holes, areas with maximum dip should be given first consideration for slurry injection. Likewise, areas with minimum overburden should be given first consideration to minimize pillar size requirements and to maximize reserve recovery.
- Flat areas, where the direction of water and solids flow is questionable, should be avoided.
- Areas that have swags large enough that the water will pool to the roof prior to reaching the water recovery area should be avoided.

To maximize the utilization of slurry injection holes and water recovery holes, the following procedures should be followed:

- To limit the areal extent of underclay subject to pooled water (and therefore degradation), the water recovery hole should be at the lowest elevation of the injection area, and only minimal amounts of water (enough to keep the recovery pump submerged) should be allowed to accumulate in the recovery area.
- The first injection hole should be at the highest elevation farthest from the recovery hole so that the delta it creates will not interfere with future injection hole slurry flow.
- Once injection is started in a hole, that hole should be used until it plugs or black water comes out of the recovery hole.
- Additional holes in an injection area should be progressively down dip and closer to the recovery hole.
- Spacing of holes will be a function of the bottom grade and the size composition of the slurry. A reasonable starting point for spacing is 750 feet if the holes are in line with seam dip, and 400 feet if the holes are equal in elevation or in an area slurry has not flowed through.
- A spare slurry injection hole should always be available and connected to the system to minimize downtime when plugging occurs.

SUMMARY

Through changes in pillar design and operational procedures, Solar Sources is now prepared to resume slurry injection at its Monroe City Mine with the expectation that future system failures will be avoided. Although changes in the mine plan have postponed slurry injection operations for the near future, the numerical and empirical/analytical techniques applied during the course of this study have given Solar Sources a rational approach to pillar panel design for future injection panels under various cover depths and weak floor conditions.

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Development of Time-dependent Surface Subsidence Over the Total Extractions Areas of the Potash Beds Stopped with Interchamber Yield Pillars

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ABSTRACT

The method of predicting the time-dependent surface subsidence for gently-inclined potash deposits where total extraction using various sizes of yielding barrier pillars was discussed.

PROBLEM DEFINITION

When mining gently-inclined potash beds the remnant barrier pillars may have different yielding behavior. The duration of surface subsidence process may last from one/two to several hundred years. Therefore a reliable prediction of time-dependent surface subsidence is an important subject in potash mining.

OBJECTIVES

Surface subsidence above the area of total extraction is one of the major parameters that determine the movement characteristics of the subsidence basin because those characteristics grow with time as stope mining progresses.

This research involves the development of a method for predicting surface subsidence at any time after its initiation when mining a single potash seam with yield pillars without backfilling the rooms.

METHODOLOGY

The methods of investigation for the development of methods for surface subsidence prediction are

1. Annual surveys after initiation of surface subsidence using various yield pillars (1,2,3).

2. Periodical visual observation on the characteristics of deformed pillars and rooms (4).
3. Measurements of stresses in yield pillars of various sizes, as they are being deformed (5).
4. Laboratory tests of samples and models of potash pillars under various modes of loadings and deformations (6,7).

Data analyses were performed using the theory of ultimate balance for bound media and rheological properties of saliferous rocks (8,9,10).

RESULTS

The investigations performed have established:

1. There are two types of time-dependent surface subsidence $\eta(t)$, when yielding pillars are employed.
2. Criteria for classifying the type of time-dependent surface subsidence are listed in Table 1.

Table 1. Criteria for classification of time-dependent surface subsidence $\eta(t)$.

Type	Criteria
1	$C \leq 0.3 : e^* > e_f$
2	$C > 0.3 : e^* < e_f$

where C : degree of pillar loading
 e^* : limiting deformation of pillars
 e_f : final deformation of pillars

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3. Types and stages of time-dependent surface subsidence are shown in Fig. 1.
4. Pillar dimension v. relative deformation at stage boundaries is shown in Fig. 2.

5. Degree of pillar loading v. rate of relative deformations at stage boundaries is shown in Fig. 3.

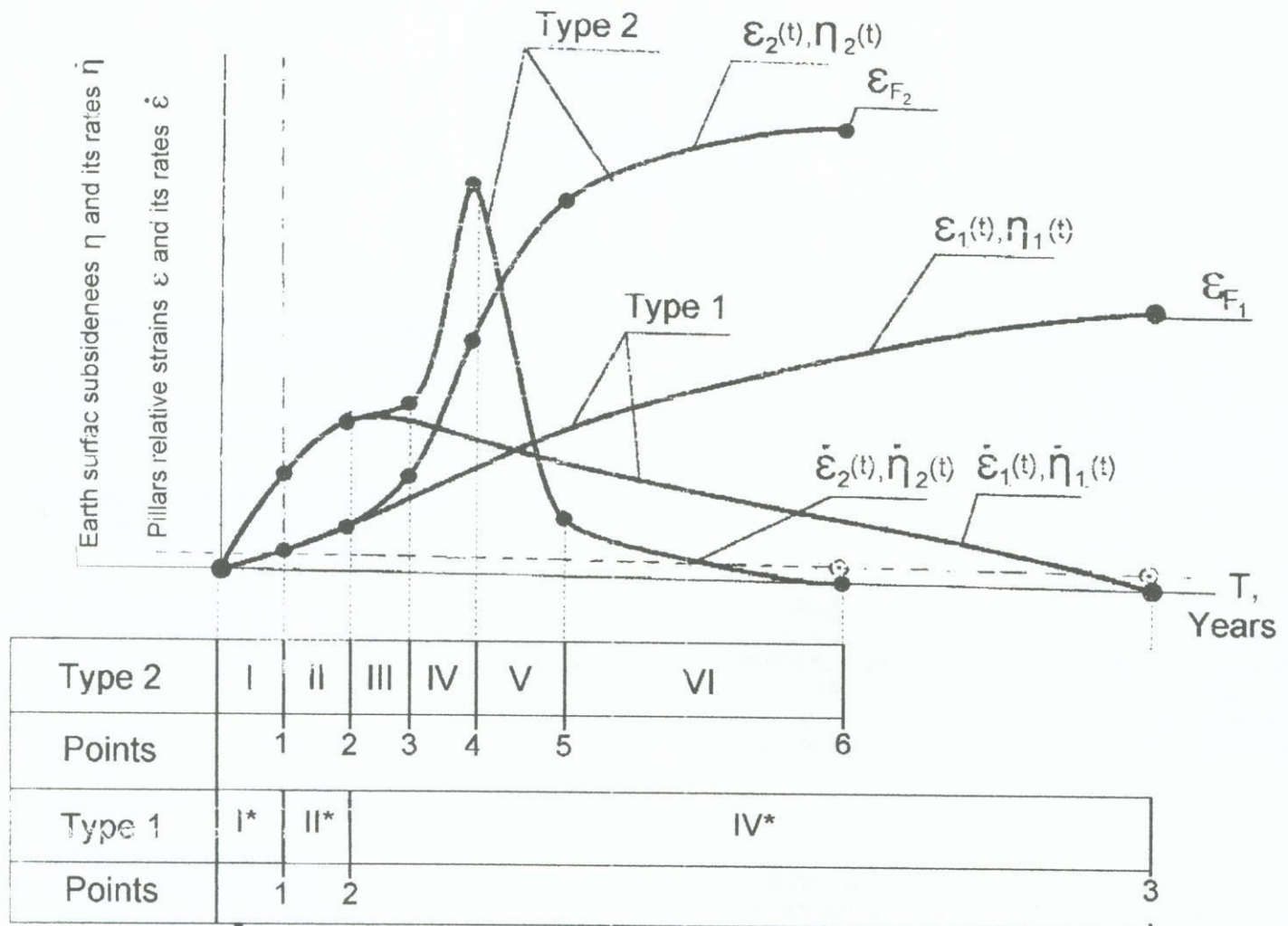


Figure 1. Types, stages, and characteristic points of time-dependent surface subsidence, $\eta(t)$ and pillars relative deformations, $\epsilon(t)$ with time.

- I, II* - Subsidence that initiates prior to the approach of stope mining front.
- II, II* - Subsidence caused by the growth in size of the mined-out space (gob).
- III, III* - Subsidence caused by the growth in pillars' height after roof falls in the rooms have occurred.
- IV* - Subsidence caused by the decrease in pillars' height and increase in their width as they deform.
- IV - Subsidence caused by progressive creep of pillars.
- V - Subsidence caused by compaction of broken pillars.
- VI - Subsidence caused by elimination of cavities left in the arched room and between larger lumps.

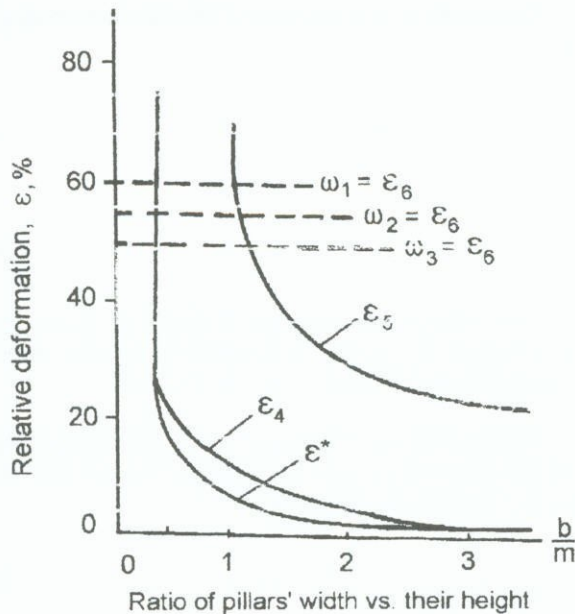


Figure 2. Relationships associating the ratio of pillar width to height with relative deformations at stage boundaries: (for the second type $\eta(t)$).

- ϵ^* - boundary between third and fourth stages,
- ϵ_4 - boundary between fourth and fifth stages,
- ϵ_5 - boundary between fifth and sixth stages,
- $\omega_1, \omega_2, \omega_3$ - various degrees of ore extraction from the seam.

CHARACTERISTICS OF SUBSIDENCE STAGES

The first two stages shown in Figure 1 are common to both types of time-dependent surface subsidence because they are caused by similar mechanisms of pillar deformation and their interaction with the surrounding rocks.

In the 1st stage subsidence is created at the instant when rooms and pillars are formed.

In the 2nd stage subsidence is due to the increase of loading on chain pillars as the gob dimension grows and/or due to reduction in pillar strength resulting from roof falls that increase pillar height.

In the 3rd stage subsidence is in response to deformation of pillars under full overburden weight. It continues until roof caving stops. If pillar loading does not exceed the long-term strength, C^* , this is the first type of time-dependent surface subsidence. If $C_{\max} > C^*$,

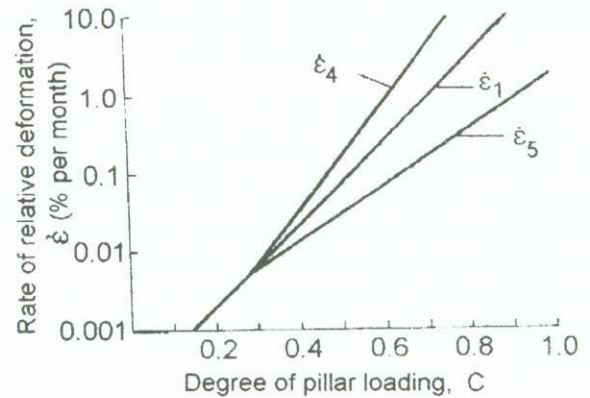


Figure 3. Degree of pillar loading v. rates of relative deformations, ϵ at the boundaries of various subsidence stages (for the second type $\eta(t)$):

- $\dot{\epsilon}_2$ - at the boundary between second and third, as well as third and fourth stages,
- $\dot{\epsilon}_4$ - at the boundary between fourth and fifth stages,
- $\dot{\epsilon}_5$ - at the boundary between fifth and sixth stages.

then the 3rd stage goes on until the relative deformation of pillar reaches an ultimate value, ϵ^* . This is the 2nd type of time-dependent surface subsidence.

In the 4th stage, the rate of pillar deformation decreases gradually for the 1st type of surface subsidence. It is caused by a continuous increase of strength as the ratio of width to height grows. In this case the 4th stage will go on until the end of the subsidence process.

Conversely in the case of the 2nd type of time-dependent surface subsidence the rate of pillar deformation increases at the 4th stage. It is caused by the progressive creep with relative deformation exceeding the ultimate values. This stage ends with pillar strength reduced to the minimum residual values.

In the 5th stage existing only for the 2nd type of time-dependent surface subsidence rock mass strengthens. By the end of this stage, the rock mass strength returns to the initial values (4).

In the 6th stage, the remaining cavities at the roof and between rock blocks are eliminated.

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PREDICTION OF TIME-DEPENDENT SURFACE SUBSIDENCE OVER TOTAL EXTRACTION AREAS AND ITS APPLICATION

In the situation when the immediate roof in the rooms is stable, subsidence prediction becomes very simple. The 3rd stage of the 1st type no longer exists. As a result the prediction reduces to the determination of the coordinates of three characteristic points and η intermediate points between the 2nd and 3rd characteristic points. In the case of the 2nd type of surface subsidence, there are six characteristic points (see Fig. 1).

The coordinates of the first characteristic point:

$$t_1 = \frac{H \cot \delta}{V} \quad (1)$$

$$\eta_1 = t_1 \frac{\dot{\epsilon}_2 m}{2} S(Z) = 0.25 \frac{H \cot \delta}{V} \dot{\epsilon}_2 m_o \quad (2)$$

Where:

δ - boundary angle
H - depth
V - rate of advancing stoving front

The coordinate of the second characteristic point:

$$t_2 = H \frac{\cot \delta + \cot \psi}{V} \quad (3)$$

$$\eta_2 = \eta_1 + 0.75 \frac{H \cot \psi}{V} m_o \dot{\epsilon}_2 \quad (4)$$

Where:

ψ - angle of full (complete) movements
 m_o - initial height of pillars

The coordinate of the third characteristic point:

$$t_3 = t_2 + \frac{\eta_3 - \eta_2}{m_o \dot{\epsilon}_2} \quad (5)$$

$$\eta_3 = \eta_2 + m \epsilon^* \quad (6)$$

Ordinates of the fourth and the fifth characteristic point:

$$\eta_4 = \eta_1 + m_o \epsilon_4 \quad (7)$$

$$\eta_5 = \eta_1 + m_o \epsilon_5 \quad (8)$$

The relative deformations of pillars ϵ^* , ϵ_4 and ϵ_5 for equations 6, 7, and 8 are determined by b/m , using the corresponding graphs shown in Figure 2.

The abscissas of the fourth and fifth characteristic points are determined using a nomogram such as that shown in Figure 4 by the formulas:

$$t_4 + t_3 + \Delta t_{3-4} \quad (9)$$

$$t_5 = t_4 + \Delta t_{4-5} \quad (10)$$

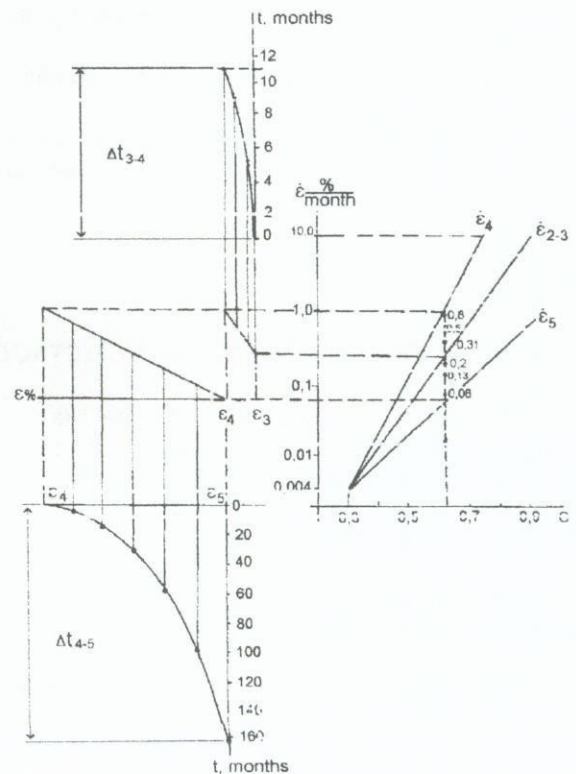


Figure 4. An example of the nomogram for determination of time intervals Δt_{3-4} and Δt_{4-5} .

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These nomograms are being constructed using the graphs, ϵ_2 , ϵ_4 , ϵ_5 , shown in Figure 3, and the values of relative deformations of pillars, ϵ_4 and ϵ_5 .

The degree of pillars loading, C, in these nomograms, are determined by using a special method based on the application of the following dimensionless parameters:

1. Mining depth vs. uniaxial strength of ore-bearing rock mass,
2. Distance between longitudinal axes of continuous rooms vs. pillar width,
3. Pillars' width vs. height,
4. Surrounding rocks adhesion vs. ore-bearing rocks, and
5. Angle of internal friction and angle of friction at the contact of pillars and surrounding rocks.

And lastly, the coordinate of the sixth characteristic point:

$$t_6 = t_5 + 2 \frac{\eta_6 - \eta_5}{m_o \epsilon_5} \quad (11)$$

$$\eta_6 = \frac{F_o}{a_o + b_o} \quad (12)$$

Where:

F_o , a_o , b_o - initial cross-sectional area of rooms, width of rooms, and width of pillars, respectively.

The rates of relative deformations of pillars, ϵ_2 and ϵ_5 for equations 2, 4, 5, and 7 are determined by using the graphs shown in Figure 3, depending on the degree of their loading, C.

PREDICTION OF SUBSIDENCE FOR A SPECIFIC PERIOD OF TIME

After having determined the coordinates for the six characteristic points for the two types of time-dependent surface subsidence, prediction of subsidence for a specific period of time requires the following tasks:

1. Plot the coordinates of those points on a graph and draw a smooth curve over them, and
2. Find the coordinates corresponding to the set period of time.

