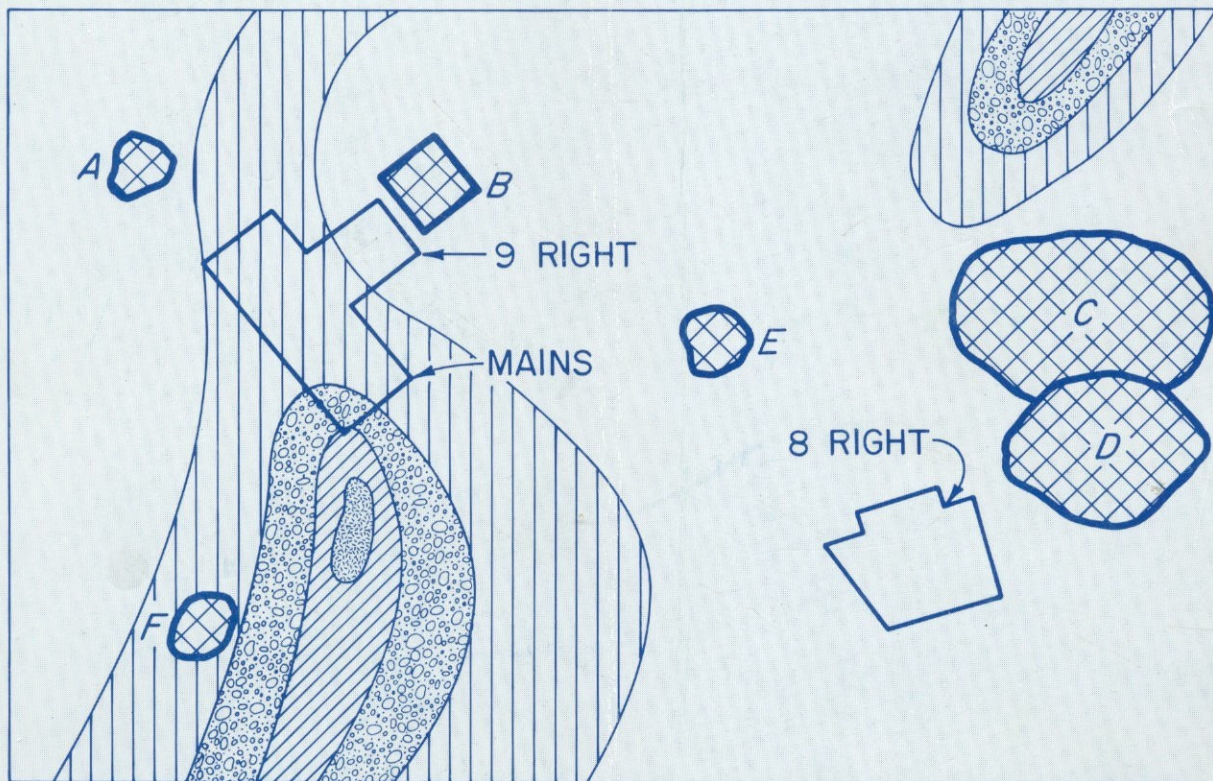


PROCEEDINGS

6th INTERNATIONAL CONFERENCE ON GROUND CONTROL IN MINING

June 9 - 11, 1987



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MICROSEISMIC MONITORING OF MOUNTAIN BUMPS AND BOUNCES: A CASE STUDY

by J. L. Condon and R. D. Munson

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ABSTRACT

The Bureau of Mines, through in-house and contract research, monitored mountain bump-prone areas of the Olga #2 Mine, near Welch, WV, using microseismic techniques for 15 months during 1985 and 1986. During this period, two ground failures occurred that were considered "mountain bumps" owing to excessive stress buildups. One failure damaged the roof and 4 pillars around an intersection; the second failure, which released much greater energy, resulted in significant damage to over 100 coal pillars.

Analysis of the microseismic data collected provided significant insights into the magnitude and location of each failure. The actual failure zone was clearly delineated prior to the mountain bump occurrences through analysis of source locations of each microseismic event. Trends in cumulative rate plots indicated the onset of structural instability. Although the data could not be analyzed automatically, the significance of the data interpretation cannot be overemphasized. Microseismic monitoring of bump-prone areas can be successful in providing a means to actually observe the structural stability occurring in underground coal mines.

This paper discusses data collection techniques, analysis methods, and implications of the monitoring technique, with special emphasis on actual data from two large mountain bumps in West Virginia.

INTRODUCTION

The Bureau of Mines has been conducting research in microseismic monitoring to study rock bursts in hard-rock mines for many years. Most of this work was conducted in ASARCO's Galena Mine in Wallace, ID, and resulted in technology

that, in some cases, could provide warning of imminent rock bursts. More recently, this technology has been adapted to study coal mine bounces, bumps, and mountain bumps, first in Mid-Continent Resources' Dutch Creek #1 Mine near Redstone, CO, and more recently in Olga Coal Company's Olga #2 Mine near Welch, WV. The Dutch Creek #1 Mine is an advancing longwall, while the Olga #2 is a retreating room-and-pillar mine which provides optimal conditions for microseismic studies. This paper will discuss the specifics of the mountain bump research conducted in the Olga #2 coal mine. The authors would like to acknowledge Jozef Descour and The Colorado School of Mines for work in securing this data through a Bureau of Mines contract.

We have used the terms "rock burst," "coal bounce," "bump," "mountain bumps," and "bounces." These terms mean different things to different people or different mines, and frequently have regional or local definitions. However, they all refer to basically the same phenomena of violent ground failure induced by excessive stress buildups in the geologic media of the mine structure. For this paper, we shall use the terms interchangeably.

Mining removes coal and rock material that previously helped support the weight of the rock mass above the material removed. This inevitably causes a stress redistribution to support the weight of the rock strata above the mine opening. Frequently, this redistribution results in a stress concentration in the rock that exceeds the rock strength with a resulting structural failure and loss of stability. Every structural failure in the rock strata is the source of a transient microseismic wave field that develops in the surrounding geologic media. The wave field contains information about the ground failure itself and about the stress conditions along the wave path from the

source to any detection point. This information is vital to locate the source (failure) and to reconstruct its mechanical characteristics.

Under specific conditions, the development of instability leads to a violent rupture, a "coal bounce," inside the stress field surrounding the opening. The complex time and space analysis of microseismic data with other types of information, i.e., mining situation, mining history, mining progress, destressing, visual inspection, other measurements, etc., is used for calibration of the microseismic method for hazard detection and prediction in a particular mine or mine section. This calibration is necessary to accurately delineate and forecast hazards related to coal mine bounces, to effectively control the strata, and to further study the mechanics of rock failure under the actual mine conditions.

Development of microseismic monitoring to its full potential for mine applications is only possible based on investigation under real mine conditions. Location of an individual source is undoubtedly the most fundamental piece of information to recover. It tells where the stresses exceeded critical levels of rock strength. An individual source location indicates the central point of the area from which a particular portion of the wave energy was radiated. Source location is basic for reconstruction of most of the other pieces of information contained in every seismic record.

In current practice, routine processing of multichannel microseismic data is mainly directed to detection and location of individual seismic events. Another fundamental piece of information is the microseismic activity function. It is important to know not only the location of microseismic events, but also the number of events occurring over a given time period. Increases in microseismic activity levels in a specific area can be precursors to major failures.

Therefore, the location of microseismic events gives us a clue as to where to expect problems, and the activity or event functions gives us clues as to when failure may occur.

MICROSEISMIC SYSTEM

The microseismic system used at the Olga #2 Mine consists of two independent sections. The event-recording section has a 14-channel, FM-analog, instrumentation-grade tape unit, signal-conditioning amplifiers and filters, and a time-of-day clock. The event-reproducing section uses

a compatible FM-analog tape unit, dc amplifiers, and an oscillograph. This arrangement allows data to be recorded in a nearly continuous mode, interrupted only by the short time required to change the data tape. The second tape unit also provides backup for the recording unit, should it fail. Figure 1 shows a schematic of the microseismic system as deployed in the Olga Mine.

At the end of each period of 24 to 26 hours, the tape is removed from the recording unit. Recorded events, along with the time-of-day, are retrieved by playing out the data tape on the second unit and producing an oscillograph record for each event. This requires about 6 hours to complete.

The paper records are then manually marked and timed. The arrival times of the signals are manually entered into a minicomputer which has the source location algorithm and data manipulation routines. Once a source location is obtained, the raw times and resultant location are stored on a floppy disk. Source locations are then manually plotted on the appropriate section of the mine map. The complete cycle requires about 3 days.