Figure 5 shows examples of predicted time-dependent surface subsidence. They were done using

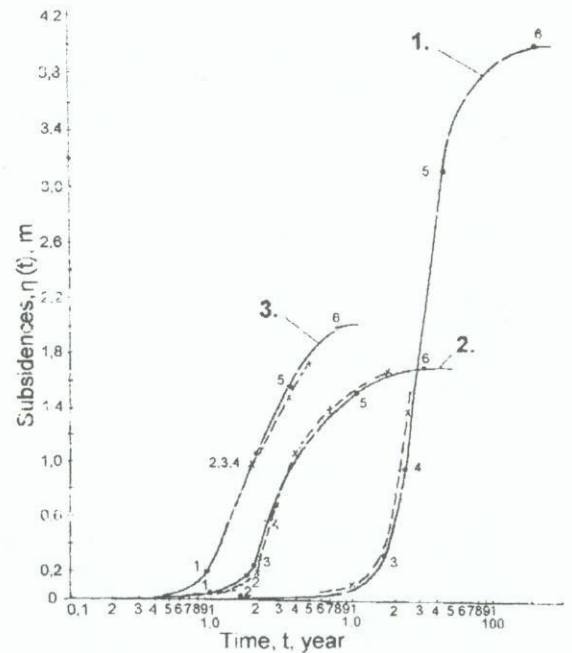


Figure 5. Comparison of the predicted and measured surface Subsidence, corresponding to pillars of various degrees of yielding (see Table 2 for corresponding conditions). Dashed line - measurements Solid lines - prediction

the above-mentioned methods for specific stopping conditions. For sake of comparison the measured subsidence are also shown.

Therefore the method of subsidence described in this paper is reliable as demonstrated by the wide ranging case conditions (Table 2).

Table 2. Conditions corresponding to examples shown in Figure 5.

Nos.	Mines	Seam depth H, m	Width		Height of pillars M, m
			Rooms a, m	Pillars b, m	
1.	B-1	210	14.9	7.0	7.4
2.	B-2	360	5.2	3.8	3.7
3.	B-2	360	6.1	2.9	3.7

Where:

B-1, B-2 - Berezniki Mines 1-2.

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CONCLUSIONS

1. The method described in this paper can predict surface subsidence due to total extraction of a single potash seam with yield pillars of wide ranging sizes.
2. This method is an empirical method. It is based on the principles common to yield pillars of saliferous rock masses. The techniques employs experimental data that will differ only quantitatively for stope mining in different rock masses. It is believed that it may be applicable to conditions other than the Upper Kama potash ore deposit.

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Transversely Elasto-Plastic Analysis of Surface Subsidence Arising from Longwall Mining of Inclined Coal Seams

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ABSTRACT

A Transversely Elasto-Plastic Model was configured in FLAC (Fast Lagrangian Analysis of Continua) as a more realistic simulation of stratified and inclined strata behaviour examining surface subsidence above the inclined longwall panels (1). The failure criterion employed was a modified Mohr-Coulomb failure criterion that assumed a variable cohesion, which changed with respect to the orientation of the plane of anisotropy. Using the proposed numerical modelling technique several longwall panels with different configurations were simulated. The results of both surface subsidence and horizontal displacements were validated against UK data using the Subsidence Engineer's Handbook (S.E.H) subsidence prediction method and the Influence Function Method (2). It was found from the research that the insitu properties as well as laboratory to field reduction factors are depth dependent. A set of relationships for calculation of anisotropic stiffness and strength parameters have been derived. The proposed modelling approach has been successfully applied to surface subsidence prediction for 3 collieries in the UK allowing the results of the simulation to be validated directly against measured data.

INTRODUCTION

Increasing demand for energy and mineral resources has resulted in the mechanisation and introduction of rapid excavation techniques in modern mining operations through out the world. This in turn has given rise to surface and subsurface subsidence problems. Surface subsidence prediction for inclined and steep seams has received less attention than more level seams due to the difficulties involved in the extraction of such coal seams. In some countries (e.g. China, India, Spain) a great deal of the coal reserves of high quality are classified as inclined and steep strata. For local economic reasons these deposits have to be extracted

and consequently the problem of surface subsidence from steep coal seam extraction by longwall method is highly relevant in many parts of the world. Numerical modelling techniques particularly the finite difference method is a powerful tool for obtaining solution to these problems. It differs from physical modelling techniques, surface profile functions and more basic mathematical methods in that it can take into account the mechanical properties of the overburden and analyse the failure and post failure behaviour of rock masses surrounding the mine openings in a reasonable time.

During the course of this research a Transversely Isotropic elasto-plastic model was developed in FLAC (1) using programming facilities in the package. Then the proposed modelling approach was applied to surface subsidence simulation above longwall panels with different panel configurations, varying widths, seam extraction thicknesses, mining depths and seam inclinations. The results of these simulations were validated against UK data using the S.E.H method (3) for seams with up to 30° inclinations. For steeper seam inclinations the results were validated against the influence function method using Surface With Influence Function Techniques (SWIFT) package (2). Furthermore the proposed methodology was applied to a number of case histories to directly validate the model results against measured data. This modelling approach proved to be a reliable technique for simulation of the anisotropic pattern of displacement distribution above inclined seams.

MODEL CONFIGURATIONS

Figure 1 shows the general panel configurations and boundary conditions used in the numerical analyses. This configuration has been used effectively for surface subsidence simulation of longwall panels by several investigators, (4,5). Insitu stresses were assumed to be hydrostatic for UK conditions, (6,7,8).

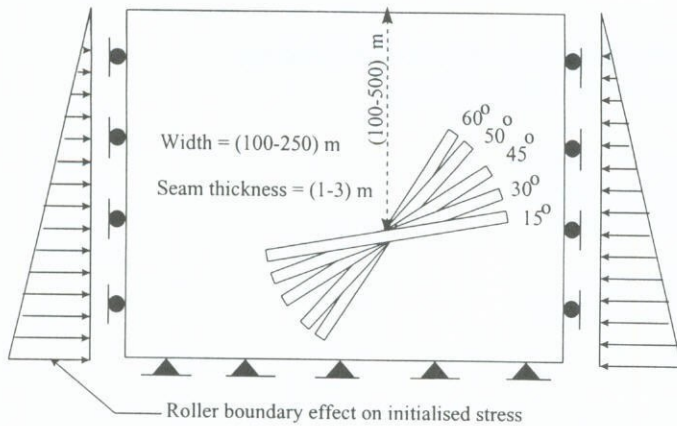


Figure 1 General panel configurations and boundary conditions

EVALUATION OF FLAC STANDARD MODELS

The main initial objective of the investigation was to examine the basic capabilities of each standard FLAC model for surface subsidence simulation above inclined longwall panels. The base material properties used was those suggested by Yao et al (5) as presented in Table 1. An effort was made to recognise the basic trends of strata movement suggested by different models by using the methodology recommended by established investigators (9,10). This technique is based on trial runs using the available models adjusting material properties for each run and evaluating the results. The following numerical models were evaluated.

Table 1 Base material properties used in initial modelling

E_m (MPa)	E_x/E_y	G (MPa)	ν
1100	1.3	458	0.2

Isotropic Models

- Elastic Isotropic Model
- Mohr-Coulomb Model
- Strain hardening/Softening Model.

Anisotropic Models

- Transversely Isotropic Elastic model
- Ubiquitous Joints Model

Several longwall panels with different configurations were simulated using these constitutive relations. An example of the surface subsidence troughs predicted for a 200m wide panel with a seam extraction thickness of 2m is presented in Figure 2. The evaluation of these initial results led to the following important conclusions:

- The position of maximum surface subsidence calculated by the isotropic models regardless of adjustment of the material properties was always above the centre of panel rather than on the down dip side which is the case for inclined seams (11).
- For more gently inclined seams the transversely isotropic model realistically simulated the general pattern of strata movement, however it underestimated the magnitude of maximum surface subsidence and the horizontal displacements; because of the lack of a failure criterion in the model.
- A ubiquitous joint model was able to partially simulate the trend of strata movement just around and above the centre of panels, however it created serious anomalies in the shape of subsidence trough toward the model boundaries. Similar anomalies were observed by Mohammad et al (12) when they used this model for surface subsidence simulation of more level seams.

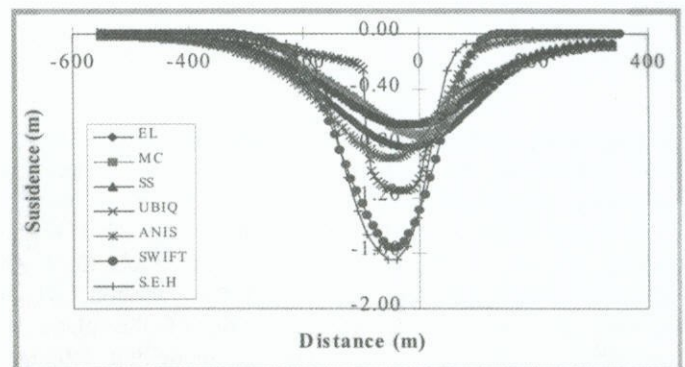


Figure 2 comparison surface subsidence predicted by different models

On the basis of these analyses it was concluded that none of the FLAC standard constitutive models could accurately simulate the asymmetric pattern of strata movement in inclined seam situations. Further more it was also concluded that the introduction of plasticity into the transversely isotropic elastic model could result with a more realistic approach to simulation of surface subsidence in inclined situations, both in terms of the position of maximum surface subsidence and magnitudes of the components of surface strata movement.

DEVELOPMENT OF A TRANSVERSELY ELASTO-PLASTIC NUMERICAL MODEL

A user-defined transversely isotropic elasto-plastic constitutive model was configured in FLAC using a programming language embedded in FLAC called FISH (FLACish). Stress-strain relations prior to failure were calculated using the theory of transversely isotropic elastic bodies (13). The plasticity was introduced into the model using the flow rules (14,15,1). The failure criterion employed was a modified Mohr-Coulomb failure criterion assuming a variable cohesion, which changed with respect to the orientation of the plane of anisotropy (16,17).

According to Lekhnitskii (13) in a transversely isotropic body where the plane of anisotropy coincide with principal stress directions and are inclined to the global co-ordinate system the incremental form of stress to strain in a two dimensional space can be expressed by equation 1.

$$\begin{bmatrix} \Delta e_{11} \\ \Delta e_{22} \\ 2\Delta e_{12} \end{bmatrix} = \begin{bmatrix} s_{11} & s_{12} & 0 \\ s_{12} & s_{22} & 0 \\ 0 & 0 & s_{66} \end{bmatrix} \begin{bmatrix} \Delta \sigma_{11} \\ \Delta \sigma_{22} \\ \Delta \sigma_{12} \end{bmatrix} \quad (1)$$

Where

$$s_{11} = \frac{\cos^4 \theta}{E_1} - \left(\frac{1}{G_{12}} - \frac{2\nu_{12}}{E_1} \right) \sin^2 \theta \cos^2 \theta + \frac{\sin^4 \theta}{E_2}$$

$$s_{22} = \frac{\sin^4 \theta}{E_1} + \left(\frac{1}{G_{12}} - \frac{2\nu_{12}}{E_1} \right) \sin^2 \theta \cos^2 \theta + \frac{\cos^4 \theta}{E_2}$$

$$s_{12} = \left(\frac{1}{E_1} + \frac{1}{E_2} + \frac{2\nu_{12}}{E_1} - \frac{1}{G_{12}} \right) \sin^2 \theta \cos^2 \theta - \frac{\nu_{12}}{E_1}$$

$$s_{66} = 4 \left(\frac{1}{E_1} + \frac{1}{E_2} + \frac{2\nu_{12}}{E_1} - \frac{1}{G_{12}} \right) \sin^2 \theta \cos^2 \theta + \frac{1}{G_{12}}$$

E_1 is elastic modulus in the plane of isotropy (x axis)

E_2 is elastic modulus in the direction perpendicular to the plane of isotropy (y axis)

G_{12} is shear modulus

ν_{12} Poisson's ratio

θ is angle between the plane of isotropy and the x axis

In Figure 3 the concept of a transversely isotropic model and how it applies to a steep seam mining situation is presented.

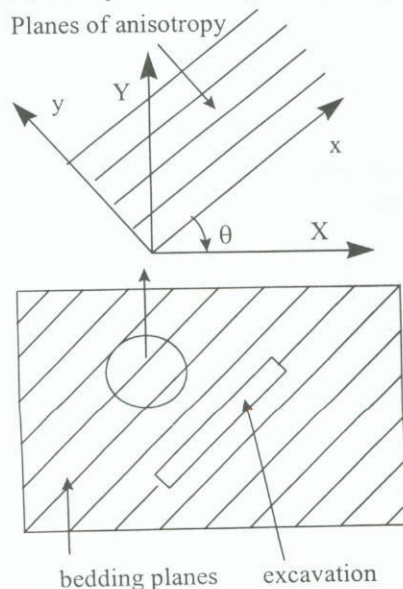


Figure 3 Concept of a Transversely isotropic body oriented at angle of θ from the X axis

Failure Criterion

The failure criterion incorporated in this model was a modified Mohr-Coulomb criterion with variable cohesion and internal friction angle criterion.

$$\sigma_1 - \sigma_2 N_\phi + 2c\sqrt{N_\phi} = 0 \quad (2)$$

$$N_\phi = \frac{1 + \sin \phi}{1 - \sin \phi} \quad (3)$$

Where c and ϕ are cohesion and internal friction angle which are dependent upon the orientation of the bedding planes. If β is the angle between the axial stress (major principal stress) and bedding planes in triaxial compressive tests then the values of c and ϕ can be calculated from the following equations (15).

If $\alpha \geq \beta \geq 0$

$$c = a_1 - b_1 \cos^n 2(\alpha - \beta) \quad (4a)$$

$$\tan \phi = c_1 - d_1 \cos^n 2(\alpha - \beta) \quad (4b)$$

and if $90 \geq \beta > \alpha$

$$c = a_2 - b_2 \cos^n 2(\alpha - \beta) \quad (5a)$$

$$\tan \phi = c_2 - d_2 \cos^n 2(\alpha - \beta) \quad (5b)$$

Where and a_i, b_i, c_i, d_i ($i = 1, 2$), α are constants and n is a factor depends on the type of anisotropy. This factor has values of 1 to 3 for planar type of anisotropy (cleavage and schistosity) and values of 5 or 6 or even greater for linear type of anisotropy associated with bedding planes (16). α is the value of β at which cohesion or friction angle are minimum. It should be noted that α is not necessarily the same in equations of cohesion and friction angle. The above mentioned values were calculated using laboratory data carried out by Hassani (18).

Table 2 Average laboratory strength parameters for UK coal measures (18)

β (degrees)	c (MPa)	ϕ (degrees)
0-10	15.91	34.7
40-50	12.84	34.6
80-90	13.94	34.8

Calculation of Plastic Strains

The values of plastic strains were calculated applying an associated flow rule for shear failure and a non-associated flow rule for tensile failure respectively (14,15,1).

Plasticity was allowed to propagate along the plane of isotropy surrounding the excavated longwall panel and was governed by the angle of the anisotropic planes related to the above failure criterion and properties. This facility was initiated within the modelled zone of influence caused by the excavation geometry.

In order to save on model run time the surrounding model area was simulated as a transversely isotropic elastic medium.

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Numerical Model Implementation

The implementation of this new modelling technique is illustrated in Figure 4 in the form of a simplified flow chart.

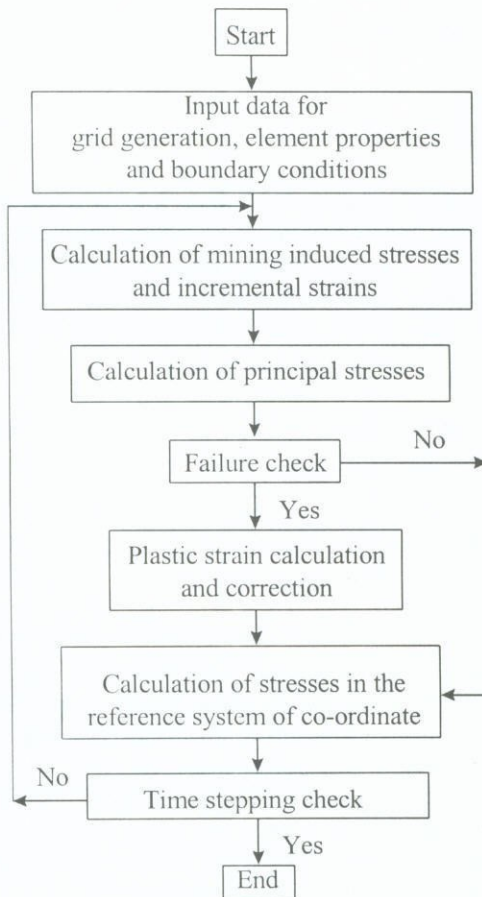


Figure 4. a simplified flowchart of the proposed model

MODEL VALIDATION

Laboratory Properties

The average laboratory properties for British Coal Measures were gathered from a significant amount of work carried out by Hassani (18) and Yasar (19). These data are presented in Table 3. The values of a_i , b_i , c_i , d_i $i=1,2,\beta$ and n have been defined in equations 4a, 4b, 5a and 5b.

Table 3 Average laboratory data for British Coal Measures (18,19).

E_x (MPa)	12440	c_1 (MPa)	0.7
E_y (MPa)	9530	d_1 (MPa)	0
a_1 (MPa)	15.9	c_2 (MPa)	0.7
b_1 (MPa)	3.07	d_2 (MPa)	0
a_2 (MPa)	13.94	α (degrees)	40
b_2 (MPa)	1.1	n	6

Numerical Modelling Input Parameters

To derive input parameters representative of insitu conditions for a 200m wide longwall panel within strata inclined at 15° at a depth of 200m and 2m extraction thickness a series of models were run. The results of the simulations were compared with both the S.E.H and SWIFT subsidence prediction methods. After extensive numerical simulations and result interpretation a reduction factor of one-tenth to the laboratory data Young's Moduli and cohesion parameters provided the best model results. The shear modulus and Poisson's ratio were used from research by Yao et al., (5). They carried out surface subsidence modelling of inclined longwall panels using a transversely isotropic finite element method. Within these series of analyses once an element failed its properties were further reduced to one-tenth of the original insitu values (20,21).