MINING AND GEOLOGIC CONDITIONS IN OLGA COAL MINE

The Pocahontas #4 coal seam has been mined in the Olga #1 and #2 Mines for more than half a century. Mining in the eastern part of the coal deposit, around Portal #1 near Coalwood, WV, was finished years ago. The mine in the western part of the deposit, near Portal #2 at Caretta, was extensively developed, leaving large coal isles surrounded by wide gob spaces. Historically, the tendency for mining to produce bumps has existed in Olga #1 and #2 Mines from the very onset of mining.

The Pocahontas #4 coal seam is approximately 6 ft thick, dipping generally to the west. The dip in the Olga #2 Mine ranges from about 3° in the southeastern part to approximately 1° at the center, increasing gradually to nearly 3° by the western perimeter of the mine area. The overburden averages some 1,000 ft, with variations from 500 to 1,500 ft, depending on the topography. The presence of a washout system that sharply cuts the coal seam into large, irregular sections significantly affects the structure of the mine support system in Olga #2. The washout system appears to align itself with surface topography. This has been considered as an indication of the tectonic history, which had an influence on structural uniformity and, therefore, on stress distribution in the rock formations.

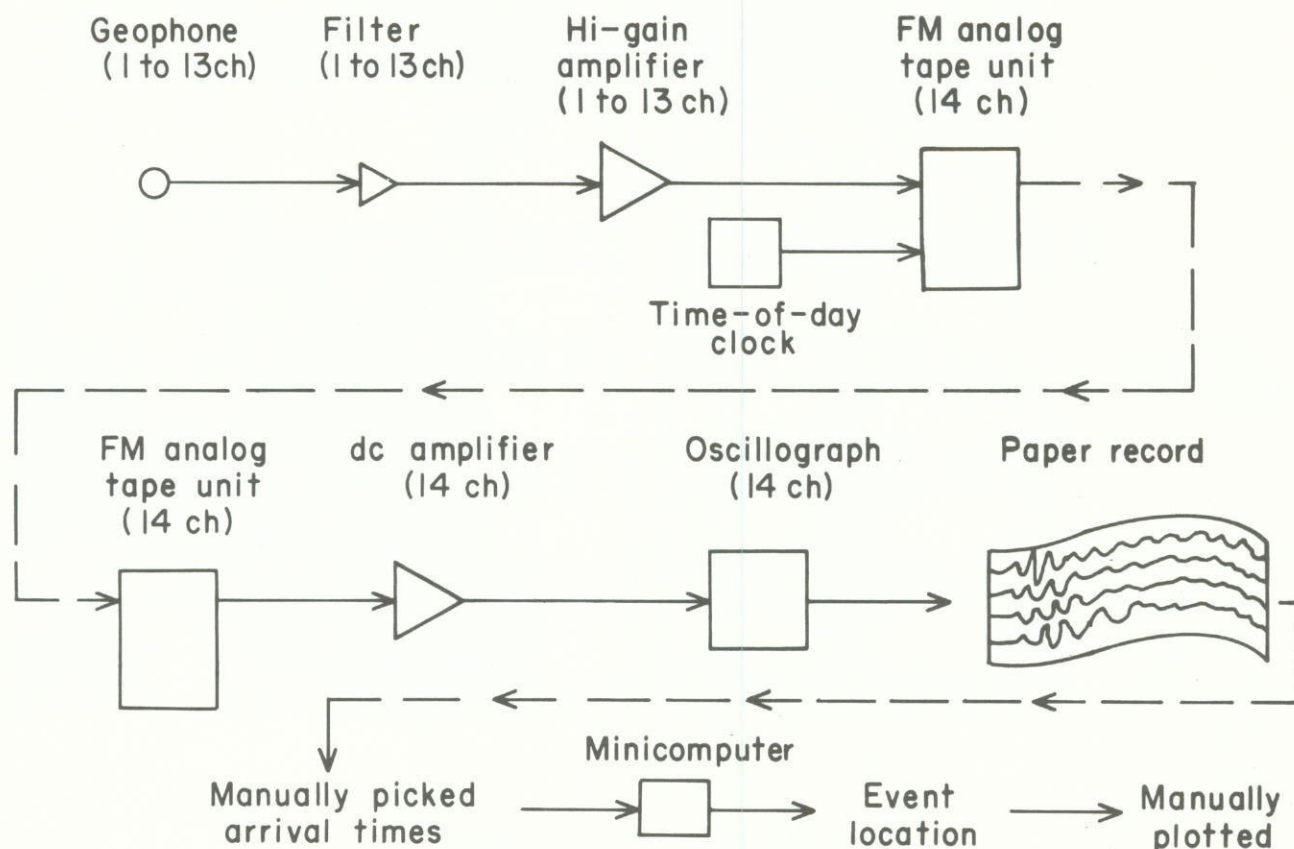


Figure 1. Schematic of the Microseismic System Deployed in the Olga #2 Mine.

INSTALLATION OF MICROSEISMIC INSTRUMENTATION

The primary goal in selecting individual geophone locations was to provide a geophone array that would surround a potentially active area of mining to assure optimum microseismic event locations. Figure 2 shows a typical geophone array. The conditions in the microseismic monitoring station were designed to satisfy strict environmental requirements for the protection of the sensitive microseismic monitoring equipment. The station was constructed close to the geophone array, in a location considered safe for continuous equipment operation and personnel working there. Cable routes were selected to avoid frequent changes due to mining progress and to reduce the risk of damage from mining operations, rock and roof falls, etc. Figure 3 is the microseismic monitoring station as installed in the Olga Mine.

It was decided that the active portion of the mine that best met all our requirements was the 9 Right section. To better understand some of the results from the microseismic monitoring in the 9 Right section, major changes in the mine structure prior to the period of monitoring should be considered.

The gob area south of the 9 Right section was completed in the late fifties. The western part of the large gob area adjacent to and northwest of the 9 Right section was developed in the early forties. The eastern part of this gob was developed in 1975. The development of the area into standard blocks of 50 by 70 ft with entries 20 ft wide was completed by 1965. The mining in this area did not start until 1983. A strip of barrier pillars was developed into standard blocks for retreat mining during the first quarter of 1984. The mining method during the period of microseismic monitoring is illustrated in Figure 4. The microseismic monitoring system was installed in the 9 Right

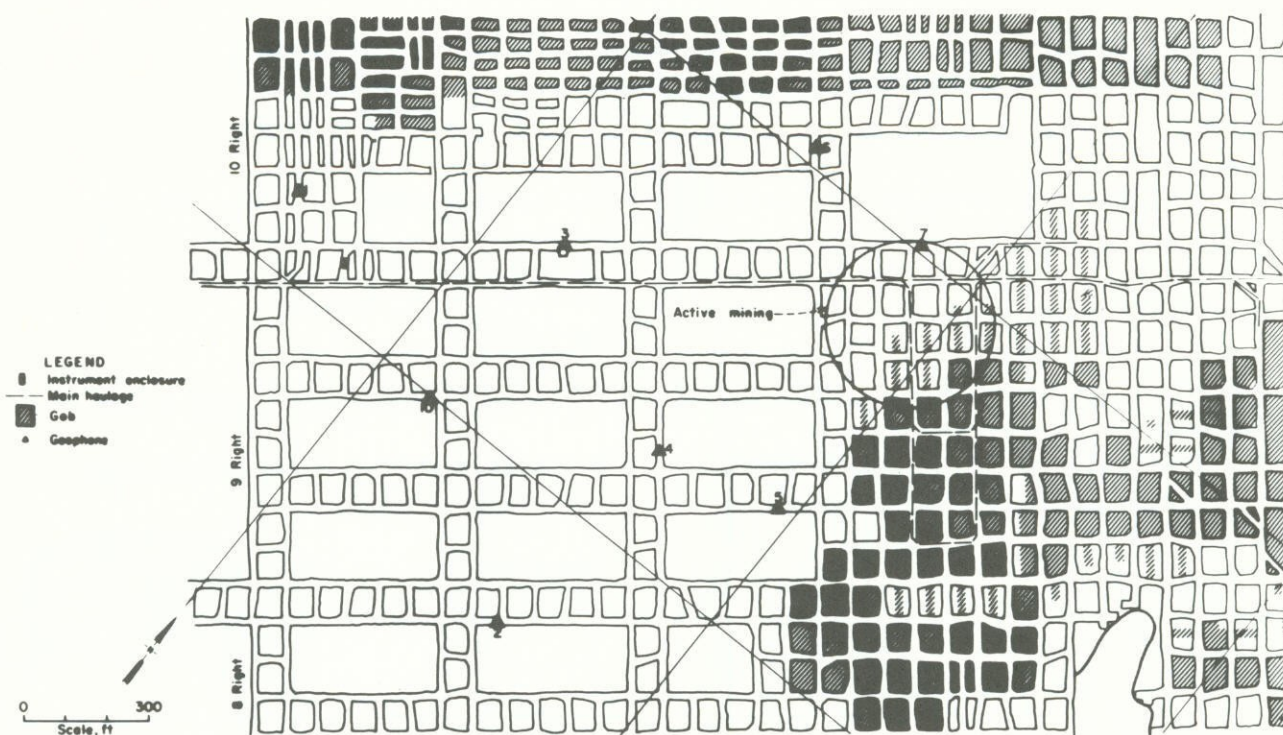


Figure 2. The Geophone Array in the 9-Right Section.

section at the end of January 1985. Regular monitoring covered in this report started on February 1, 1985, and continued until November 30, 1985.

RESULTS OF THE MONITORING

During the interval, March 1, 1985, to March 4, 1985, a pillar was cut into a regular pattern of coal blocks, characteristic for room-and-pillar retreat mining in the Olga #2 Mine (fig. 4). Cutting this pillar into coal blocks caused a substantial reduction in its supportive capacity. This resulted in a considerable stress concentration within the main stress system for the immediate area. This stress concentration led to the occurrence of a few moderate-scale bounces near the intersection at the western corner of the original pillar and floor heave some 130 ft southeast. A roof fall, which occurred 2 days later over an intersection almost in the middle of the former pillar, was considered part of the same stress rearrangement process.

Figure 5 shows the increase in microseismic events during the period, culminating in the bump of March 21, 1985. This buildup of events would indicate that possibly this damage-causing event may have had adequate precursors to provide a timely warning.

Figure 6 shows the location of the microseismic events, indicating a concentration of events in the area that was damaged.

The second significant event occurred during a mining sequence that began in late May 1985. The mining resulted in massive stress release and bounce occurrence which started at about 2:00 AM on June 3, 1985, and lasted several hours. In effect, approximately 100 coal blocks bounced, some of them more than once. The section was immediately closed by MSHA and remained that way until July 28, 1985.

An analysis of figure 7 shows that there was a massive buildup of microseismic events during late May and

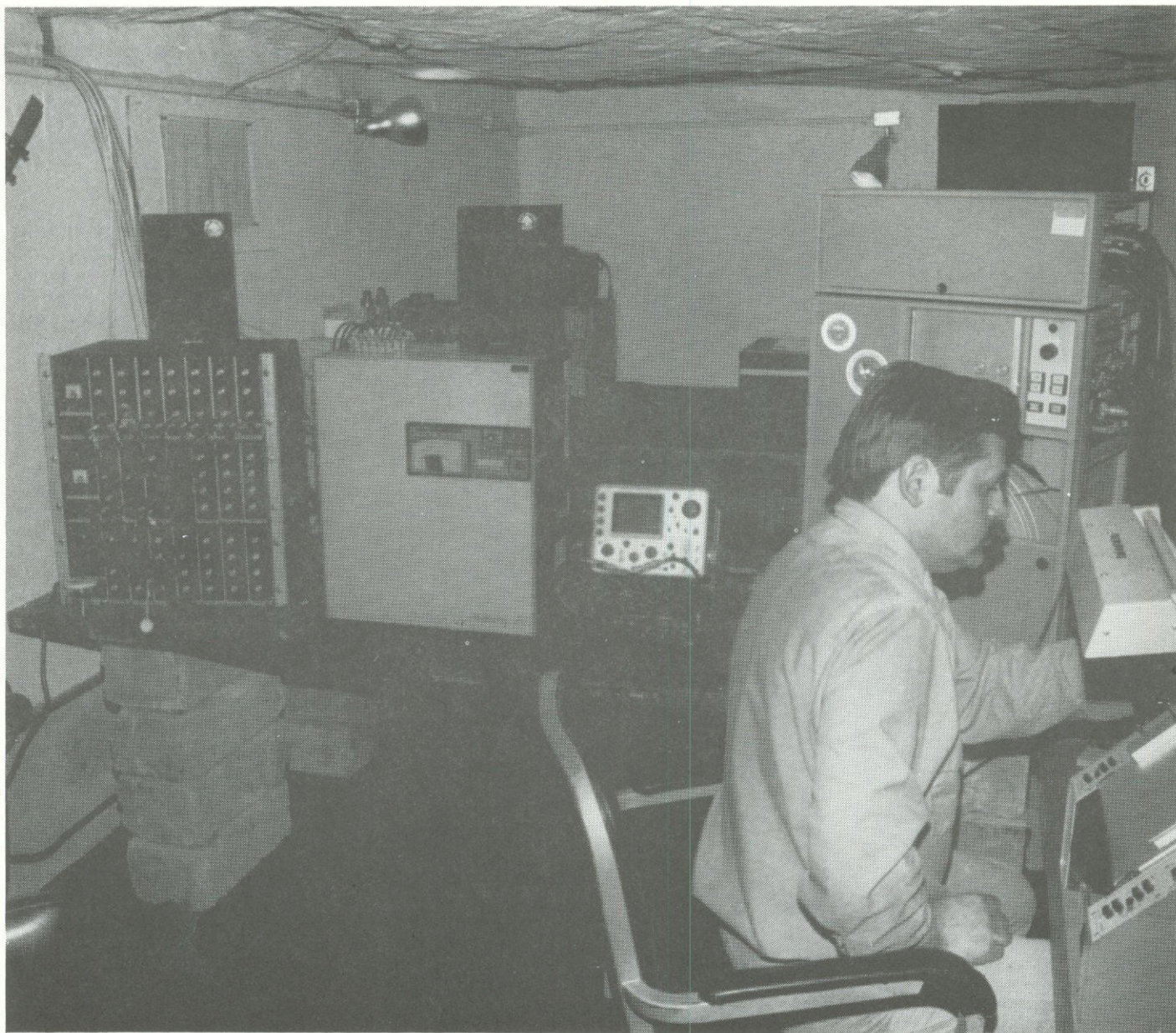


Figure 3. The Microseismic Monitoring
Station installed in the 9-Right
Section.

CONCLUSIONS

Microseismic monitoring of coal mines has the potential to provide warning of when and where bumps may occur. Further, it has potential as an indicator of stress buildup so that mining can be halted or altered to prevent damaging bumps.

This research demonstrates that acquiring adequate data to achieve these results requires a dense microseismic monitoring network surrounding the active mining area. A single, minewide system that monitors the entire mine would not provide adequate data for accurate forewarning of the time and location of impending bumps.

Furthermore, this study shows the necessity for monitoring with automated, rapid-analysis instrumentation. The instrumentation used in this study provided adequate detection and location of microseismic events, but did not provide information timely enough to provide advance warning of major bumps. The hand-picking and plotting of data from an analog tape required several days before the data could be analyzed. This delay was too long to enable us to provide any forewarning of the bumps. A look at figures 5 and 7 shows that the dangerous increase in microseismic rates occurs only a few days immediately prior to the bump. Therefore, new instrumentation and methods are required to provide useful, timely information to mine management.

The Bureau of Mines has assembled and is testing new state-of-the-art digital hardware to record and analyze microseismic data in near real-time. This system will be able to provide energy and spectral information that will give us greater insight into the dynamics of rock mass behavior and lead to more accurate and timely forecasting of bumps.