Sensitivity of the New Model to Varying Panel Widths

The sensitivity of the modelling approach to changes of panel width was examined by using the general model configuration illustrated in Figure 1. The panels widths were 150m, 200m and 250m, a depth of 200m from surface to the centre of panel was used with seam thickness and bedding inclination set at, 2m and 15° respectively. The results of maximum surface subsidence (S_{max}) simulation were found to be in excellent agreement with those predicted by the S.E.H and SWIFT methods (Figure 5). The displacement results showed a closer match with those predicted by the SWIFT method than the S.E.H for this seam inclination (Figure 6).

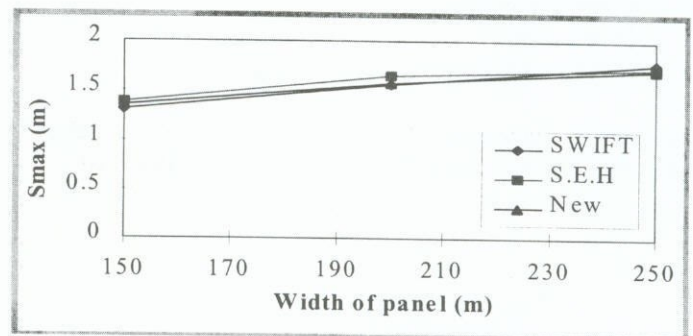


Figure 5 Modelled and predicted maximum surface subsidence (S_{max}) for different width of panels.

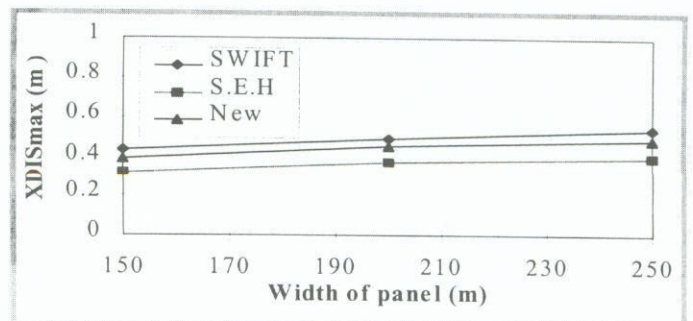


Figure 6 Modelled and predicted maximum horizontal displacements ($XDIS_{max}$) for different width of panels

Sensitivity of the New Model to the Seam Thickness

The accuracy of the new modelling methodology to changes of seam thickness was examined using the panel configuration used for the previous case. Seam thicknesses were set at 1m, 2m and 3m respectively. These results were consistent with previous results showing an excellent agreement between subsidence troughs predicted by each of the three methods. The horizontal displacement values predicted by the numerical model again showed a good agreement with those calculated by SWIFT and both of them produced greater values than those suggested by S.E.H. These results are presented in Figures 7 and 8.

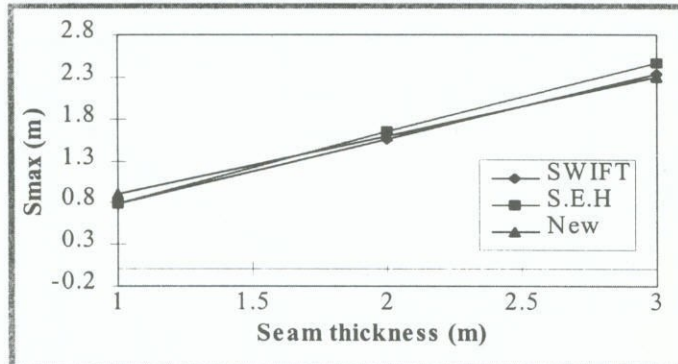


Figure 7 Modelled and predicted maximum surface subsidence (S_{max}) for different seam thicknesses.

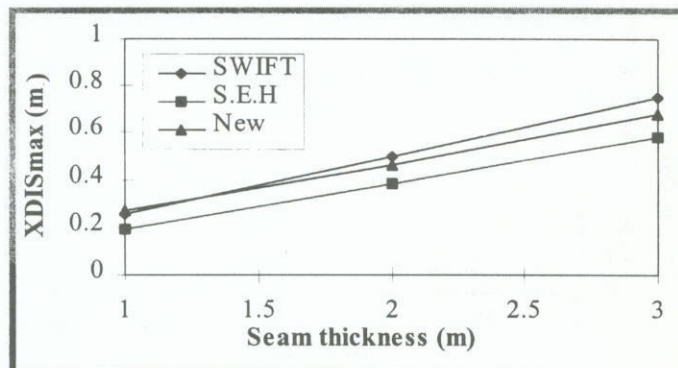


Figure 8 Modelled and predicted maximum horizontal displacements ($XDIS_{max}$) for different seam thicknesses

Sensitivity of the Model to Mining Depth

The sensitivity of the modelling technique to changes of depth was examined using a 200m wide longwall panel in strata inclined at an angle of 15° and a seam thickness of 2 m with depths 100m, 200m, 300m, 400m, and 500m. The input parameters were those used for the previous cases. The results of simulations showed an excellent match with those predicted by SWIFT and the S.E.H methods for only the 200m depth case. For the 100m depth case the surface subsidence trough predicted by the numerical model was shallower than that predicted by both SWIFT and S.E.H. Whereas for the cases greater than 200m below surface the troughs were deeper than those predicted by S.E.H and SWIFT methods. To better understand the reasons for this discrepancy in the results, relevant literature

was carefully reviewed. This review showed that firstly stiffness and strength parameters as well as reduction factor are best represented as functions of Rock Mass Rating (RMR) which has been suggested by Bieniawski (22), Hoek et al., (23) and Nicholson et al (24). Secondly stiffness parameters are depth dependent (21,25). When Battachryya and Shu (25) studied surface subsidence in the coalfields of New South Wales in Australia, they found that the value for model input elastic modulus increased when the depth of working increased. Similarly when Lloyd et al., (21) were analyzing the surface subsidence in UK coal measures situations, they found that both RMR and elastic modulus increased when mining depth increased. They justified their results with comparison of model surface subsidence profiles of a range of depths with those predicted by S.E.H.

On the basis of this work it was concluded that reduction factors were depth dependent parameters. Then different reduction factors were applied to the laboratory data for different mining depths until modelled surface subsidence compared reasonably with that predicted by S.E.H and SWIFT. After extensive model runs the insitu parameters presented in Table 4 proved to be the most representative for the range of depths modelled. (a_i , b_i , $i=1,2$ are defined in equation 4a and 4b). The best fit line graphs for material properties and reduction factors are presented in Figures 9 to 16.

Table 4 Input parameters for different depths

Depth (m)	100	200	300	400	500
E_x (MPa)	830	1244	2488	5598	7141
E_y (MPa)	630	935	1906	4289	5470
G_{xy} (MPa)	32	48	96	216	276
a_1 (MPa)	1.06	1.59	3.18	7.19	9.65
b_1 (MPa)	0.2	0.307	0.61	1.39	2.01
a_2 (MPa)	0.92	1.394	2.78	6.14	9.01
b_2 (MPa)	0.07	0.11	0.22	0.5	0.72

The results of simulations using this approach were found to be in close agreement to that predicted by the S.E.H results for all depth cases in terms of surface subsidence. However as the depth of workings increased the SWIFT method predicted lower surface subsidence values. SWIFT and the new model were in very close agreement to prediction of horizontal displacements for the all cases, whereas S.E.H showed a reduced value of horizontal displacement when compared with SWIFT and the new model (Figures 17 and 18).

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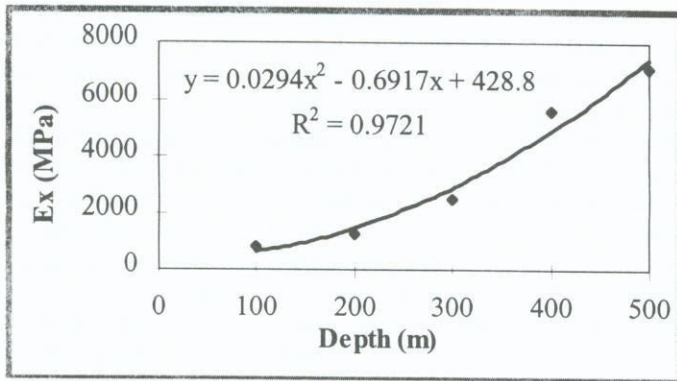


Figure 9 Elastic Modulus in the x direction against depth

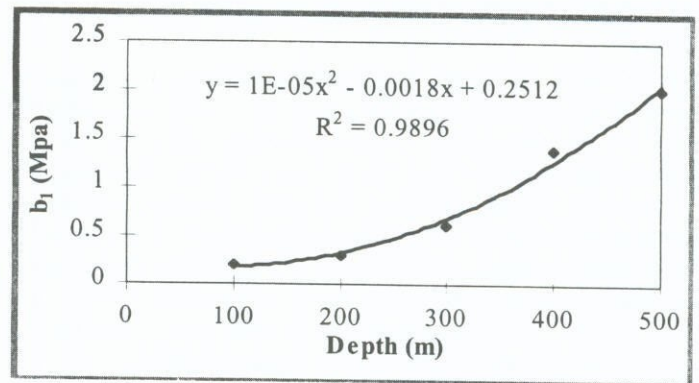


Figure 13 Cohesion parameter b_1 against depth

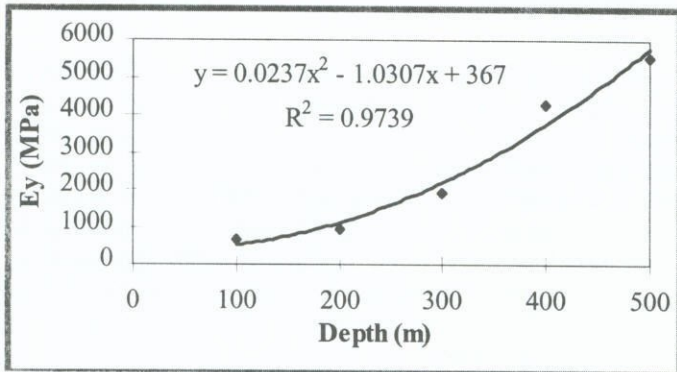


Figure 10 Elastic modulus in the y direction against depth

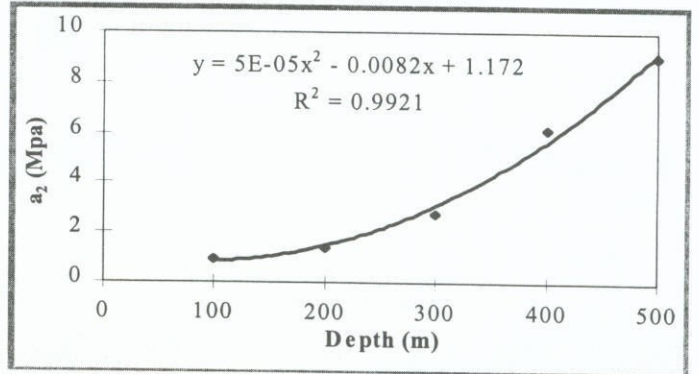


Figure 14 Cohesion parameter a_2 against depth

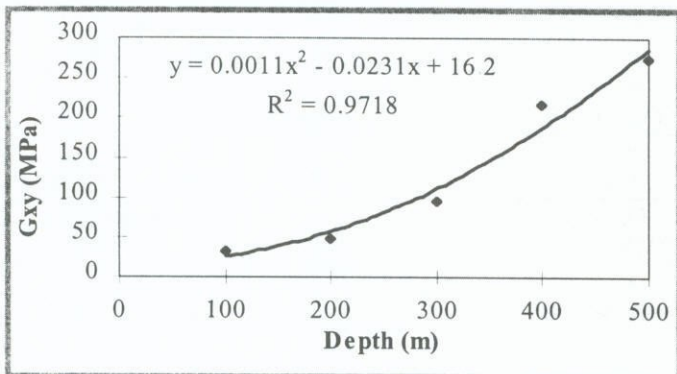


Figure 11 Shear modulus against depth

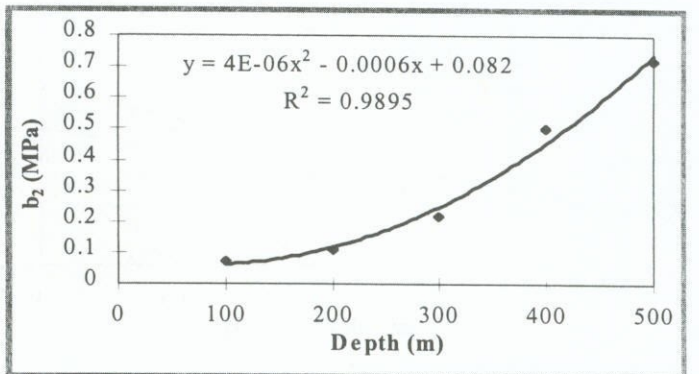


Figure 15 Cohesion parameter b_2 against depth

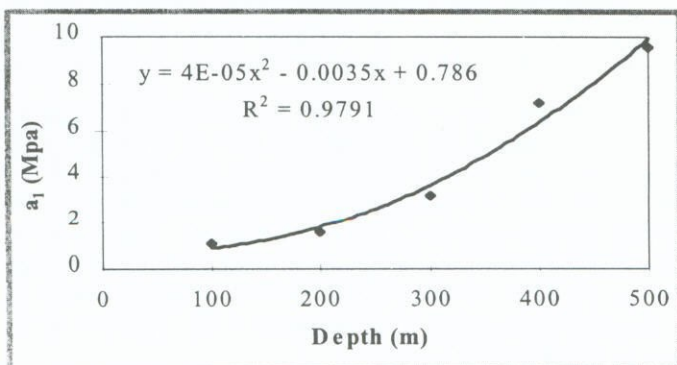


Figure 12 Cohesion parameter a_1 against depth

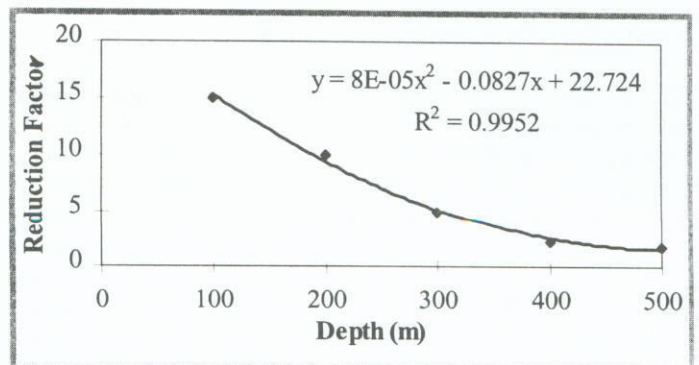


Figure 16 Reduction factor (Lab value/ rock mass value) against depth

The following equations can best fit the experimental data.

$$E_x = 0.0294h - 0.691h + 428.8 \quad (6)$$

$$E_y = 0.0237h^2 - 1.0307h + 367 \quad (7)$$

$$G_{xy} = 0.0011h^2 - 0.0231h + 16.2 \quad (8)$$

$$a_1 = 0.00004h^2 - 0.0035h + 0.782 \quad (9)$$

$$b_1 = 0.00005h^2 - 0.0018h + 0.2512 \quad (10)$$

$$a_2 = 0.00005h^2 - 0.0082h + 1.172 \quad (11)$$

$$b_2 = 0.000004h^2 - 0.00006h + 0.082 \quad (12)$$

$$\text{Reduction Factor} = 0.00008h^2 - 0.0827h + 22.724 \quad (13)$$

h = depth (m)

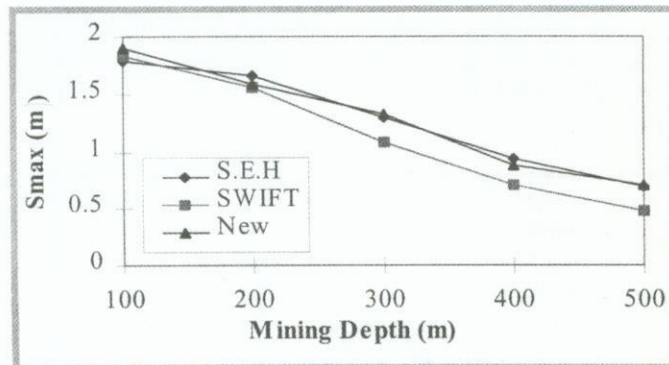


Figure 17 Comparison of maximum surface subsidence at different mining depths.

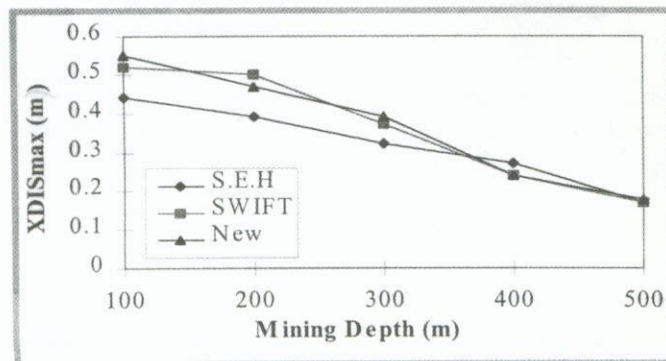


Figure 18 Comparison of maximum horizontal displacement (XDISmax) at different depths

In Figure 19 a comparison between elastic modulus used in this study with those suggested by Lloyd et al., (21) are presented. A close agreement can be observed between these two data.