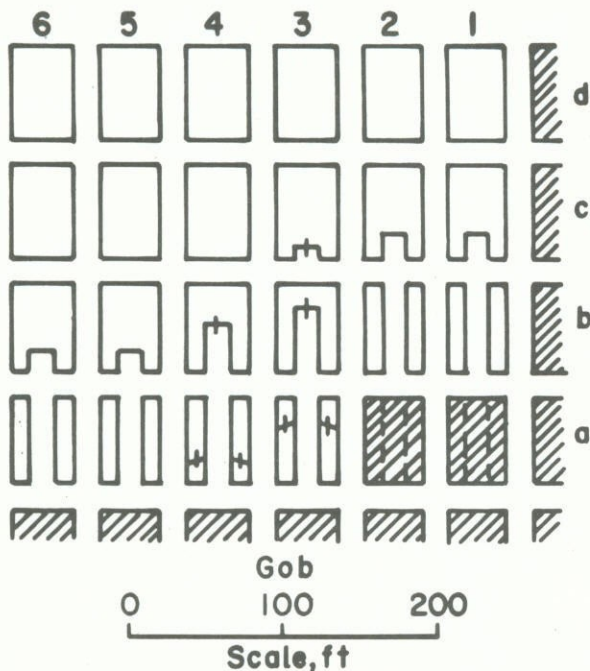


Figure 4. Standard Steps of Retreat Mining in the Olga #2 Mine: 3c, Initial Cutting; 1c, 2c, Bump Cuts; 3b, 4b, Cutting to Split; 1b, 2b, Split Coal Blocks 3a, 4a, Pulling Wings; 1a, 2a, Pillar Falls.

early June of 1985, reaching a level of 800 events a day on June 3, when the major bump occurred. This is an order of magnitude greater than the number of events that usually occurs. It was obvious from these data that the area was unstable several days before the bump.

Figure 8 shows the concentration of microseismic events that occurred over a large area. Almost all of the pillars within this area bumped.

These data contained enough information to indicate when and where the bumps would occur.

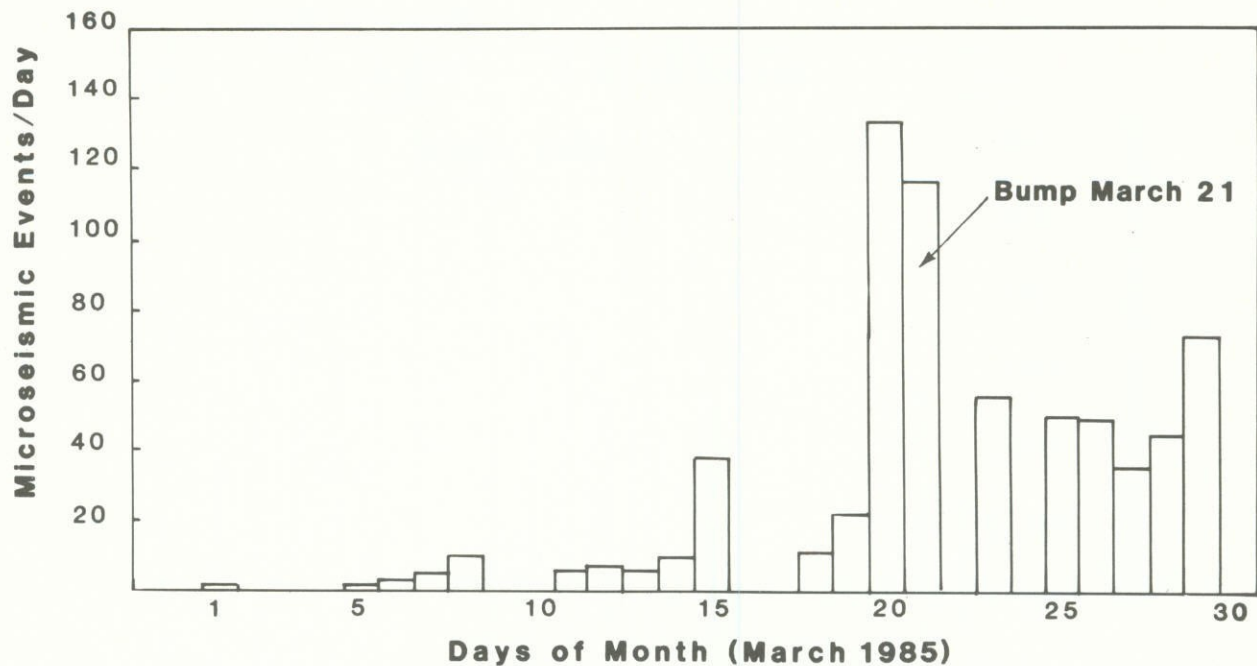


Figure 5. Microseismic Activity Levels During the Month of March 1985, Showing the Increase in Events Prior to the March 21 Bump.

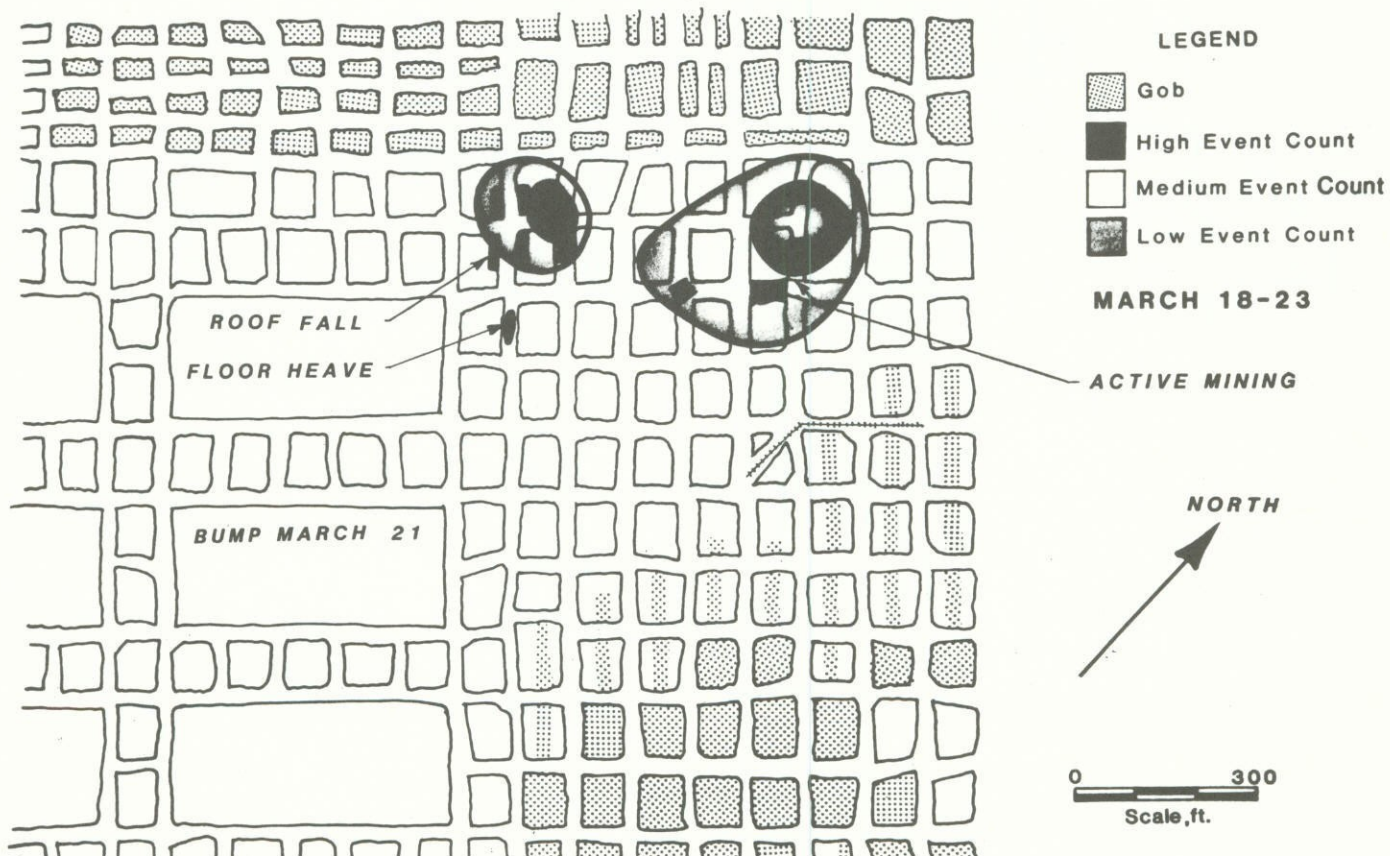


Figure 6. Location of Microseismic Events During the Critical Week of March 18-23, 1985.