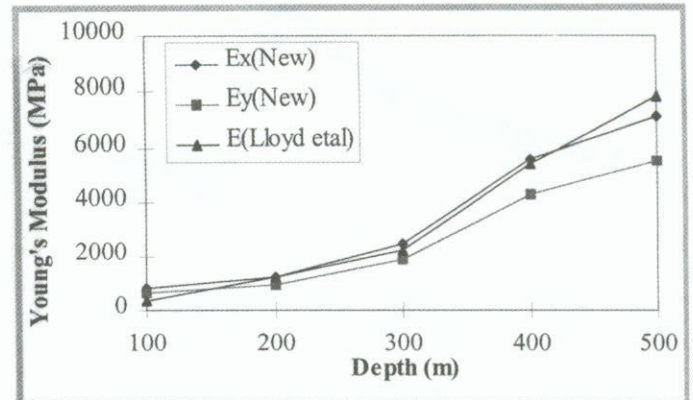


Figure 19 Comparison between the elastic modulus proposed in this study and those predicted by Lloyd et al (1997).

Sensitivity of the Model to the Seam Inclination

The sensitivity of the modelling approach to seam inclination was examined using the general model configuration presented in Figure 1. The results were compared with those predicted by S.E.H only for 15° and 30° seam inclinations and SWIFT for steeper situations. Excellent agreement was observed between both surface subsidence and horizontal displacement results between the proposed modelling methodology and SWIFT for all inclinations and S.E.H up to 30°. A typical example of results for 45° seam inclination have been presented in Figures 20 and 21. The comparison of the maximum surface subsidence and horizontal displacement results for different seam inclinations predicted by each method are presented for a panel width 200 m, seam extraction thickness 2m and a central depth of 200m in Figures 22 and 23. Material properties were those presented in Table 6.

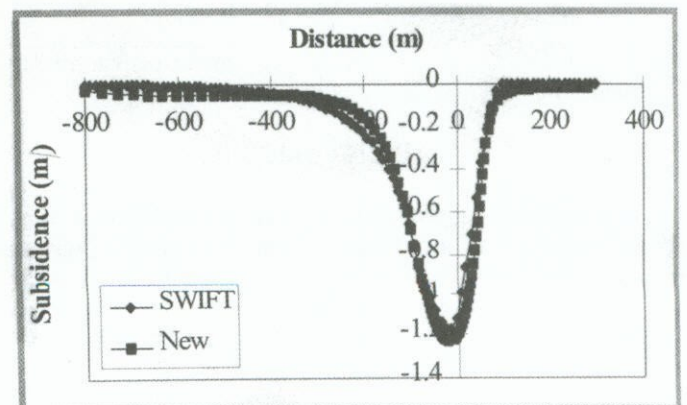


Figure 20 Comparison surface subsidence troughs, panel width = 200m, seam thickness = 2m, central depth = 200m dip = 45°

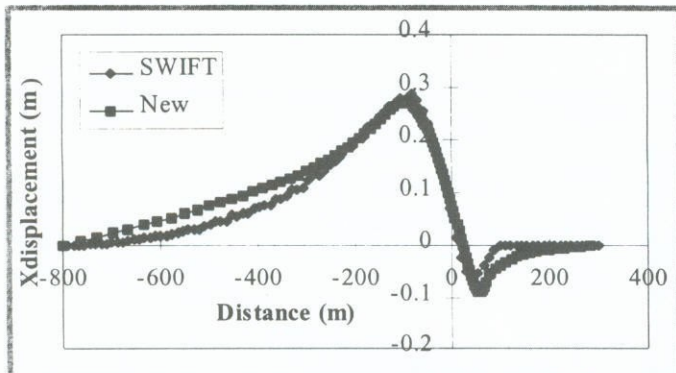


Figure 21 Comparison horizontal displacement troughs, panel width = 200m, seam thickness = 2m, central depth = 200m dip = 45°

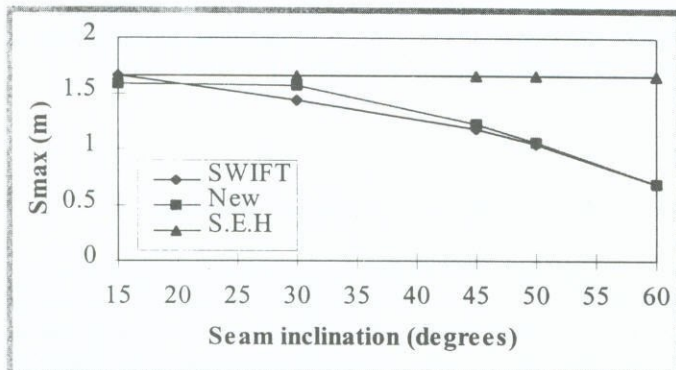


Figure 22 Comparison of maximum surface subsidence for different seam inclinations.

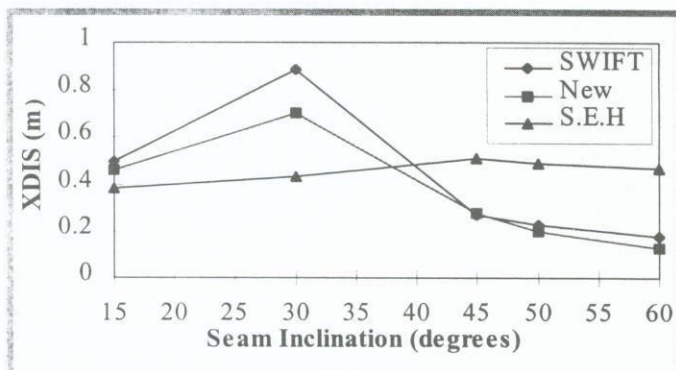


Figure 23 Comparison of maximum horizontal displacements (XDISmax) for different seam inclinations.

CASE HISTORIES

The proposed technique was applied to three case histories from three different collieries to further validate the modelling results directly against measured data. A good agreement was observed between the numerical modelling and surface subsidence magnitude measured data. The results for one of these cases (case No.2) are presented in Figures 24 and 25.

The material properties for this case as with the other cases were derived on the basis of mining depth using the relationships displayed in Figures 9 to 15. These properties are presented in Table 5.

Table 5 Material properties used in case No.2

E_x (MPa)	196	a_1 (MPa)	2.44
E_y (MPa)	1468	b_1 (MPa)	0.47
G_{xy} (MPa)	74	a_2 (MPa)	2.14
ν_{xy}	0.2	b_2 (MPa)	0.16
ν_{xz}	0.2	ϕ (°)	35

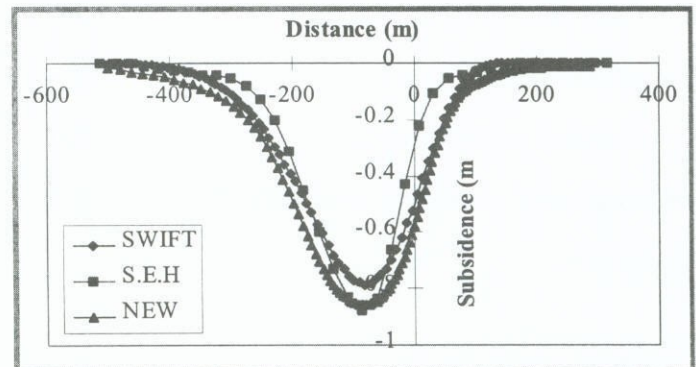


Figure 24 Comparison of surface subsidence troughs predicted by the New model, S.E.H, SWIFT and measured data, central depth = 254, panel with = 207m, seam thickness = 1.17 and dip = 21°

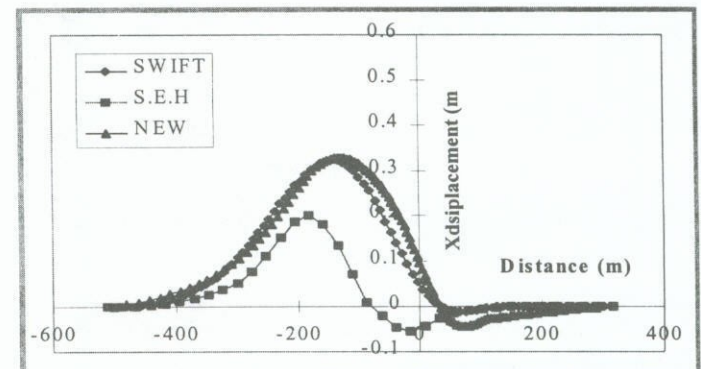


Figure 25 Comparison of horizontal displacement troughs predicted by the New model, S.E.H, SWIFT and measured data, central depth = 254m, panel width = 207m, seam thickness = 1.17 and dip = 21°

CONCLUSIONS

The transversely isotropic elastic theory that also introduces plasticity is better able to simulate the asymmetric pattern of strata movement arising from longwall mining of inclined and steep seams.

The reduction factor for the numerical modelling insitu properties derivation is a depth dependent parameter which reduces dramatically when the depth of working increases.

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The rock insitu stiffness and strength parameters including elastic modulus parallel to the plane of anisotropy and perpendicular to the plane of anisotropy, shear modulus and variable cohesion are varying with mining depth.

The magnitude of maximum surface subsidence for a given panel decreases when the seam inclination increases whereas the magnitude horizontal displacement increases sharply up to around 30 degrees, then there is a sharp fall until it reaches around 45 degrees from which decreases very gently up to 60 degrees inclination.

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Landslide Occurrence and Causation in Steep Slope Areas of Appalachia

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ABSTRACT

Landslides and soil creep often occur with varying degrees of severity on steep slopes within Southern Appalachia. Ground movement may take place over years with subtle changes in topography and vegetation or it can occur rapidly in response to water inflow or disturbance at the toe of the landslide. Although the ground movement typically is restricted within the soil and colluvial layers, failure can extend into the weathered rock zone. Certain areas of Appalachia are described as "slide prone" because of historical precedent. To develop a profile of "slide prone" areas, the strength and physical properties of the soil and colluvium, topography, groundwater and surface water runoff, present and historical land use, and any surface disturbance are examined in detail. These attributes are useful in determining whether an area is potentially unstable before surface development including construction, logging, or mining. Recommendations are provided to avoid initiating ground movement concurrent with mining operations.

Two case histories of landslides in Eastern Kentucky are presented. Both occurred in historically "slide prone" areas and within close proximity to mountaintop surface mine operations. In the first case history, the landslide is composed of two separate and distinct landslides. The upper landslide was caused by mine spoil pushed off permit while the second was pre-existing and reactivated through erosion of the toe by a stream. The extent of these landslides and their associated failure planes were clearly delineated using resistivity and seismic geophysical investigations. Landslide causation was traced to seeps from an abandoned and inundated underground mine. In the second case history, a landslide occurred downslope from an active surface mine in the Hazard Nos. 7 and 8 seams. Based on observations on both mined and unmined mountaintops, groundwater seeps are present in the Hazard No. 5A seam. Construction of logging roads accelerated the groundwater flow into the landslide area culminating in a large failure.

INTRODUCTION

Landslides and debris flows in soils and unconsolidated materials are initiated by several sources. Rainfall (Anderson and Sitar, 1995, Johnson and Sitar, 1990, Matsuo and Suzuki, 1988) and an elevated groundwater table (Reddi et al., 1991) are two of the most common sources of slope failure that results in a landslide. Increased pore pressure within a slope occurs during heavy rainfall as the slope is saturated. A rise in the phreatic surface or the elevation of the groundwater table affects pore pressure within the soil or colluvium. These phenomena cause a decrease in the effective stress within the slope and the cohesion and friction angle of the soil or unconsolidated material. The resistance to gravity induced movement as described by the Mohr-Coulomb or other laws correspondingly decreases the slope failure. Landslides are also initiated through natural disasters including earthquakes and volcanoes (Voight, 1992), mining induced subsidence (Peng and Lu, 1989, Barends, 1991), or the operation of mobile equipment near the slope crest (Michalowski and May, 1991).

In the Appalachian coalfield, the two significant causes of landslides are heavy rainfall or groundwater and long term soil creep. Stump et al., (1995) examined creep slope failure in Pike County, Kentucky. They found that although the safety factors for deep seated rotational failure were sufficient, borehole inclinometers showed that the slope was sliding 0.18 to 0.34 cm/day. The phenomena of soil creep has long been recognized in steep slope areas of Appalachia. Physical indications include colluvial bulges at the base of hillsides, portions of a hillside where the drainage parts around a soil mass in a "Y" shape, tilted trees and utility poles, and bowed tree trunks.

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LANDSLIDE CASE HISTORY A - EASTERN KENTUCKY



Case history A involves two separate and distinct landslides that occurred below a mountaintop removal surface mine operation. Visually the site appears to be a single, continuous landslide beginning at a circular failure scarp at the boundary of the surface mine and terminating in a large colluvial soil mass beside a creek at the base of the hillside. However, a detailed subsurface geophysical investigation, topographic surveying, and historical data germane to the mining operation and antecedent site conditions enabled the clear identification of two landslides within the hillside. The extent and causation of each landslide, the thicknesses of the soil layer, weathered rock, and unstable material were the primary objectives of the investigation.

HISTORICAL MINING ACTIVITY AND CONTEMPORANEOUS SITE CONDITIONS

The surface mine operation was a mountaintop removal operation to recover the Hazard No. 8 and Hazard No. 7 coal seams with contour mining of the Hazard No. 5A seam. The Hazard No. 5A seam had previously been deep mined from two underground mines dating to the mid 1950's to early 1960's based on the final mine map dated May 18, 1963 and two posting dates of 11-01-54 and 12-06-54. Evidence of the second mine includes a rough outline on the map of the first mine and an excavation associated with a two-entry portal and mine face-up that are still visible on the site. The underground mine and abandoned portal are assumed to be contemporaneous with the first mine because the pillar geometry and style of mining are consistent with mining practices of the 1950's and 1960's. Although the portal is separate from the mine workings, it is within two hundred (200') feet to two hundred-fifty (250') feet of the first mine and possibly connected although this is not shown on the mine map.

A bench from a pre-law contour surface/auger mine in the Hazard No. 5A seam is also present across the site. While the exact date of the surface mining is unknown, the extent of augering is shown on the underground mine map. The remnants of the pre-law auger bench are visible from both the air and ground along with accumulations of spoil pushed over the hillside. Because of the prior underground and surface mining in the Hazard No. 5A, the present operator opted to forego additional contour mining and limited mining and surface disturbance to the mountaintop removal area. In Figure 1, the premining topography, an outline of the underground mine and portal, and the outcrops of the three seams are shown at approximately 1,340 feet amsl (Hazard No. 8), 1,300 feet amsl (Hazard No. 7), and 1,230 feet amsl (Hazard 5A). From the Figure it is apparent that the landslide and adjacent areas are on steep slopes ranging between forty (40%) percent and fifty (50%) in grade. Outerbridge (1979) described the vicinity of the landslides as slide prone based upon historical

occurrences, steep slopes, presence of seeps and springs, and the colluvial composition of soil.

During mountaintop surface mining of the Hazard No. 8 and Hazard No. 7 seams, mine spoil either slid or was inadvertently pushed off the permit boundary. The spoil material extended below the pre-law Hazard No. 5A bench forming a small landslide, referred to as the upper landslide in the remainder of the paper. The upper landslide that was stabilized at the time of the investigation, is characterized by a sharp semicircular scarp against the reclaimed surface mine. This circular failure consistent with slope failure in the unconsolidated spoil material.

The areal extent of the upper landslide was incorporated into the mountaintop permit and a stable toe was established using the large rocks that rolled farther down the hill. The smaller rocks and finer spoil was regraded above the stabilized toe area. The spoil landslide was then revegetated and appeared stable with no tension cracks visible in the scarp face. The surface material consists of both soil and rock fragments of varying dimensions. No water was visible on the upper landslide and the material was stiff under foot. No movement of the large rocks placed at the toe of the landslide was evident, except in the area excavated for a drainage ditch, discussed later in the paper, used to divert runoff from the abandoned Hazard No. 5A seam mine.

Prior mining of the Hazard No. 5A is important to the causation of the lower landslide that was active at the time of the investigation. During site inspections, water was observed actively flowing out of the two entry portal area during a site inspection at the rate of approximately twenty (20) gallons per minute. The portal openings were not visible because material had covered the excavation, but their presence was readily discernible in the hillside. The Hazard No. 5A surface mine bench was saturated and generally marshy. Periodically, the water was ponded in localized depressions on the bench. Cattails and other vegetation associated with permanent saturation were prevalent on and immediately below the old bench. Water was observed flowing from the left side of the abandoned Hazard No. 5A portal area, along the bench, and ultimately into a diversion ditch, constructed at the recommendation of a regulatory agency, across the toe of the upper landslide. The purpose of the ditch is to drain and divert water from the abandoned Hazard No. 5A mine across the base of the upper landslide area into a natural drain located to the North of the upper landslide. Outside the landslide area, no water or damp areas were observed either above or below the Hazard No. 5A horizon.

In Figure 2, the areal extent of both landslides is illustrated in addition to the location of the creek, two homes, and two drains on the North and South sides of the lower landslide. The basal portion of the lower landslide is a soil cliff as shown in Figure 2 and in the profile developed for the geophysical investigation. Periodic movement of

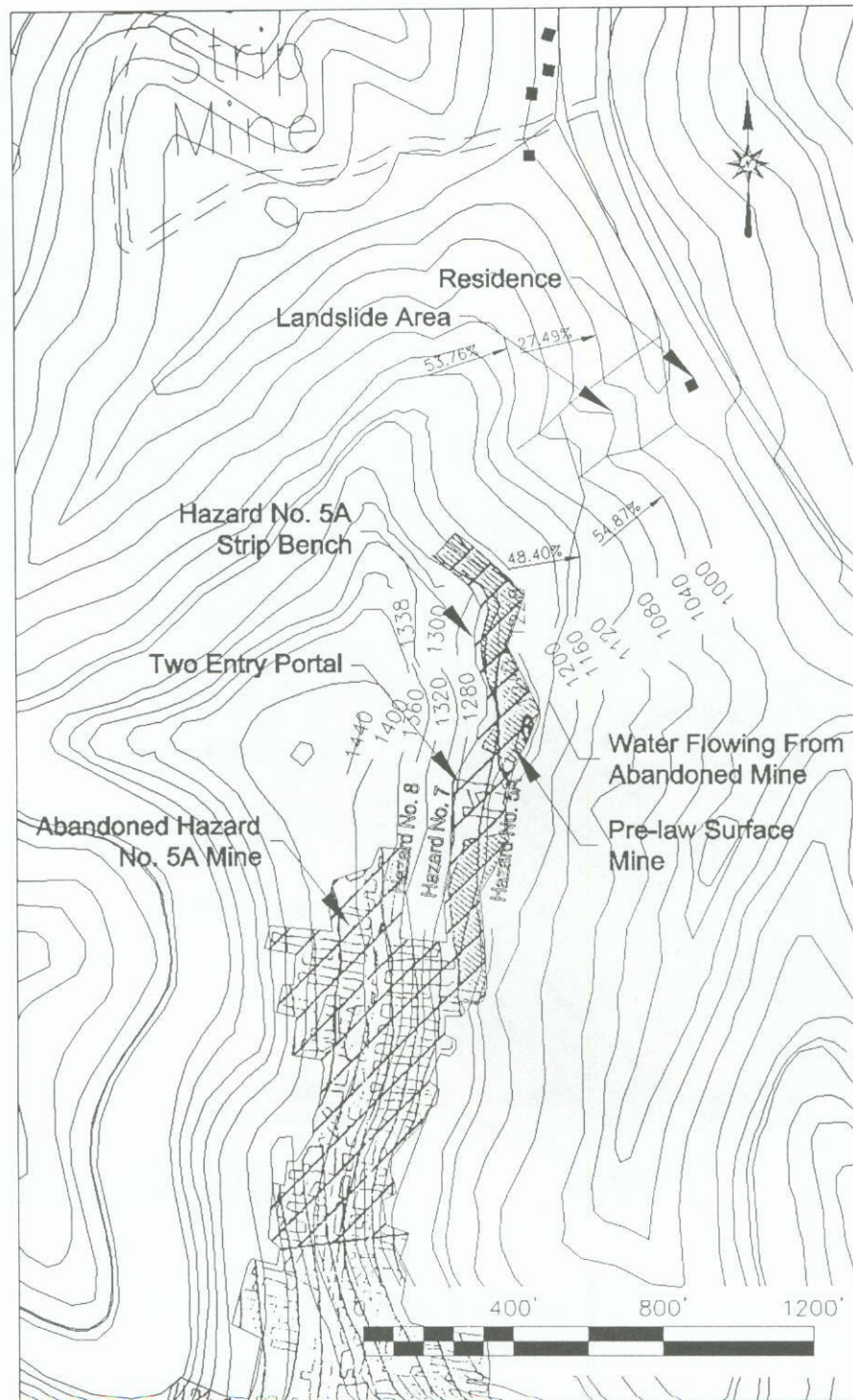


Figure 1. Pre Mining Topography and Location of Physical Features at the Case A Mine Site.

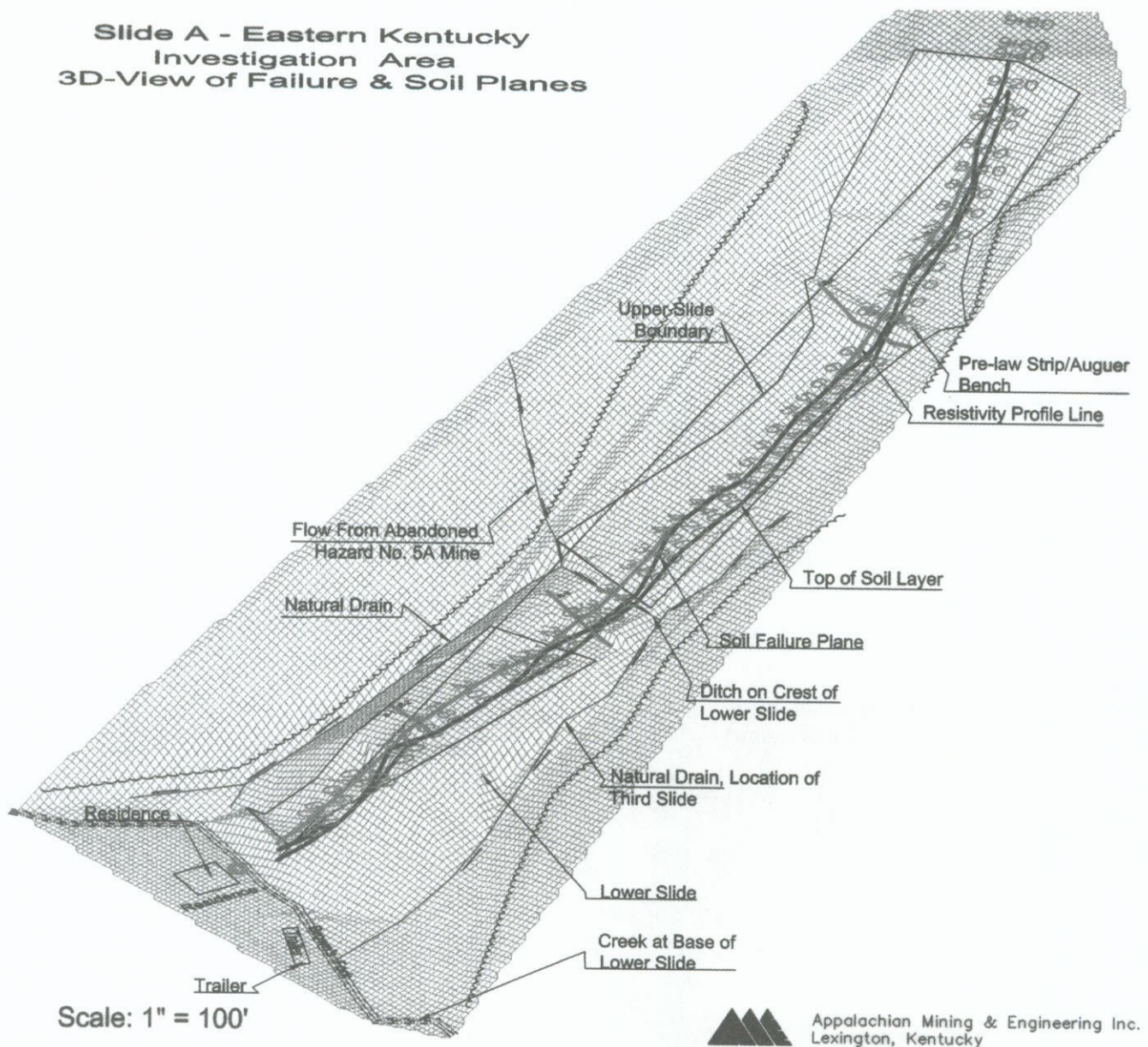


Figure 2. Three Dimensional View of Case A Landslides.

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small rock and soil particles was observed along the portion of the landslide across the creek and directly behind the home. Soil from the movement of the lower landslide had entered the creek and creek flow was actively undercutting the lower portion of the lower landslide. As movement of the lower landslide continued there was a concern that the landslide would flow across the creek, effectively damming it. Furthermore, the lower landslide was a threat to the safety of the residents on the East side of the creek.

The lower landslide surface was completely denuded of vegetation and multiple seeps are present in the soil cliff face. Walking on the soil cliff and adjacent bulge area was difficult because the saturated soil is soft. Frequently the author sank to ankle or calf level while walking across this portion of the landslide. Above the soil cliff is a flat area where brush and trees were piled during the clearing and grubbing operation. The soil within the flat area is less saturated than the material down slope within the soil cliff. Continuing up slope from the flat area, the large rocks associated with the lower landslide are encountered along with the diversion ditch.

Multiple soil tension cracks were seen on the flat area. These cracks are a direct result of down slope movement and evidence that the lower landslide was active. Surface water from the Hazard No. 5A mine and natural runoff from precipitation events was free to enter the tension cracks. Because the natural grade is in the direction of the soil cliff, water entering the tension cracks continued to travel down slope toward the soil cliff and promote continued soil creep. As the soil creep continued, the tension cracks dilated and provided a free face for movement of soil on the uphill side of the tension crack. This cycle of down slope movement, development of up slope tension cracks, and the unrestricted entrance of water into the tension cracks and through the landslide area is the mechanism through which the lower landslide propagated uphill.

As this up slope propagation continues, the mine operator was concerned that the lower landslide would destabilize the toe of the upper landslide and the entire soil and spoil mass would move down hill. A regulatory agency contented that the two landslides were in fact connected as one landslide and the mine operator was responsible for remediation of the entire hillside. This process had already begun as is shown in the geophysical investigation with the location of a failure plane in the boundary between the two landslides.

A third landslide was observed during active surface mining by the residents of the home and trailer on the opposite side of the creek. This landslide was initiated by the water draining from the diversion ditch at the toe of the upper landslide and passing down a natural drain to the North of the lower landslide. Soil material within the drain has traveled down hill and was seen mounded at the base of the natural drain, beside the creek. Trees on either side of the drain had been undercut and uprooted by the

landslide. The causation and development of the landslide from the diversion of the Hazard No. 5A seam water are identical to the continued instability of the lower landslide.

Based upon the findings and conclusions of the site inspection, it was recommended that a geophysical investigation of both landslides be conducted to determine the thicknesses of both soil and fractured rock zones, and the depth to competent bedrock.

GEOPHYSICAL INVESTIGATIONS OF THE LANDSLIDES

A geophysical investigation of the upper and lower landslides was conducted by GECOH Exploration, Inc. The purpose of the investigation is to define the depth to bedrock and the thickness of the unconsolidated soil, mine spoil, and colluvial material that comprise both landslides. A combination of surface resistivity and surface seismic methods were used in the investigation. At this site geophysical methods were deemed preferable to drilling to confirm soil depth because:

- i) a larger number of data points can be obtained and
- ii) drilling is impractical because of a lack of access to the landslide areas and the concern that the weight and vibration of a drill rig would destabilize the lower landslide.

Resistivity measurements were recorded at surveyed stations located along a centerline passing through both landslides from the creek to the crest of the upper landslide. The results of the geophysical profile line confirm the surface observations and preliminary conclusions drawn during the site inspection. Specifically, there are two distinct landslides with well defined circular failure planes. The first (lower) landslide is the active landslide located directly above the house and the second is the upper landslide. The lower landslide as illustrated in Figure 2 occurs from Station 4 to Station 20. Within the lower landslide slide, three distinct failure surfaces are defined through changes in resistivity. These resistivity changes are attributable to increased moisture present along the circular failure plane. The first failure plane, found between Stations 4 and 10, corresponds with the soil cliff described in the site inspection. A second failure plane, found from Station 10 to Station 20, occurs near the "flat." While the third failure plane, between Stations 20 and 28 is found above the "flat" in the boundary between the lower and upper landslides. The mechanism of up slope propagation is clearly illustrated in Figure 2. If the soil cliff fails and slides into the creek, the second failure plane has a free face to which it can move. It will shift into the void created through the movement of the first failure plane. The same process will initiate movement of the third failure plane.

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The upper landslide is found between Stations 24 and 59. A zone of relative stability occurs between the lower and upper landslides, at Stations 28 through 37. This zone corresponds with the toe of the upper landslide, stabilized by large rocks and regrading the spoil material. The toe is critical to maintaining the stability of two failure planes (Stations 37 to 48 and Stations 48 to 55) found within the upper landslide. These two failure planes illustrate the value of the geophysical investigation because they are not readily apparent from a surface inspection of the upper landslide.

The main conclusion of the landslide investigation is that there are two distinct and separate landslides. These have been referred to as the lower and upper landslide. The extent of these landslides was observed during the site inspection and subsequently confirmed through a geophysical analysis of the subsurface conditions. The lower landslide is active and consists of two separate failure planes with a third between the lower and upper landslides. If the first failure plane, located in the soil cliff area failed, movement of the other two failure planes, located up slope will be initiated. The up slope propagation of the landslide is clearly illustrated in the geophysical study and through the development of tension cracks. Currently, the upper landslide is stable as the result of remedial work conducted by the mining company. However, once failure of the soil cliff is initiated, the failure within the lower landslide will cause toe failure in the upper landslide and eventually the entire hillside will be connected in one large mass that has distinct segments.

The third conclusion is that water is critical to the stability of the lower landslide. Construction of the diversion ditch provides a continuous flow of water from the Hazard No. 5A seam into the lower landslide area through soil material and tension cracks which have developed in response to slope movement. The detrimental effect of water is clearly seen in the smaller landslide that developed in the adjacent drain, behind the mobile home.

LANDSLIDE CASE HISTORY B - EASTERN KENTUCKY

The second case history is similar to the first in that it involves a mountaintop removal surface mine operation where the Hazard No. 8 and Hazard No. 7 seams were removed. Unlike Case A, the mountaintop operation was not contiguous but was laterally extensive and best characterized as series of knob removals. Approximately fifteen (15') feet below the Hazard No. 7 seam is an intermittent coal, locally referred to as the Hazard No. 7 Sulfur. No surface or underground mining was conducted on the Hazard No. 7 Sulfur seam because it occurs as a series of thin splits that are not economically recoverable and as the name implies, the coal quality is undesirable. The regional dip of all seams is to the Northwest as shown by making a cross section of coreholes drilled in each knob to determine their suitability for surface mining. The Hazard

Nos. 8, 7, 7 Sulfur, and 5A seam outcrops, surface topography, landslide location, and corehole locations are shown in Figure 3 that is a portion of the surface mine site.

Similar to Case A, the area adjacent to Knob No. 7, the site of the landslide was characterized by the U.S.G.S. Geological Survey as slide prone. Physical evidence on site included older landslides at the base of the mountain in the Haddix seam and bowed tree trunks at and below the Hazard No. 5A horizon. Knob No. 7 was prepared for surface mining by logging the entire knob top. Roads were cut to encircle the knob to simplify the removal of logs from the site. During road construction on the North side of the Knob, the Hazard No. 7 Sulfur seam was partially exposed and began seeping.

No slope problems occurred until the surface mine had progressed to directly above the landslide area. A pit of coal was removed on a Friday and during the weekend, heavy rains occurred on the mine site. The landslide occurred during the weekend. It extended for several hundred feet downslope, beginning slightly below the logging road at the Hazard No. 5A elevation. Trees were toppled and the landslide was a mass of saturated colluvial soils and weathered rock. The mine operator cut a small diversion ditch around the crest of the landslide to direct water coming from the Hazard No. 5A exposure to either side of the landslide area. To gain access to the area a small road and bench was developed immediately below the Hazard No. 5A seam outcrop. Although the ditch did facilitate the movement of water away from the landslide area, water was seen pooling in the ditch and on the bench below the Hazard No. 5A.

The landslide causation was discussed with a regulatory agency that contended the presence of the surface mine contributed to the initiation of the landslide. In researching and addressing this point, no other seeps were observed on the knob other than at the road cut. The surface water draining from the now reclaimed mine site was handled by a series of diversion ditches ringing the hillside and draining into on bench ponds.

On an adjacent knob (Knob No. 4) that was timbered but not mined due to insufficient thickness in the Hazard Nos. 7 and 8 seams, natural seeps occurred in only two locations. Both seeps resulted where logging roads were cut into and exposed the Hazard No. 5A seam. Furthermore, an examination was made in September 1997 of all the bench ponds on knobs that were within the immediate vicinity of Knob No. 7 and the landslide. No rainfall had occurred for three to four weeks. Those ponds on the Hazard No. 7 seam floor had small pools with algae present, indicative of stagnate water that had not been recharged from either precipitation or groundwater. However, those ponds on the base of the Hazard No. 5A floor were full and had lush vegetative growth including cattails, that grow under acidic conditions. Unfortunately, the nature of the incident and allegations did not justify as



Figure 3. Site Map for Case B

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extensive an investigation as performed for Case A. However, it is clear that the source of the landslide was pre-existing soil conditions aggravated by water from an active seep.

CONCLUSIONS

Based upon these two cases and the experience of the author over the past thirteen years, landslides are commonplace on steep slope areas of Appalachia. As part of the pre-mine investigations associated with the mine permitting process, a thorough examination of the surrounding area should be conducted to determine if the physical manifestations soil creep (bowed or tilted trees, bulges at the base of hillsides, tilted utility poles) or pre-existing landslide(s) are present. Similarly, the presence of potential landslide initiation sources should be carefully documented including old spoil banks, side hill refuse or gob piles, natural seeps and springs, water flowing from abandoned underground mines, surface mine spoil, and auger holes.

During active mining operations the handling and diversion of surface water are essential to avoiding landslide initiation. Mine benches should not be overloaded with spoil or other unconsolidated material. Before construction of a bench or stockpile area, the lithology of the underlying strata should be determined from examining the logs of exploration coreholes. Benches should not be located above weak shales, fireclays, or coals that may rotate outwards due to shear failure in response to loading on the bench or the use of mobile equipment. Similarly, the strata and soil conditions through which haulroads are constructed should be carefully examined before excavation.

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Rock Bursting and Seismicity During Ramp Development, Lucky Friday Mine, Mullan, Idaho

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ABSTRACT

A comprehensive survey of mine seismicity and rock bursting during development of two sublevels at the Lucky Friday Mine, Mullan, ID, USA, was conducted to better define rock failure mechanisms and sources of ground control hazards. Survey data included rock burst damage reports, seismic event locations and magnitudes and, for the most energetic events, first-motion information. Several subsets of this database, including large seismic events, rock bursts, and microseismic activity near the face, were analyzed. Elements of the large-event and rock burst data sets were nearly independent. That is, there was no relationship between the risk posed by a seismic event and its energy, although such a relationship is well established for the mine as a whole. All data sets showed that certain geologic features appear to control the spatial distribution of events and the spatial distribution of rock burst risk. The data also suggest that, within the scope of this study, the greatest risk of injury occurs when a pocket of heightened rock burst risk is first encountered and that this risk is controlled when miners adapt their practices to these conditions. Recognition of the role of particular geologic features in the spatial distribution of rock burst hazards provides an opportunity for anticipating, rather than only reacting to, a changing level of rock burst hazard. However, much work will be required to make good on this promise.