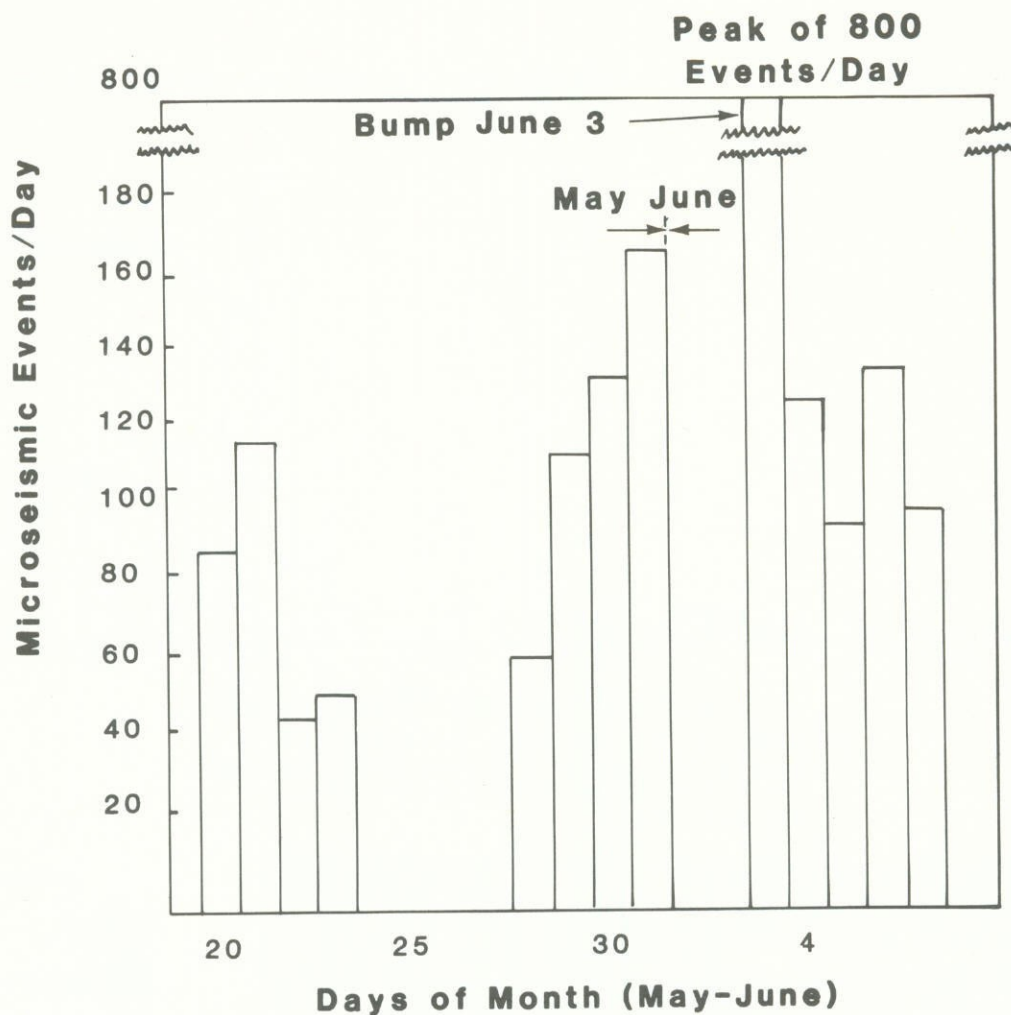


Figure 7. Microseismic Event Counts During the End of May and Early June 1985, Showing the Massive Increase Prior to the June 3 Bump.

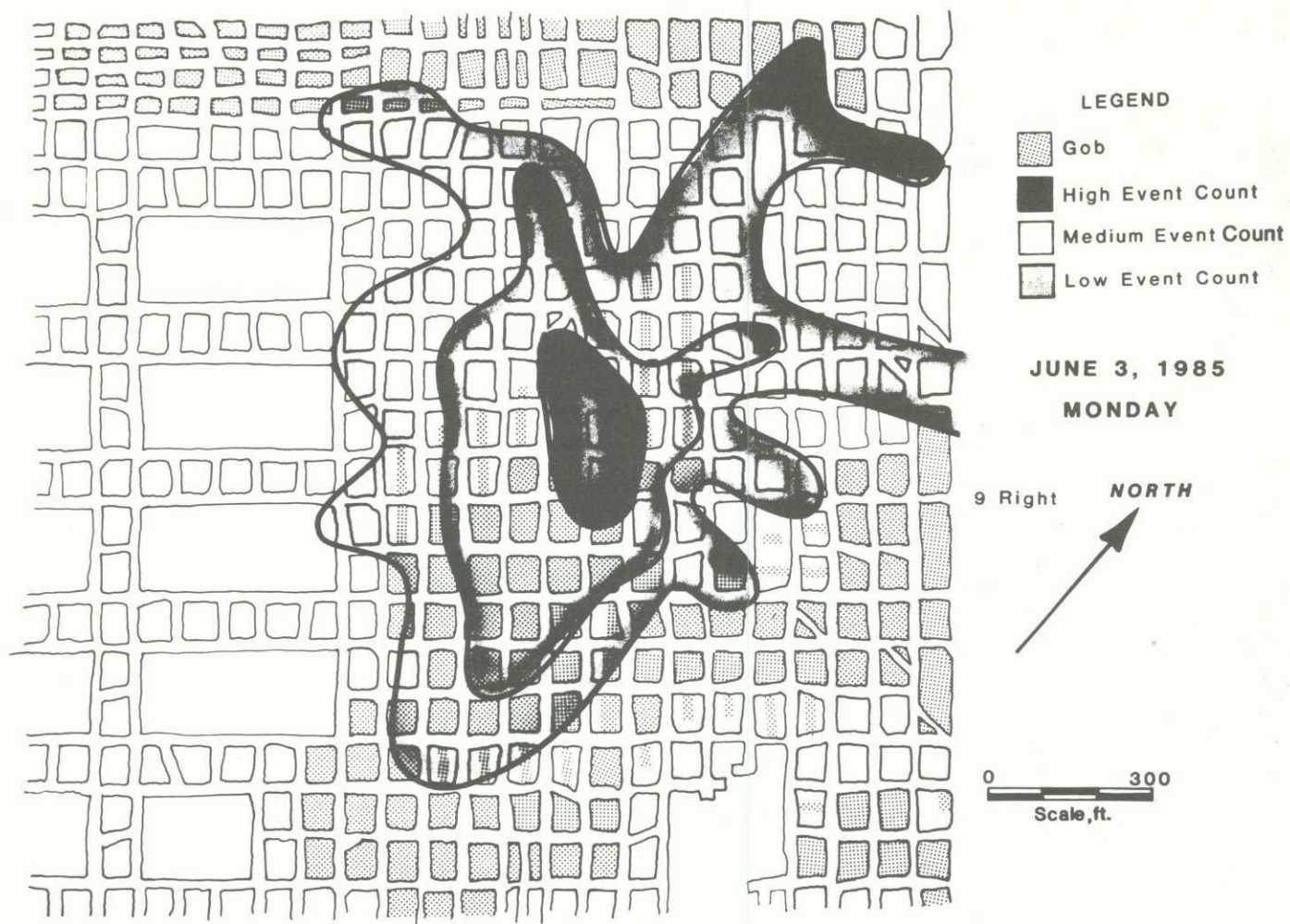


Figure 8. Location of the Microseismic Events Recorded Immediately Prior to the June 3 Bump. These Events Coincided with the Pillars That Bumped.

FACTORS INFLUENCING THE OCCURRENCE OF COAL PILLAR BUMPS AT THE 9-RIGHT SECTION OF THE OLGA MINE

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ABSTRACT

The Bureau of Mines conducted research at the 9-Right Section of the Olga Mine to determine the cause of bumps and aid in the development of appropriate control techniques. The purpose of this paper is to identify the factors which affect the occurrence of bumps at the Olga Mine. These factors have been broken down into two categories, in situ and mining related. The in situ factors include those conditions which exist prior to mining, such as variations in lithology and overburden, presence of jointing, the composition and the physical properties of the rock, and the original stress field existing in the coalbed and adjacent strata. The mining factors include the mining sequence, changes in stress in the coalbed and adjacent strata during mining, rates of convergence, and the pattern of failure of the coal and adjacent strata.

The factors which appeared to have the greatest effect on bumping were the overburden, the lithology of the roof and floor rock and the physical properties of the rock. The roof rock is generally a massive, stiff sandstone which does not break until coal pillar extraction is complete. The immediate floor is a strong shale which generally grades into siltstone and then into sandstone. The overburden at 9-Right ranged from 990 to 1,600 ft.

Secondary factors appeared to be the in situ (pre-mining) stress and the mining sequence. When barrier pillars were split, pressures measured in hydraulic cells (Borehole platen flatjacks) located in those barriers showed large pressure increases, and the incidence of bumps increased, especially when the barriers were near old gob areas. The pillar extraction system used at Olga, where three rows of pillars are mined concurrently, appears to reduce pillar loads at the face areas, thereby reducing the exposure of men and equipment to the

risks of bumps during mining operations. The horizontal components of in situ stresses at Olga were found to be nearly the same as those at 5 other mines in southern WV, where bumps were not common, so it appears that horizontal in situ stress alone is not a major influence on bumping at the Olga Mine.

INTRODUCTION

In 1985 the Bureau of Mines, Pittsburgh Research Center, began a field project at the Olga mine in McDowell County, WV, to study the occurrence of bumps. The purpose of the project was to identify the primary factors influencing the occurrence of bumps at the mines studied, and to develop mine design criteria to reduce their severity and frequency or to eliminate them altogether. The first site chosen was the 9-Right section, where seven 360 by 160 ft pillars were to be split and the entire section pillared. The purpose of this report is to describe the results of the initial study conducted to identify, from the investigation at the 9-Right site, those factors which were of significance in affecting the occurrence of bumps at the Olga Mine, and to give information which will aid in assessing the potential for bump related problems in other mines.

The factors investigated at 9-Right were broken down into two categories, in situ and mining related. The in situ factors studied were the overburden thickness, the physical properties and characteristics of the rock units (and coal), the presence of jointing or other discontinuities and the original in situ stresses existing in the rock prior to mining. The mining factors studied were the mine geometry and mining sequences. It was felt that the mining factors were best analyzed by evaluating changes in stress in the coal pillars and in adjacent strata, the rate of deformation of mine openings and the pattern of failure of the coal and adjacent strata. Each of these factors was studied in detail during

this investigation.

Past mining experience suggests and it is generally accepted that the coal mines most likely to experience bumps are those under overburden greater than 500 ft where the immediate roof and floor strata are strong, massive and thick (1).⁴

⁴Underlined numbers in parentheses refer to the references at the end of this report.

Specific mining conditions and practices such as mining pillar lines with gob on two sides have also been implicated as major contributing factors. We have attempted in this study to more precisely define these conditions from investigation of the conditions at the 9-Right section of Olga Mine and where possible by comparison with other mines.

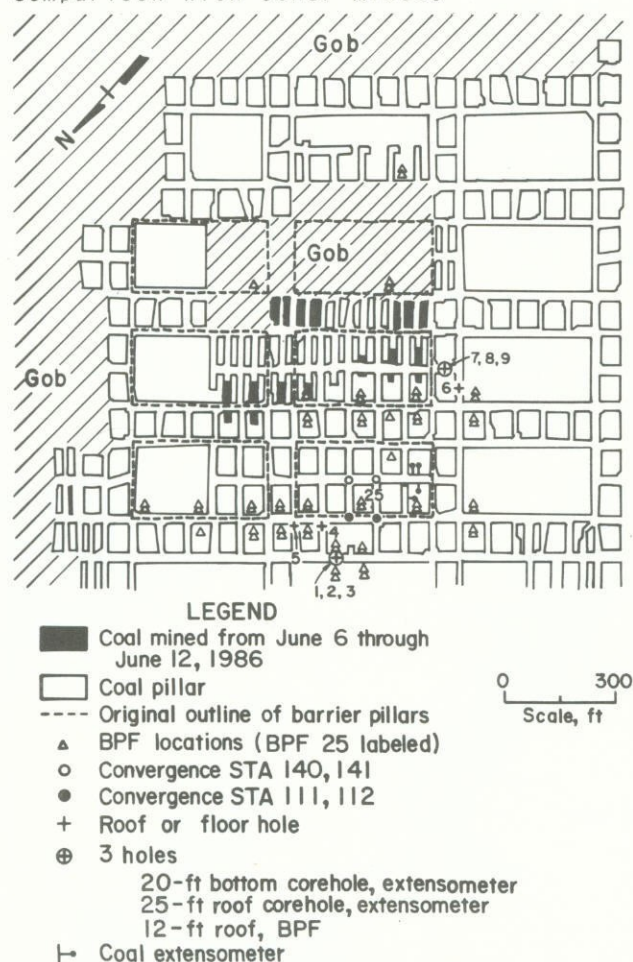


Figure 1. Section map of 9-Right showing locations of convergence stations 111, 112, 140, and 141, the BPFs and the coreholes.

PROJECT PLAN

Six major areas were studied in the 9-Right investigation. First, the general geology of the mine was determined from surface corehole data and from in-mine observations. Second, a series of nine boreholes were drilled into the roof and floor at five sites (fig. 1) to give a more detailed picture of the roof and floor lithology and to obtain rock samples for physical properties testing. Third, a series of mechanical extensometers were installed to measure movement in the coal pillars, the roof and the floor. Ten-point mechanical extensometers were installed to a depth of 25 ft in two of the roof holes and single point heave indicators were installed in two floor holes. Four 25 ft long, 10 point mechanical extensometers were installed in two adjacent coal pillars, two in each pillar. Fourth, 56 Borehole Platen Flatjacks (BPF) were installed throughout the section to monitor pressure changes in the coalbed. In most of the thirty pillars instrumented two BPFs were installed, one at 15 ft and one at 30 ft. Fifth, convergence stations were set up at each intersection. Finally, detailed records were kept of the progress and locations of mining in the section.

GEOLOGY

The Olga Mine is located in the Pocahontas No. 4 Coalbed, which averages 66 in. in thickness over the study area. In 9-Right the overburden varies from 990 to 1,600 ft (fig. 2). Over the entire mine

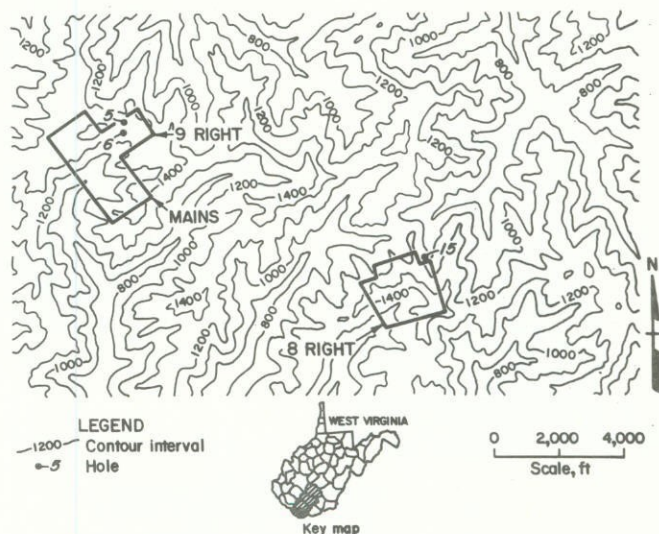


Figure 2. Overburden map.

the overburden varies from a low of 600 ft to a maximum of 1,600 ft. A generalized section of the coalbed and the adjacent strata at Olga is shown in figure 3.

The immediate roof rock at the mine is the Eckman sandstone, a channel sandstone with a thickness (based upon surface core-hole data) ranging up to 73 ft. Coreholes drilled from the surface near the 9-Right section indicated that the sandstone averaged 60 ft in thickness. Holes drilled into the roof at 9-Right found sandstone to their maximum heights of 25 ft above the coalbed. A map of the percentage of sandstone in the first 25 ft above the

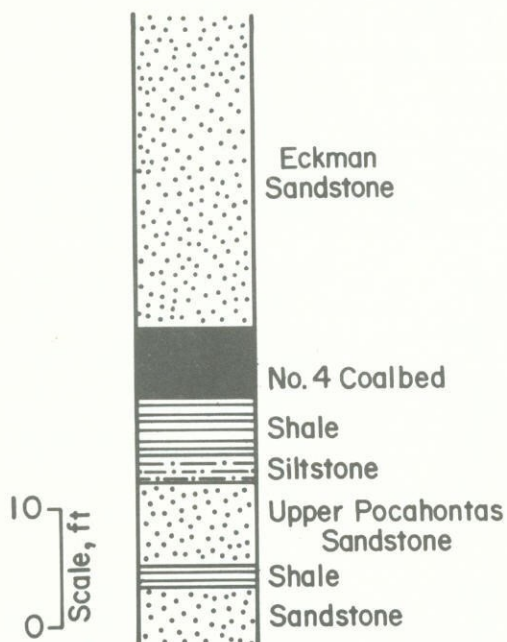


Figure 3. Stratigraphy of the 9-Right Section.