INTRODUCTION

Ground falls can be attributed to a number of failure mechanisms. Of these, rock bursting¹ is an unusually hazardous failure mechanism. Persistent mining-induced seismicity is a feature of many deep mines in the United States, and events with Richter magnitudes of 2 to 4 are not unusual. Catastrophic rock

bursts may also occur in mines with sparse records of mining-induced seismicity. The 5.2-Richter-magnitude event at the Solvay Mine in which three-quarters of a square mile of the mine suddenly collapsed is a recent example.

Despite a long history of rock burst research, much remains to be learned. Substantial efforts have been made to understand rock burst mechanisms, alter mine designs to reduce rock bursting, improve the effectiveness of ground support, and anticipate (and even predict) the occurrence of rock bursts. However, the occurrence of any particular mining-induced seismic event is still a surprise, and the consequences of a particular event are difficult to ascertain without inspection of affected areas of a mine.

This National Institute for Occupational Safety and Health (NIOSH) study was designed to address the hypothesis that geologic structures, independently and in conjunction with mining-induced stresses, control the spatial distribution of rock burst hazards. This hypothesis implies that the level of hazard can be anticipated, providing an opportunity to control and/or avoid the hazard. This hypothesis was suggested by recent studies that found variations in the premining stress field that were associated with particular geologic structures (Whyatt et al., 1995d), and that these stresses promote some types of the largest (Richter magnitude greater than 2.5) rock bursts (Whyatt et al., 1996b).

This study examines ramp development history in a portion of the Lucky Friday Mine for possible links between (1) geologic structures and (2) patterns of rock burst hazards and mining-induced seismicity. The authors examined a large population of seismic events, defined groups of events with similar characteristics, and explored the essential, repeatable characteristics of these groups. This approach serves to provide a balanced picture of overall seismic activity while de-emphasizing the effect of errors, data deficiencies, and random aspects of individual events.

¹A rock burst is defined as damage to an excavation that occurs in a sudden or violent manner and is associated with a seismic event (Kaiser et al., 1998).

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An intense seismic response to mining; exceptional cooperation by the mine operator; and a wealth of geomechanical, operational, and seismic data made the Lucky Friday Mine a particularly useful laboratory for this study. The Lucky Friday is considered to be the most seismically active mine in the Coeur d'Alene Mining District and among the most seismically active (in terms of seismic energy per ton of ore mined) mines in North America (Jenkins et al., 1990). Whyatt et al. (1996b) found that events with local magnitudes greater than 2.5 occurred, on the average, at a rate of one every 2 months over a 6-year period. This study addresses only a portion of the seismicity that occurred from mid-1990 through mid-1993 and focuses on seismicity associated with development of the 5400 and 5480 sublevels. The selection of study dates and sublevels was based primarily on the relative quality and availability of seismic, in situ stress, geologic, and damage data.

This report begins with an overview of Lucky Friday Mine geology and its geomechanical setting, then discusses development and screening of the seismic event data set. Whyatt and White (in progress) provide detailed data on individual events. Various groups of seismic events are identified, evaluated for degree of rock burst hazard, and investigated for links to various geologic features and mining progress. Finally, the study discusses the potential for integrating these results into planning future development openings in rock-burst-prone mines.

LUCKY FRIDAY MINE: BACKGROUND

The Lucky Friday Mine is located in the eastern portion of the Coeur d'Alene Mining District of northern Idaho. The mine used a traditional overhand cut-and-fill method until a temporary closure of the mine in 1986. A mechanized underhand longwall mining method, which eliminated burst-prone pillars, was introduced when the mine was reopened. A sublevel ramp system was developed to provide rubber-tired vehicles with access to these stopes. The 5400 sublevel was the first to use the current ramp configuration. The change in mining method was supported by a cooperative research effort by Hecla Mining Co., the U.S. Bureau of Mines (USBM), and the University of Idaho (Poada et al., 1995; Whyatt et al., 1992). The underhand cut-and-fill method is proving to be a much safer way to mine in rock-burst-prone ground (Poada et al., 1995). Ground control systems were also substantially improved during this period (Blake and Cuvellier, 1990, 1992). These measures contributed to a remarkable six-fold reduction in Mine Safety and Health Administration (MSHA) reportable accidents at the Lucky Friday (Wilson, 1997).

Geology

The Lucky Friday vein forms an S-shape in plan view and extends horizontally about 460 m (1,500 ft). Splits off this vein extend potential stope length to over 610 m (2,000 ft). The vein is composed primarily of galena, with lesser amounts of quartz, siderite, and sphalerite. The vein ranges from 0.6 to 9 m (2 to 30 ft) wide, but averages about 1.5 m (5 ft) wide. The Revett Formation, which has hosted most historic production in the Coeur d'Alene Mining District, forms the wall rock. Because the

vein itself dips steeply (70° to 90°) to the south and east, it comes into contact with progressively older rocks with depth (figure 1B). Presently, mining is active within the quartzitic lower member of the Revett Formation under 1.5 km (1 mile) of overburden.

Numerous faults and folds are apparent, and many of these intersect the vein structure. The most pronounced faults are the North and South Control faults that delineate the ends of the Lucky Friday vein. The most pronounced fold, the Hook Anticline, divides the mine into southern and eastern limbs. A minor fault approximately traces the axial plane of this fold. The rock mass surrounding the vein is made up of thick vitreous and sericitic quartzite beds with soft interbeds of argillite generally less than 5 cm (2 in) thick. These beds have been grouped into 15- to 46-m (50- to 150-ft) thick subunits of predominantly hard, brittle, vitreous quartzite and relatively soft, plastic argillite and sericitic quartzite (figure 1A). An in-depth study of rock and rock mass properties was recently conducted for each of these three rock types and their various combinations as found in formations and stratigraphic subunits of the Coeur d'Alene district (Whyatt et al., 1996a).

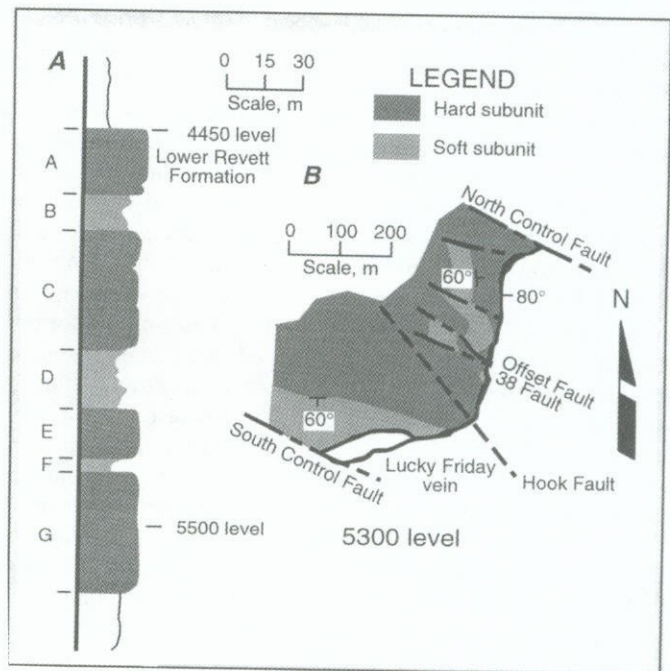


Figure 1.—Stratigraphy (A) and structure (B), Lucky Friday Mine.

In Situ Stress

The in situ stress regime at the Lucky Friday Mine was the subject of a recent in-depth investigation (Whyatt et al., 1995a-1995d). Considerable variation in the in situ stress field was observed among overcore cells and sites. Unusual stress conditions, including a rotation of horizontal principal stresses and a vertical stress component double that of overburden loading, were measured at a site on the 5300 level of the mine directly above the ramp sublevels examined in this study. Stress



conditions were confirmed by unusually intense and concentrated seismicity and rock bursting encountered during subsequent excavation in this area. Raisebore breakouts observed in a number of locations from the 5100 through the 5700 levels of the mine were mapped to establish bounds to the spatial extent of the stress field rotation. These results are plotted on a map of the 5300 level on the basis of lithology (figure 2). Breakouts in softer subunits showed the maximum horizontal principal stress oriented uniformly to the northwest. Breakouts and stress measurements in the harder subunits showed similar results, except in a service raise on the 5400 level and at the overcore measurement site on the 5300 level, both of which are adjacent to the Offset Fault. Thus, this stress perturbation appears to be limited in extent, but likely extends into a portion of the study area along the Offset Fault.

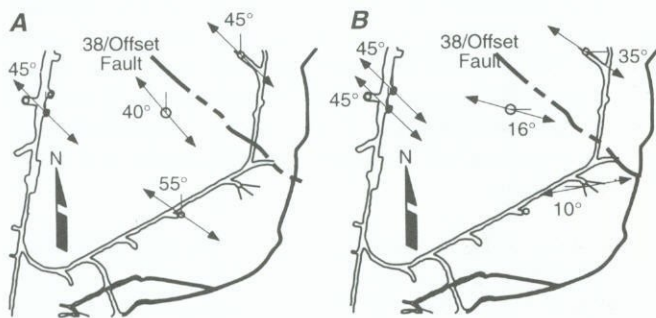


Figure 2.—Orientation of horizontal maximum principal stress at various locations projected onto 5300 level plan. A, Locations in relatively soft subunits; B, locations in relatively stiff subunits.

Seismic Monitoring Systems

Seismic activity was monitored by three seismic networks during the study period. The oldest of these systems, operated since 1973, is an analog microseismic system that determines the location of seismic events and includes a surface seismograph for estimating relative magnitudes of events. The development of powerful digital systems based on the personal computer (PC) provided an opportunity to supplement the microseismic system with full-waveform monitoring systems. The mine-wide digital seismic monitoring system (macroseismic system) was installed in 1989 (Girard et al., 1995), and the district-wide North Idaho Seismic Network (NISN) began operation in March 1992 (Lourence et al., 1993). Williams et al. (1995) provide a detailed summary of system components, geophone networks, and the types and format of data produced by these systems.

Data Screening and Sorting

The present study was based on a database containing records provided by the USBM, the University of Idaho, and Hecla Mining Co. Data was collected for events that occurred from mid-1990 through the first half of 1993. These data included records from all three seismic systems discussed above, rock burst damage reports, and the progress of stope and development

headings. Reports included, wherever possible, eye-witness accounts and on-site inspection of rock burst damage. This database grew to include information on several thousand events. Detailed event records and seismic first-motion data are too voluminous to be included here, but are available in a related paper (Whyatt and White, in progress).

The database was screened using a number of criteria to define sets of events of interest to this study. Data on events occurring outside the physical or temporal limits of this study were eliminated, and then the remaining data were sorted into three sets of events for further analysis. The first of these sets consists of events with the largest seismic magnitudes located within 91 m (300 ft) of a development face. A second set was defined to include events of all magnitudes but required a location within 30 m (100 ft) of a development face. A third set was defined to contain the 20 events with reported damage. These data sets are examined in the following sections.

DATA SET 1: LARGE SEISMIC EVENTS

The seismic energy released by large mining-induced seismic events has always attracted attention. These events have been associated with many large-scale failures, like the 5.2-magnitude event produced by the collapse of a portion of the Solvay Mine in Wyoming. As such, they are generally perceived as posing a great risk to miners. The seismic signals from these events can be detected at great distances, providing seismologists and ground control engineers with abundant information on event energy, location, and mechanism that is often not available for smaller events. Thus, the largest events, defined as events registering 30 mm or more on the surface seismograph trace (approximately 0.5 M_L), were chosen for the first group of events addressed in this study. The mine has found the 30-mm threshold to be useful for identifying the events most likely to damage the mine. Thirty millimeters roughly corresponds to the threshold for collection of digital seismic records as well. This set was also limited to events occurring within 91 m (300 ft) of a development face. Large events occurring farther than 91 m from the ramp face were considered unlikely to damage the ramp, and their occurrence would be unrelated to ramp progress.

Groups of Large Events

Twenty-nine large events were sorted into a number of groups with common characteristics, including location, activated geologic feature, and failure mechanism (figure 3). Damage to mine workings was reported for only six of these events, but for three of the six, the relationship proved to be only coincidental. Overall, these events accounted for surprisingly little damage and appear to be part of the rock mass response to mining rather than to development.

Vertical Slip Events: Macroseismic first-motion patterns showed that this set of six seismic events was generated by dip-slip movement, north block down, on two east-west-striking vertical fracture zones in the footwall of the 05 and 06 stopes. The district-wide NISN consistently detected implosional first motions. These events occurred over a single 45-day period and

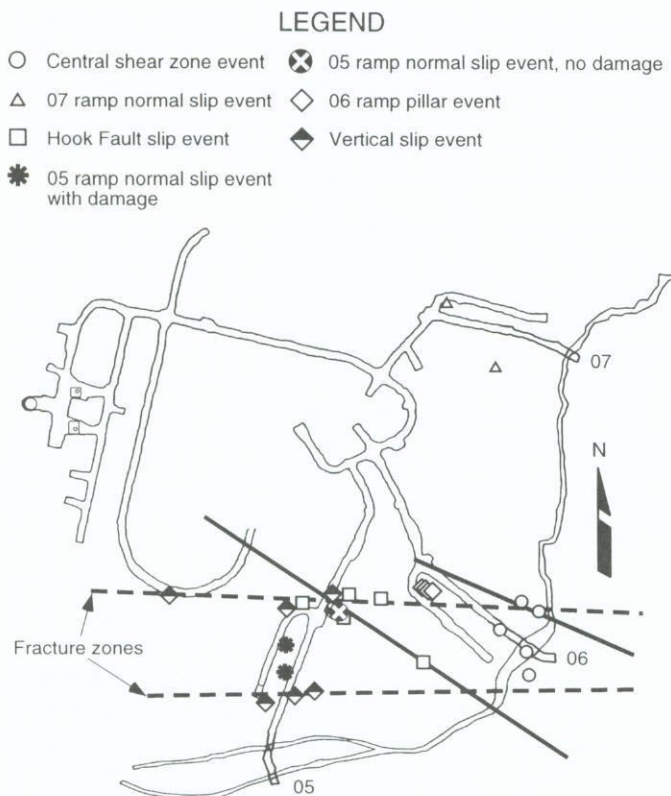


Figure 3.—Locations of large events assigned to various groups.

showed remarkable consistency in first-motion attributes. The discrepancy between systems can be interpreted as showing vertical shear on a plane of weakness with net deformations that significantly close (implode) the stope. The vertical relative movement (northern block down) of these events is the same as the offset observed between walls of the Lucky Friday vein. This sense of movement also matches the first-motion pattern observed in a series of very large ($2.5+ M_L$) vertical shear seismic events (Whyatt et al., 1996b) that occurred on the vein.

Central Shear Zone Events: This group of five seismic events occurred in a cluster near the vein at the southern margin of the central shear zone between the Hook and Offset faults. The group appears to be bounded to the north by the northernmost vertical shear zone. Macroseismic first motions for these events were primarily dilational, and the one event captured by the NISN appeared as an implosion. However, compressional first motions measured on a few macroseismic system geophones suggest left-lateral first motions for three of these events. Left-lateral movement is consistent with closure of the vein. Moreover, these movements are consistent with the stress field measured directly above these events on the 5300 level (Whyatt et al., 1995d). However, this sense of movement (and stresses measured at the 5300 level) is inconsistent with the northwest orientation of the regional stress field. Vertical slip on a plane parallel to the vein is indicated for another event, and a confused pattern was developed for the fifth.

Hook Fault Slip Events: This group of five seismic events occurred over a period of 60 days and is the first evidence that the Hook Fault, which is nearly coincident with the axial plane of the Hook Anticline, plays a role in the generation of seismic events. The macroseismic system and NISN both reported first-motion patterns dominated by right-lateral motion. The first event is interpreted as having occurred on the Hook Fault between the two vertical shear zones, while all four subsequent events were concentrated around the intersection of the Hook Fault with the northernmost shear zone.

05 Ramp Normal Slip Events: This group of four events occurred throughout the study period in two separate areas of the 05 ramp. First-motion information for these events was somewhat sparse, but generally indicated normal slip on bedding planes, which is a well-established mechanism for seismic events in the southern limb of the mine (Whyatt et al., 1996b). Spatially, the first two events, each of which damaged the ramp, occurred between the up and down ramps midway between the vertical shear zones. The second set of two events occurred at the same location near the intersection of the ramp with the northernmost vertical shear zone within three weeks of each other and did not damage the ramp.

06 Ramp Pillar Normal Slip Events: The first event in this group damaged the development face. Repair required removal of 100 mt of broken rock. While the magnitude of this event was measured, the seismic systems failed to provide a location or first-motion pattern. Three additional events occurred close to the first during the following 5 months. The events were located in a single 6 m (20 ft) diameter cluster in the vicinity of damage from the first event. This cluster was located just to the north of the northernmost vertical shear zone and just above the pillar between 5480-06 up and down ramps. The macroseismic system showed normal slip first-motion patterns, but not on any known plane of weakness. NISN reported implosional first-motion patterns for these same events, suggesting some reduction in excavated volume. The repeated occurrence of these events at the same location in the 06 ramp suggests that some feature, or set of features, probably including the ramp pillar, contributed to this concentration of activity. It also suggests that the hazard posed by the first of these events at the unsupported face was fully controlled by ground support measures installed prior to the subsequent large events.