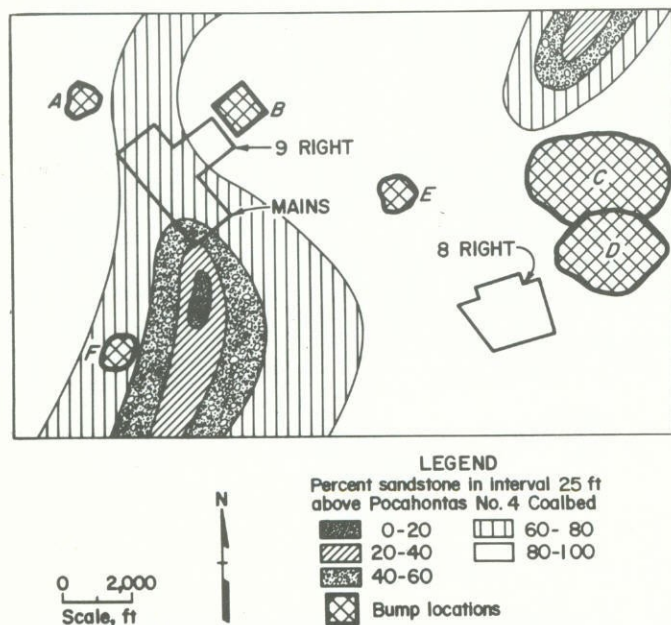


Figure 4. Percent sandstone in the first 25 ft above the coalbed.

roof is shown in figure 4. In general the roof rock was observed to be massive and uniform, although some crossbedding and signs of disturbed zones were present in some portions of the cores. The roof rock had a Rock Quality Designation (RQD) of 95 to 100. It was found to be predominantly sandstone, except in a few areas where a thin shale was present immediately above the coal. The shale, where present, varied in thickness from a fraction of an inch to slightly more than 60 in. Shale thickness of more than a few inches existed in only small portions of the section. No prominent joints or fractures were encountered in the boreholes. Some thin coal streaks (less than 1/4 in and typically about 1/16 in. in thickness) were seen in some of the holes. In the first 4 ft above the coalbed rock was sometimes encountered with irregular, discontinuous thin coal streaks scattered throughout the sandstone. In these areas the rock was susceptible to fracturing during coring operations where the coal streaks were large enough to cover the entire area of a core. In hole 6 (fig. 1) two thin clay bands were also encountered at 66 and 73 in above the roof and one core broke while coring on the lower one. Vertical joints were measured in the roof sandstone at an orientation of N75°W, generally spaced 10 ft or more apart. In the roof shale the joints were more randomly oriented and more closely spaced.

The floor rock is laterally discontinuous and varies greatly over the study area. Also, at increasing depths below the coalbed, the floor rock changes from a shale to a siltstone to a sandstone (Upper Pocahontas). Figure 5 shows, for the

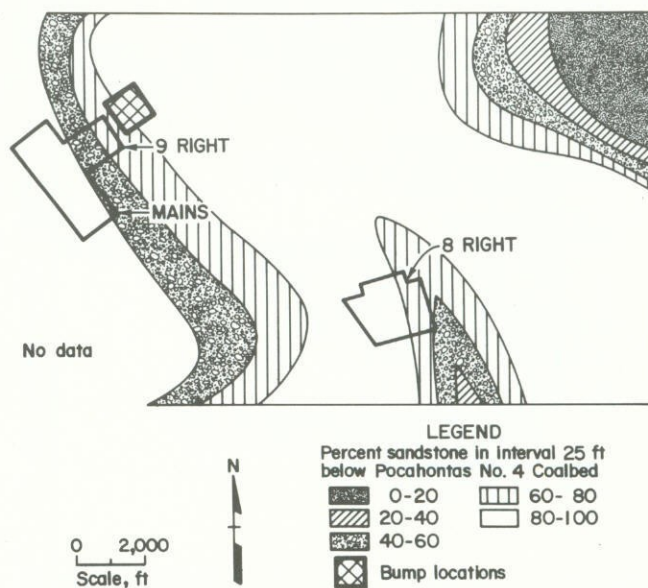


Figure 5. Percent sandstone in the first 25 ft below the coalbed.

entire mine, the percentage of sandstone in the first 25 ft below the coalbed. Generally there are 4 to 5 ft of shale in the immediate floor, underlain by from 1 to 9 ft of siltstone and followed by 2 to 7 ft of sandstone. Below that in the deepest hole, which was drilled to 16.7 ft, an interval of thinly interbedded shales and sandstones was encountered, ending with 2 ft of sandstone at the bottom of the hole. The ROD for the floor rock ranged from 80 to 100. Numerous mining related fractures or breaks on bedding planes were observed in the first 5 to 10 ft of the floor rock. The floor rock is prone to heaving during pillar extraction in areas near the gob line. The immediate shale floor breaks to depths of 1 to 2 ft and the floor can raise as much as 2 ft.

The rocks above and below the coalbed at 9-Right appear to be generally massive, thick, uniform and include a high percentage of strong sandstone, especially in the roof rock (the physical properties of the strata will be discussed in detail in the next section). This combination of conditions is relatively rare in other coal basins in the U.S., seldom occurring in the Warrior Basin of Alabama, the Illinois basin, western Pennsylvania and northern West Virginia. Recent studies (2) on the incidence of bumps in the U.S., as determined by reports of fatalities attributable to bump accidents, show that for the period 1978-83, 78 pct of the bump accidents in the eastern U.S. occurred in Virginia and southern West Virginia, in areas with geologic conditions generally similar to those at Olga. Where

information was available, 90 pct of the fatal bump accidents in the U.S. occurred under strong sandstone roof and in 80 pct of the cases strong floor rock was also present. The rock at the Olga Mine meets all of these criteria and this suggests that the geologic conditions at the mine are a major contributing factor to the incidence of bumps at the mine.

PHYSICAL PROPERTIES

The physical properties of the roof and floor rock were determined from samples obtained in four NX coreholes (1, 2, 7, and 8) and three 6-in diameter coreholes (4, 5, and 6) (fig. 1). Samples of the coal were also obtained from newly exposed mining faces and tested. The physical properties measured were the uniaxial compressive strength, triaxial compressive strength, Young's Modulus, Poisson's Ratio, and Brazilian tensile strength. No tensile tests were run on the coal samples. The resulting average physical properties are summarized in table 1.

The rocks at 9-Right are both much stiffer and stronger than the average for roof and floor rock in deep U.S. coal mines, while the Pocahontas No. 4 Coalbed at Olga is of average or below average compressive strength. This suggests that the rock at 9-Right could be subjected to much higher stresses, before failure, than the adjacent strata of many other coalbeds. It is also to be expected that the roof rock would be able to bridge loads over greater spans than in an average mine roof rock. This could allow greater loads to be concentrated on individual pillars. In

TABLE 1. - Physical properties of rock at the 9-Right Section of the Olga Mine

STRATA TYPE	COMPRESSIVE STRENGTH 1bf/in ²	TENSILE STRENGTH 1bf/in ²	YOUNG'S MODULUS ¹ 1bf/in ² x10 ⁺⁶	POISSON'S RATIO ¹
Sandstone roof	x ² =24200 s ³ = 1900 n ⁴ =40	x=1070 s= 280 n=21	x=5.16 s=0.04 n=3	x= 0.30 s= 0.02 n=3
Sandstone floor	x=21900 s= 2700 n=15	x=1010 s= 260 n=12	x=5.45 s=0.39 n=3	x= 0.39 s= 0.15 n=3
Shale floor	x=12600 s= 2600 n=18	x= 970 s= 390 n=6	x=3.97 s=0.83 n=3	x= 0.29 s= 0.03 n=3
Siltstone floor	x=16900 s= 3400 n=14	x=1320 s= 320 n=25	- - -	- - -
Coal (N=6) ⁴	x= 2400 s= 500	- -	x=0.61 s=0.17	x= 0.31 s= 0.03

¹Tangent (calculated at 50 pct of the ultimate strength).

²Mean.

³Standard deviation.

⁴Number of samples tested.

addition, since the roof rock is massive and infrequently jointed, it is possible for the roof rock to cantilever further into the gob area before breaking. This cantilevering also adds to the stress to which pillars near the gob line are subjected. It was possible in some recent gob areas to actually watch the roof gradually sagging toward the floor without actually touching the floor rock within cap lamp view, at distances up to 200 to 300 ft.