07 Ramp Normal Slip Events: This group of two events occurred during a 10-day period. The events were located above the 5480-07 ramp and above the mining horizon. The in-mine macroseismic system generated a first-motion pattern showing normal slip on bedding planes for the first event, but missed the second. The regional NISN network reported an implosional first-motion pattern for the first event, but the pattern for the second event was confused. This discrepancy between near-field and far-field readings is similar to that observed in other sets of events and likely reflects initiation of movement along bedding combined with significant closure of the 07 stope. The proximity of event locations to the ramp suggests that the ramp might have had some influence on event initiation, even though the first motions appear to be unrelated to the ramp.

Interaction of Large Events with Excavation

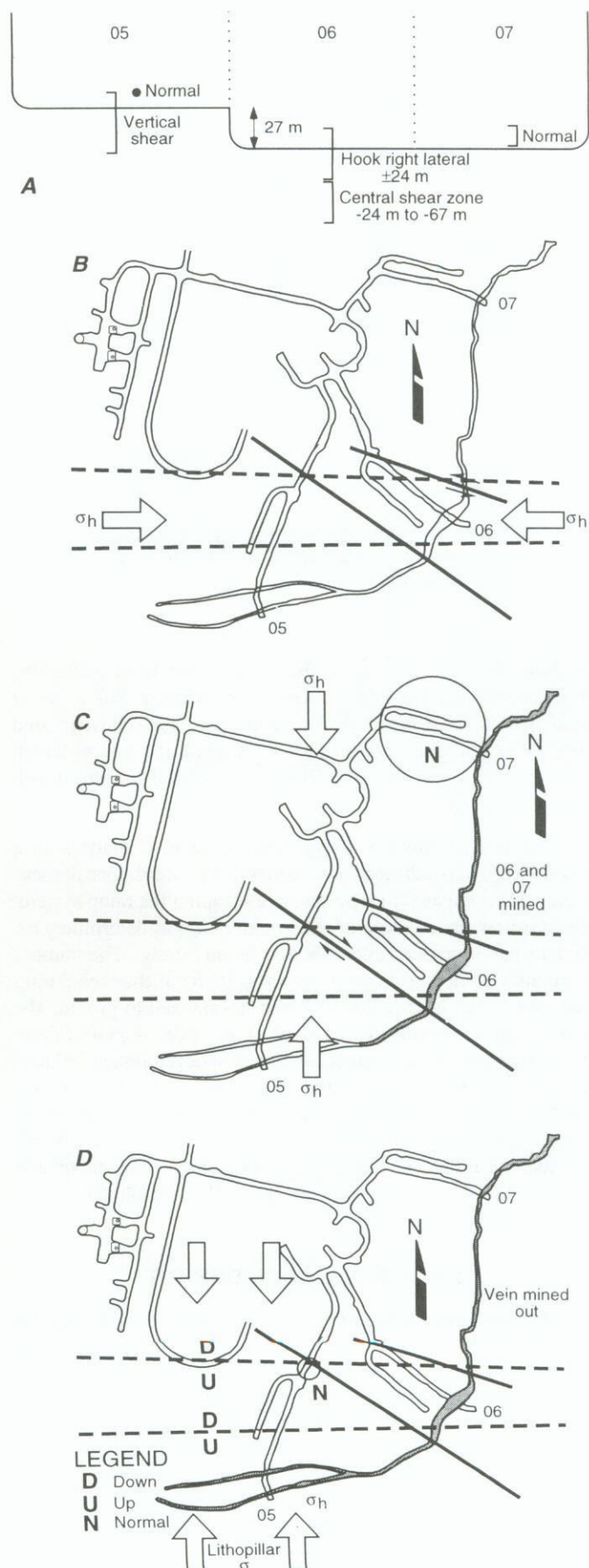
The link between a mining-induced seismic event and excavation at a particular location is not always clear. However, it is reasonable to expect that if an excavation truly triggers a seismic event, that event should be located in a region that undergoes a nontrivial change in stress field as a result of excavation. Only three of the 29 large events were located within 10 m (30 ft) (three diameters) of the ramp face, where ramp excavation could conceivably play a role in triggering these events. Damage to the ramp was reported with two of these events. First-motion information from the third event was characteristic of other events occurring at much greater distances from the ramp face, suggesting that the event was independent of the ramp and was only coincidentally located near the ramp face. The two damaging events, on the other hand, displayed normal first-motion patterns consistent with movement of rock into the ramp. Moreover, since both of these events were located beneath the mining horizon, the ramps were the only voids that could be closed by normal movement. These two events and, possibly, the unlocated event that damaged the 06 pillar, are the only cases in which ramp excavation appeared to trigger a large seismic event.

The relationship between large seismic events and mining can be examined by plotting the location of these events relative to the mining front (figure 4A). Each type of event appears to maintain a fairly constant position with respect to the mining front, and the first-motion patterns reported for these sets of events are consistent with the influence of mining-induced stresses (figure 4B-D). The deepest events were located at the southern end of the central shear zone, and relieved concentrated horizontal stresses below the 06 stope. The next major set of events, right-lateral events along the Hook Fault, was driven by relief of east-west horizontal stresses by mining in the 06 and 07 stopes and concentration of north-south stresses under the lagging 05 stope. Concentration of stress in the dipping beds was relieved through vertical shearing (figure 5), and vertical stress on bedding planes above the stope floor was relieved by normal movement toward the stope.

DATA SET 2: EVENTS AT THE FACE

A second set of events was formed from the large number of nondamaging seismic events detected during development of the ramps. Events were considered only if they occurred within a distance of 30 m (100 ft) of the development face. Detection by the seismograph was defined as the minimum seismic energy threshold. The rationale for examining these events was twofold. First, although these events did not cause damage, many occurred in similar locations and with similar levels of seismic energy

Figure 4.—Location of major groups of large events relative to mining face. A, Longitudinal section; B, deep events responding to east-west abutment stress; C, intermediate events responding to leading 06 and 07 stopes; D, uppermost events relieving stress in beds through vertical shear and vertical stresses by normal slip.



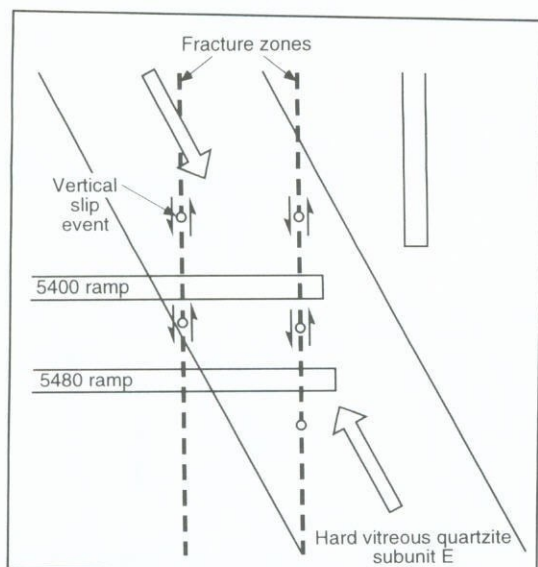


Figure 5.—Schematic showing the sense of vertical slip, stratigraphy, and stope in a vertical section.

to those that did. Some of these may have been caused by mechanisms that have little potential for damage, but some, at least, would have resulted in damage if a less robust ground support system had been installed. Second, the rate at which these events were generated is likely to be indicative of local rock mass conditions.

The large number of these events precluded analysis on a case-by-case basis. Instead, the overall rate of occurrence of these events was compared to progress in developing the ramp system. The position and advance of each ramp face was determined for each of the 36 months encompassed by this study. The number of small seismic events within 30 m (100 ft) of the face during that month was ascertained and then normalized to provide the number of events per 30 m (100 ft) of advance. A plot of these results (figure 6) shows two major zones of development-induced seismic activity. The first and westernmost of the zones lies entirely within a hard lithologic subunit and is cut off rather dramatically at the Hook Fault. A similar but more gradual cutoff is evident near the vein at the subunit boundary. The second and more centrally located zone is not as clearly constrained.

DATA SET 3: ROCK BURSTS

Data set 3 was defined for events that reportedly damaged a ramp face, rib, or back within the 5400 and 5480 sublevels. The set consists of 20 events that injured four miners and required removal of 750 mt (830 st) of rock during repair of the ramp back, rib, or face (at 10 to 100 mt per burst). Rib damage, reported in 11 of these 13 events, predominated. Back damage was reported in three events, and face damage in two events. Burst damage to the floor was reported for one event. The majority of these events are best described as strain bursts, and many appear to have been driven by buckling of slabs in the ramp rib or face.

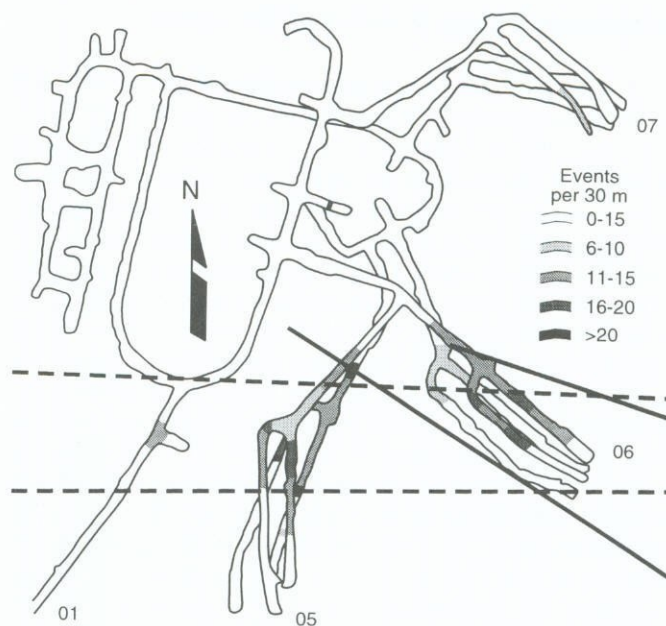


Figure 6.—Density of small seismic events

An eyewitness account of a small-magnitude rock burst in the 01 ramp on May 20, 1993, illustrates this mechanism. The burst occurred as two miners were drilling bolt holes in the rib to install bolts and chain-link fencing. One miner reported that his jackleg drill seemed to leap toward the rib, as if the drill steel had broken (it had not). In the next instant, the rib burst almost from floor to back along a 5-m (15-ft) section of the ramp to a depth of about 1 m (3 ft). A miner was bruised but not seriously injured, being protected somewhat by the fencing hanging between them and the burst.

An examination of the rib at the ends of the burst cavity revealed many closely spaced, joint-like fractures at a slight angle to the surface of the rib. These fractures had not been recorded during mapping of the rib several days earlier. Whether or not they were preexisting structures, their prominence after the burst suggests they at least opened as the burst occurred. The miner's perception that his drill leaped toward the rib suggests that quasi-static dilation of the rib occurred as a prelude to the burst. The movement was likely caused by fracturing and bending of rock layers defined by fractures prior to buckling of these slabs into the ramp.

A number of similar accounts have been gathered from other rock bursts. These accounts share the following observations:

- X Sudden, intense fracturing of intact rock into coffee-cup size or smaller rubble.
- X A loud, instantaneous report. The report is described as resembling an exploding charge or a sonic boom. For nearby observers, the sound seemed to originate at the immediate site of the burst. It seems likely that instantaneous fracturing of rock at the rock burst site is responsible for the sound.

- X Rock rubble expelled into the mine opening before miners could react. Initial expulsion of rubble is followed by a brief interval in which rock burst debris continues to be distributed about the burst site. Miners involved in these bursts are not generally knocked down by the ejected rock. Instead, they are engulfed by a fluid-like flow of rock debris. Miners have been partially or completely buried while standing erect. Their legs have been extensively bruised but not broken from the impacts of individual rock fragments.
- X A dense cloud of dust that immediately fills the air. Near a major burst, dust may be so dense for several minutes that miners have the impression that their lamps have gone out (Dolph, 1993).
- X An air pressure shock wave or "air blast" that travels through the mine. The initial air blast shock may be followed by a period of burst-induced wind that varies from very brief to sustained, depending on the change in excavated volume induced by the burst.
- X A concave cavity that narrows with depth. Weak discontinuities often form the back of the cavity, which is revealed when rubblized rock is removed. Damage reports examined in this study reported cavities ranging from 30 to 120 cm (1 to 4 ft) deep.

Rock bursts, like events in the first two sets, occurred in a few clusters involving only a small portion of the ramp system. Figure 7 illustrates the locations of these clusters and locations where these bursts resulted in injuries. The location of injuries within the clusters of rock bursts suggest that the level of hazard was influenced by sudden onsets of rock bursting that may have surprised miners. Adding distress holes to the round and adjusting support measures were probably responsible for the lack of subsequent injuries in these clusters, but detailed records of these measures were not obtained.

Figure 7 also indicates which rock bursts could be linked to records of a particular seismic event. This link could not be forged for a majority of reported rock bursts, including all bursts reported in the 07 ramp, and 3 of the 4 reported in the 06 ramp. Establishing a link between damage and a seismic event can be difficult, particularly when the event has little seismic energy or occurs with blasting. The time at which the ramp is damaged can be difficult to ascertain if a section of ramp is idle when the damage occurs and the seismic event is not unusually energetic. Finally, it is also possible that some of these rock bursts, particular those with minor amounts of reported damage, might have occurred independently of a seismic events and, thus, be more appropriately described as falls of ground. However, there was no correlation between amount of damage and seismic magnitude within this data set. In fact, many of the most damaging bursts could not be linked with a seismic event while while one of the least damaging bursts was included in the set of large seismic events.

Most of the rock burst clusters fell within the footwall of the southern limb of the Lucky Friday vein where both large and small seismic events were concentrated. However, bursting in the

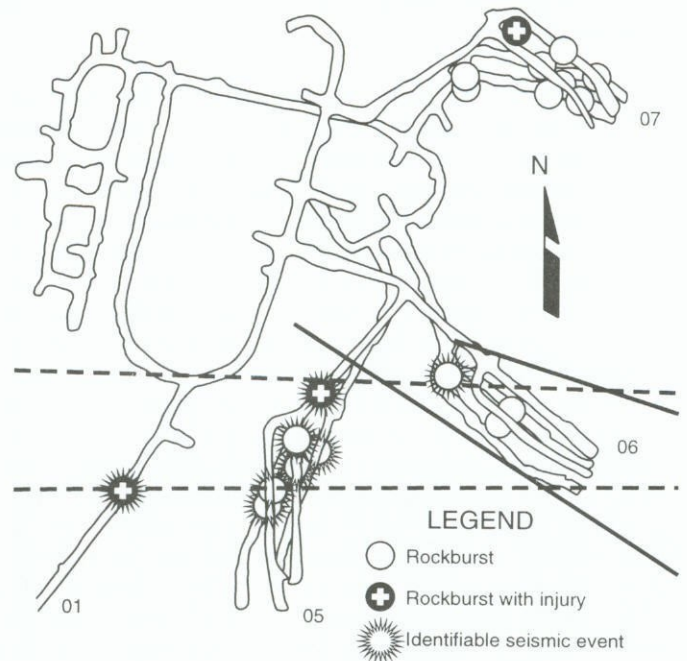


Figure 7.—Locations of rock bursts and rock burst injuries.

07 ramp was a notable exception. The scarcity of both large seismic events (data set #1) and seismic events in general (data set #2) in the 07 area suggests that stress levels were lower than in burst-prone zones to the south. Steeply dipping bedding and a steeply dipping fault both cross the 07 ramp at low angles and appear to have severely weakened ramp ribs. These bursts were largely eliminated from the 07 ramp in subsequent sublevels by reorienting to cut bedding at high angles and repositioning to avoid the fault.

DISCUSSION

This study was designed to examine rock burst hazards encountered during development of just over a mile of ramp within the 5400 and 5480 sublevels of the Lucky Friday Mine.

The study examined 29 large seismic events, hundreds of microseismic events, and 20 rock bursts. Repair of damage caused by these bursts required removal of 750 mt (830 st) of broken rock. Eyewitness accounts were used to build a composite description of a typical rock burst. These accounts indicate that most of these bursts were caused by buckling of rock plates defined by preexisting or stress-induced discontinuities within the ramp rib or face. These bursts can best be described as strain bursts. They generated low levels of seismic energy and did not meet energy threshold criteria useful in other areas of the mine for identifying potentially damaging events. However, the spatial distribution of large and small seismic events did correlate with rock bursting in some areas. The 07 ramp is an exception and may reflect a difference in burst conditions. The 07 ramp bursting occurred largely as a result of location and orientation relative to weak bedding planes and a fault. Intense seismicity and rock bursting were correlated in areas that were highly stressed, as shown by the 5300 level stress measurement, and

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were developed at favorable orientations to structure in strong vitreous quartzite.

Nearly all of the large events examined in this study were driven by mining of the Lucky Friday vein. The relatively simple mining geometry of the underhand longwall front combined with a fairly complex geologic setting to drive a rich mix of seismic events and, potentially, rock bursts. The influence of geologic features on seismicity is clearly displayed in first-motion patterns that show slip on a number of faults, bedding planes, and fracture zones. This influence is also apparent in the bounding of seismic activity by shear zones, faults, and subunit boundaries. Mining clearly concentrates the stress available to drive many of these events, but there is a marked difference in the seismic response to the concentration of stress between the two limbs of the mine. One explanation suggested by earlier in situ stress studies is that the level of premining stress differs through the rock mass. Thus, the highly stressed, and stress concentrated, southern limb is seismically active, while the quieter east limb lacks high levels of premining stress, and the deep footwall lacks a mechanism for concentration of premining stresses. Further work is needed to discern how varying initial stress fields can be reasonably included in an assessment of the spatial distribution of rock burst risk.

The lack of damage from large events reflects well on the ground control measures that have been applied. Outside the 07 area, most damage was recorded at the unsupported face or in immediately adjacent ribs, often before or during installation of support. The 06 pillar events are a case in point. The first event caused significant damage to the face, but three subsequent events were successfully contained by the support system. It also appears that injuries tended to occur when an area of intense rock bursting was first entered and that changes in practices were successful in preventing subsequent injuries in each of these areas.

The sources of information available to this study, including conventional seismic monitoring systems, geologic mapping, damage observations, and good background information on rock mass properties and in situ stress, were sufficient to identify geologic controls that spatially concentrate seismic activity and rock bursting. Recognition of the role of particular geologic features in the spatial distribution of rock burst hazards provides an opportunity for anticipating, rather than reacting to, a changing level of rock burst hazard. However, much work will be required to make good on this promise.

ACKNOWLEDGMENTS

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Advances in Remote Sensing Techniques for Monitoring Rock Falls and Slope Failures

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ABSTRACT

Ground control problems at surface mining operations can occur for a variety of reasons. Stress, gravity loading, rock strength, geology, pore pressure, weather effects, underground workings, and many other factors contribute to slope instabilities that range from small rock falls to massive slides of material. While some of these failures can be predicted or controlled by preventive measures, each year many completely unexpected failures occur. Current methods for monitoring generally involve measuring displacements at a few, selected points in and around the suspected area of instability. While most of the displacements along these points will be in a downslope direction, freeze-thaw cycles of water-filled joints, horizontal stresses or pressure, buoyancy in saturated soils, human measurement errors, or other situations can produce deformation in almost any direction, even without any instability in the slope. Determination of which, if any of the observed movements represents a potential hazard is essential.

Because of the enormous surface area of many large open-pit mines, several varieties and scales of instability can occur. Small, unexpected rock falls may indeed be more hazardous than a massive failure that involves slow displacement of material over a longer period of time. Complete vigilance to detect all small potential falling blocks is neither feasible nor economical and certainly is not attainable using today's most common point displacement monitoring techniques. As part of an on-going study at the Spokane Research Laboratory, several new methods for monitoring slope instabilities are being investigated. This paper describes the potential adaptation of systems such as interferometric synthetic aperture radar, imaging spectroscopy, and time-domain reflectometry, to slope monitoring and design.

INTRODUCTION

In February 1997, the Spokane Research Laboratory, of the Office for Mine Safety and Health Research, initiated a research

program with the goal of reducing the number of injuries and fatalities resulting from slope failures at mines. To distinguish the number of incidents associated with slope failures, Mine Safety and Health Administration (MSHA) narratives of 1,293 fatal accident investigations from 1978 to 1997 were reviewed. The results of this data analysis showed that 85 deaths (approximately 6.6% of all surface mine fatalities) were attributable to surface ground control problems. Each year since 1978, at least one, and as many as 11 miners, have been killed in slope failure accidents.

While MSHA fatality data is useful in approximating the danger level associated with slope instability, actual risks from surface ground control problems are much higher than the statistics indicate. Many falls of ground at surface mines do not injure mine personnel or significantly affect operations; such incidents are seldom reported. Also, the MSHA classification codes can lead to ambiguity when trying to define which accidents are directly attributable to a slope failure. As an example, a haul truck could back up to a dump and go over the edge. Such an incident would be classified as a "powered haulage" accident. However, the truck may have gone over the edge because the berm was not adequate, because of operator error, because the rock slope failed, or a combination of factors. An MSHA report of surface powered haulage accidents (Fesak, et al., 1997) states, "Between January 1990, and July 1996, 136 trucks and other haulage vehicles overturned while dumping material at edges of dump locations. This type of accident occurred more frequently than any other." In addition, the report goes on to say, "Most frequently, the haulage vehicle backed onto unstable fill material that gave way..."

These analyses of fatality data indicated that surface mine ground control problems are directly responsible for a significant number of fatalities, and steps are needed to minimize hazards to workers. There are several means to minimize the chances of surface ground control failures: 1) safe geotechnical designs; 2) secondary support systems or rock fall catchment systems; or 3) systems for advance warning of impending failures. However,



Figure 1 -- Massive slope failure in the highwall of an open pit mine.

while geotechnical designs can be improved to increase factor of safety, and support systems can enhance the overall rock mass strength, even a carefully designed slope may be subject to instability. The objective of this paper is to briefly introduce the most common slope monitoring systems currently in use, describe the limitations of these systems, and present new remote sensing technologies that may be applicable to advanced monitoring and design of dumps, highwalls, tailings dams, and other slopes.

CONSEQUENCES OF SLOPE FAILURES

At any open pit mine, some degree of slope instability should be expected. Ground control problems at surface mining operations occur for a variety of reasons and instabilities can range from small rock falls to massive slides of material. Figure 1 shows a massive slope failure in the highwall of an open pit mine. While some failures can be predicted or controlled by preventive measures, each year many completely unexpected failures occur which result in fatalities, loss of equipment, significant changes to mining plans, and extreme costs to mining companies. Figure 2 shows a close-up of the devastating results of a much smaller, but unexpected, slope failure.

Open pit slopes are generally designed to a minimum acceptable safety factor. However, variability in rock strengths, hidden geologic structures, underground workings, gravity

loading, pore pressure effects, and many other factors can cause instability even in a "safe" slope. The fact that a slope is designed with some risk of failure must not be viewed as a disregard for safety. As long as the risks are known, and a sufficient, suitable monitoring system is provided, remedial engineering and safety measures can be taken (Call and Savely, 1990).



Figure 2 -- Unexpected slope failures endanger lives and demolish equipment.

TYPICAL SLOPE MONITORING SYSTEMS

Displacement measurements are the most common type of monitoring, complemented by monitoring of groundwater pressure. The objectives of a slope monitoring program are to: 1) maintain safe operational practices; 2) provide advance notice of instability; and 3) provide additional geotechnical information regarding slope behavior (Sjöberg, 1996). The following is a brief summary of the *most common* monitoring systems currently in use and is not intended to be an all-inclusive list of monitoring equipment.

Surface Measurements

Survey Network: A survey network consists of target prisms placed on and around areas of anticipated instability on the pit slopes) and one or more control points for survey stations. These stations need to be located close enough to the pit crest so that all prisms can be seen, and must be located on completely stable ground. The angles and distances from the survey station to the prisms are measured on a regular basis to establish a history of movement on the slope. Dust, haze, human error, damaged prisms, or displacement of the survey station can affect measurement accuracy.

Tension Crack Mapping: The formation of cracks at the top of a slope is an obvious sign of instability. Measuring and monitoring the changes in crack width and direction of propagation is required to establish the extent of the unstable area. Measurements can be as simple as driving two stakes on either side of the crack and using a survey tape or rod to measure the separation. One problem with monitoring cracks at the crest of a slope is that the movement has already occurred. Additional cracking will potentially weaken the entire area making the ground loose and measurements inaccurate. Existing cracks should be painted or flagged so that new cracks are easily detected on subsequent inspections. Further movement may also introduce safety concerns or limit access to the area. In an unstable area, care should be taken to quantify the failure mode and monitor the instability as a whole. Tension crack mapping at the crest is oftentimes not indicative of the extent or seriousness of the potential failure.

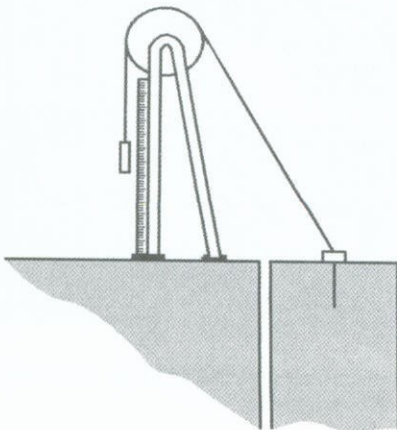


Figure 3 -- Wire extensometer

Wire Extensometers: Wire extensometers are simple systems that can be used to monitor areas of active instability. A common setup (figure 3) is comprised of a wire anchored in the unstable portion. The wire runs over the top of a pulley and has a weight suspended from the other end. As the unstable portion moves away from the pulley stand the weight will move, and the displacements can be recorded. The length of the extensometer wire should be limited to approximately 60 m (197 ft) because sag can produce inaccurate readings (Call and Savely, 1990). The weight of the wire used will also determine the amount of counterweight needed. Some wire extensometers are fitted with warning signals which are activated when a significant amount of displacement occurs. These signals were removed by mine personnel at one particular site visited by NIOSH researchers because birds kept landing on the wires and setting off false alarms.

Subsurface Measurements

Inclinometers: An inclinometer consists of a casing that is placed in the ground through the area of expected movements. The end of the casing is assumed fixed so that the lateral profile of displacement can be calculated. The casing has grooves cut on the sides that serve as tracks for the sensing unit. The deflection of the casing, and hence the surrounding rock mass, are measured by determining the inclination of the sensing unit at various points along the length of the installation. Inclinometers are relatively expensive and are not well suited to routine monitoring.

Extensometers: An extensometer consists of tensioned rods anchored at different points in a borehole. Changes in the distance between the anchor and rod head provide the displacement information for the rock mass. Extensometers are most suited to measuring deformation behind retaining structures. Since conventional extensometers only measure deformation parallel to the borehole, they are of less benefit in slope applications than inclinometers which measure deformation normal to the borehole.

Piezometer: Piezometer are used to measure pore pressures and are valuable tools for monitoring the effectiveness of mine dewatering programs. Excessive pore pressures, especially water infiltration at geologic boundaries, are responsible for many slope failures. Data on water pressures is essential for maintaining safe slopes.

REMOTE MONITORING TECHNOLOGIES

Synthetic Aperture Radar

Synthetic aperture radar (SAR) is a type of ground-mapping radar originally designed to be used from aircraft and satellites. Exploration geologists have benefited from SAR imagery since the 1970's (Rossignol and Corbley, 1996). SAR can be used to generate terrain maps, to produce high quality digital elevation models (DEM's), and to detect surface disturbances or changes in



surface moisture. SAR instrumentation that works from a ground level has been developed recently.

A variation of SAR -- Interferometric Synthetic Aperture Radar (IFSAR) -- uses differences in time-lapsed SAR images to generate maps of displacements (Fruneau and Achache, 1996). This technique has been successfully applied to produce displacement maps of ground movement caused by earthquakes and volcanic activity (Massonet, 1997). Figure 4 is an example of a displacement map generated as a result of the 1992 Landers earthquake. The interference fringes indicate displacements.

IFSAR has also been used to measure sea ice movement in Antarctica (Goldstein et al., 1993), model ocean wave patterns,

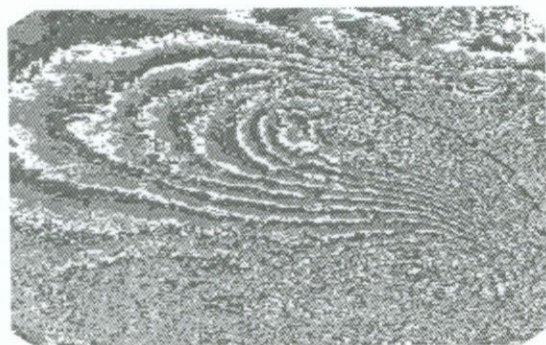


Figure 4 -- Interferogram of earth displacements along a fault.

monitor and map clear-cut or fire damage to forests, map the extent of flood damage and produce hydrologic models. IFSAR can also be used to monitor displacement of unstable slopes or landslides.

IFSAR's have many advantages over current types of monitoring systems. Able to work in nearly all weather, IFSAR can acquire imagery through fog, mist, rain, haze, or cloud cover, and can operate day or night. Also, they sample a large area for ground displacement which gives them a tremendous advantage over survey networks, surface extensometers and other instruments that sample movement only at a few, select points. Recent developments in instruments such as prismless laser range finders partially address the problem of undersampling large areas for movement. However, the range and accuracy of these units can vary greatly depending on the reflectivity of the rock, the angle of the rock face, weather and other factors. Manufacturers of prismless range finders generally claim a range of 500 meters or less.

For the purpose of slope monitoring at mines, IFSAR could be used to produce displacement maps of the *entire* highwall. In addition, information could be obtained on soil moisture content and the effects of dewatering could be tracked. Volume estimations from IFSAR-generated DEM's could be used for production purposes, and information obtained on the delineations of geologic structures would provide input for design purposes and possibly even ore control.

Imaging Spectrometry

In addition to displacement monitoring, information on geologic structures and weak rock units that have the potential for causing slope instabilities need to be incorporated in the monitoring program. An imaging spectrometer is a device that can determine the composition of rocks from a distance by their spectral signature. Like IFSAR, imaging spectrometers have been used from satellites and aircraft for geologic mapping, but recent advances in technology have led to the development of smaller units that can also work from a ground level. Spectrometers can also be used for such diverse applications as monitoring pollution or hazardous waste spills and monitoring the health of replanted vegetation. Some simple spectrometers are already being used in the mining industry to log drill core, identify rock in outcrops or exposed faces, and for petrographic analyses in the laboratory.

The need for geologic mapping of mine areas needs to be improved. The design of mine slopes, the placement of waste dumps, tailings dams, mine roads, etc. is based on geologic maps that show the location, attitude, and character of geologic structures and ore-bearing rock units. However, mine maps can vary greatly in quality and detail due to the subjectivity of various geologists, and the extreme geologic complexity of many deposits. In addition, there are financial and practical limits to the number of geotechnical samples that can be taken for analysis and engineering purposes, so much of the data shown on geologic maps is an estimation by a geologist or a mathematical interpolation of geotechnical results. Figure 5 is a spectrometer image clearly delineating fault areas, folds, and numerous rock types.

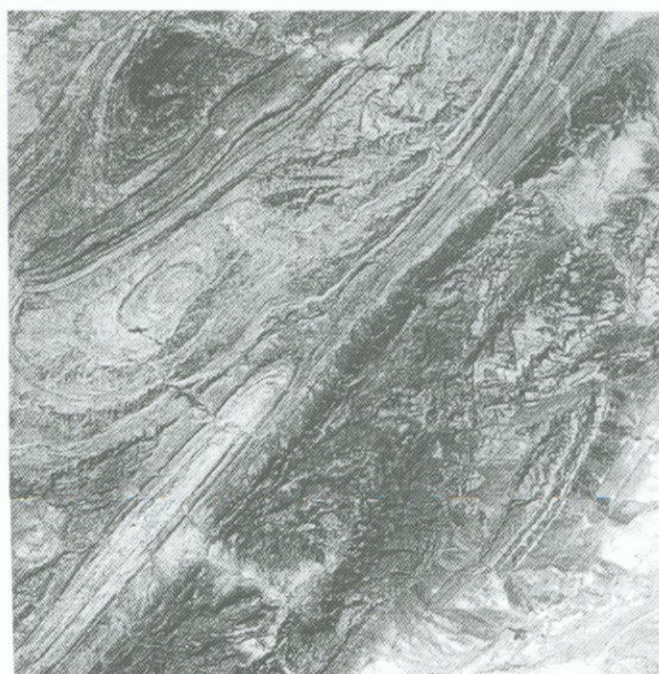


Figure 5 -- Spectrometer image.

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For the purposes of slope stability, hydrothermally-altered rocks, large faults or other geologic structures can be responsible for slope instability. Major features are generally well-mapped, but weak rock units may be much more difficult to identify. Imaging spectrometry provides a means of mapping highwall or other surface geology, without the subjectivity of varying geologists interpretations. The information on geology couple with IFSAR information on displacements could identify rock units responsible for causing instability. Recognition of similar situations encountered as mining progressed could be accounted for before instability occurred.

Time Domain Reflectometry

Time Domain Reflectometry (TDR) is a technique in which electronic pulses are sent down a length of a coaxial cable. When a deformation or a break in the cable is encountered, a signal is reflected. Early research on TDR began in the early 1960's to locate breaks in electrical power cables (Moffit, 1964). Panek and Tesch (1981) and O'Connor and Dowding (1984) were among the first to use TDR to measure rock mass deformation.

Inclinometers are currently the most common method of monitoring subsurface deformation in unstable slopes. However, TDR cables are rapidly increasing in popularity and have many advantages over traditional inclinometers (Kane, 1998) as follows:

- **Lower cost of installation.** Cables cost 2% to 38% less than inclinometer casing.
- **Deeper hole depths possible.** Inclinometers in deep hole require special winches and cables due the extreme weight of the equipment. All TDR monitoring equipment is at the surface.
- **Rapid and remote monitoring possible.** TDR data can be transmitted via telecommunications (Campbell Scientific Inc., 1998), and scanning and recording intervals can be programmed remotely to examine zones of interest.
- **Immediate deformation determinations.** Locations of any movement are determined immediately using TDR. Additional data reduction is generally not necessary, and the cables can be used to quantify rock movement as well as distinguish shear and tension (Dowding, et al., 1989).
- **Complex monitoring situations.** TDR cables have been installed in angled boreholes, and have monitored deep zones below moving upper zones. Neither installation could have been done with a traditional inclinometer.

Recent advances have also been made in the use of TDR for monitoring ground water levels and piezometric pressures (Dowding, et al, 1996). A summary of applications of TDR in the mining industry is provided by O'Connor and Wade (1994).

CONCLUSIONS

Remote sensing, at the present time, cannot replace conventional geotechnical methods of investigation, but emerging technologies should not be ignored. Interferometric synthetic aperture radar, imaging spectrometry, and time domain

reflectometry are already widely used to monitor hazards such as landslides, volcanoes, and active faults. As computer processing speeds increase and technology continues to advance, integration of remote sensing into mine operations can improve slope design and reliability of slope monitoring.

Our research team is interested in exploring the benefits of using multiple remote sensing methods to better understand slope failures. IFSAR coupled with an imaging spectrometer will provide detailed geologic and topographic maps and will detect and monitor hazardous earth movements, day or night, in nearly all weather, along the entire slope surface. Field tests of imaging spectrometry and interferometric radar are scheduled for two copper mining operations this summer. Results of the field tests will be available from the authors early in 1999.

Steps need to be taken to reduce the number of mining deaths resulting from slope instability. Advancement of monitoring and early warning systems is merely one option. The Slope Stability Hazard Recognition Team at Spokane Research Laboratory is pursuing many other research options to minimize the dangers associated with surface ground control problems. Results of this research will benefit others involved in the study of landslides, rock falls, avalanches, volcanic activity, and other geohazards.

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