IN SITU STRESS

The horizontal components of the in situ stress in the roof rock were measured in two vertical boreholes at 9-Right and in one borehole in the 8-Right section about 2 miles away (fig. 2). The results, determined from overcoring stress relief using the Borehole Deformation Gauge, are summarized in table 2. The eleven measurements made at the three locations gave approximately the same results and indicated (both isotropic and anisotropic stress computations were made, but in this case the results were approximately the same) that the maximum horizontal principal stress was $-3,200 \text{ lbf/in}^2$ (the negative sign denotes compressive stress) and was oriented at $N60^\circ E$, while the minimum principal horizontal stress was -2300 lbf/in^2 .

The stresses measured at Olga were compared to stresses determined between 1978 and 1980 by Tosco Research, Inc., under a contract with the Bureau of Mines, Denver Research Center (3). The measurements made in the Tosco study were also conducted by overcoring in roof rock. The mines in the Tosco study were north of and within 70 miles of the Olga Mine. However the overburden depths were less, (ranging from 830 to 1,150 ft) compared to 1,500 ft at 9-Right and 1,250 ft at

8-Right, and the Tosco measurements were made above the Beckley coalbed. The average maximum principal horizontal stress measured at five mines in Raleigh, Fayette and Wyoming Counties (these counties are indicated by the diagonal lines on the key map of fig. 2) was $-3,260 \text{ lbf/in}^2$ at $N64^\circ E$. The average minimum principal horizontal stress was $-2,500 \text{ lbf/in}^2$. These stresses are very close to those measured at the Olga Mine, both in direction and magnitude. The close agreement (especially considering the differences in the formations overcored and in overburden depth) suggests that the horizontal stresses measured at all of these sites represent tectonically created regional stresses, with the measured values little influenced by mining activity or rock type and not greatly affected by overburden depth (The vertical component of stress, not measured by the vertical hole overcoring technique, would be affected) (4). It also suggests, since the mines in the Tosco study have not been noted as having bump problems, that the horizontal in situ stresses existing at the Olga Mine, without the presence of other influencing factors, would not be sufficient to cause bumps.

MINING AND PILLAR SIZE EFFECTS

There were essentially two pillar sizes at 9-Right (fig. 1), a set of large (160 by 360 ft) barrier pillars and 60 by 70 ft chain pillars. The original layout of the study area is shown in figure 1. Entry widths were approximately 18 ft. At the beginning of the study period mining was conducted within the barrier pillars to split them into smaller, 60 by 70 pillars for later extraction and in the latter phase of mining the smaller pillars were completely extracted. Undisturbed barrier pillars were left around most of the perimeter of the section.

TABLE 2. - Horizontal components of in situ stress at six West Virginia mines

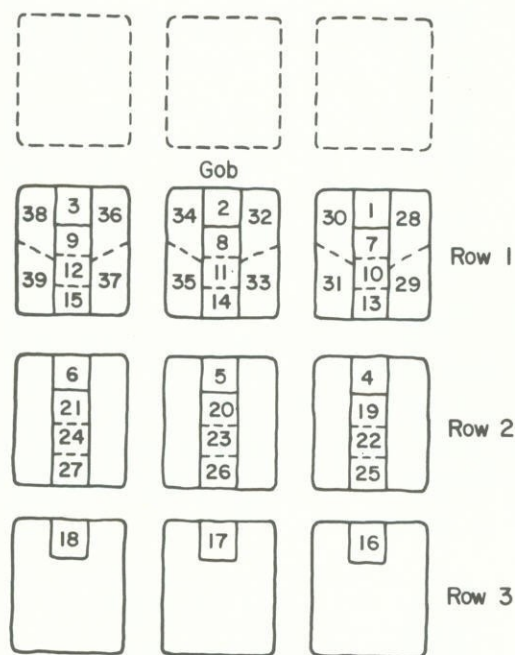
MINE	OVERBURDEN ft	STRESS		DIRECTION OF P	HOLE DEPTHS ft
		P	Q		
		lbf/in ²			
OLGA ¹	1250-1500	-3200	-2300	$N60^\circ E^3$	5-13
BONNY ²	1140	-3800	-3100	$N57^\circ E$	15-22
BECKLEY ²	830	-3300	-2500	$N59^\circ E$	10-26
BECKLEY #1 ²	1100	-3200	-1900	$N75^\circ E$	5-15
BECKLEY #2 ²	1100	-2300	-1700	$N52^\circ E$	10-12
MAPLE MEADOW ²	860	-3400	-2500	$N68^\circ E$	9-25

¹Computed by the isotropic solution of Obert (5). Minewide average.

²Minewide average. From Agapito, Et Al. (3)

³Corrected for declination. Agapito does not indicate whether a declination correction has been made.

The method used at Olga for pillar extraction (fig. 6) has been developed, through trial and error over a period of years, to reduce the occurrence of bumps. It differs greatly from a typical pillar extraction scheme where each pillar is completely removed before moving to the next one. The method consists of a series of cuts made over the width of the active section and 3 to 4 rows of pillars deep behind the gob line. The initial cut into a pillar (cuts 1 to 6 and 16 to 18) is known as the "bump cut". The next set of cuts split the pillar into two fenders or wings and the three splitting cuts required are called the "second", "third", and "fourth" cuts (cuts 7 to 15 and 19 to 27). Finally the cuts made to remove the remaining wings are called the "wing cuts". A fully developed extraction sequence using this method requires three rows of pillars being mined over the same period of time, with the row furthest from the gob being bump cut, the next row inby being split with the second, third, and fourth cuts and finally the row closest to the gob being completely extracted with the wing cuts.



LEGEND

Extraction phase with cut No.

Bump cuts 1, 2, 3, 4, 5, 6, 16, 17, 18
Center splitting 7 to 15, 19 to 27
Wings 28 to 39

0 100
Scale, ft

Figure 6. Generalized Olga Mine extraction mining sequence.

The sequence of figure 6 is an idealization and the order of the cuts is frequently changed (as may be seen in figure 1). The mine management believes that the

mining sequence developed has, in general, decreased the severity and frequency of bumps at the mine, when compared to the previously used block room and pillar methods.

A series of Borehole Platen Flatjacks (BPFs) and roof-to-floor convergence stations were installed at 9-Right to monitor the effects of mining in the section. The convergence stations were set up in the intersections and two copper BPFs were installed in each pillar instrumented at depths of 15 and 30 ft into the pillar. The initial setting pressures for the BPFs were 2,500 lbf/in²(ga). The locations of the instruments are indicated on figure 1.

The readings from a typical BPF (BPF25 installed 30 ft into a pillar) and the four convergence stations around the pillar are graphed (6) for the study period on figure 7. The major increases in BPF pressure

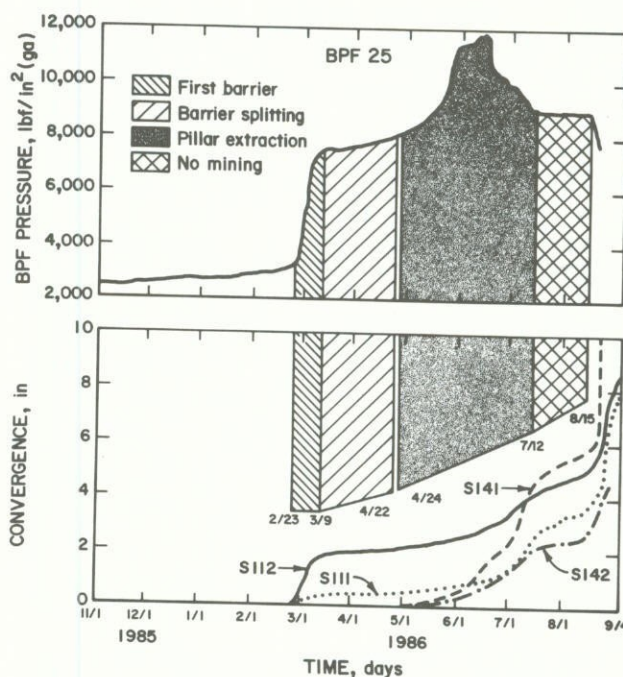


Figure 7. BPF 25 pressure graph and convergence graphs for stations 111, 112, 140, and 141; for the period November 1, 1985 to September 4, 1986.

occur during two brief periods. The first is between February 23 and March 9 and corresponds to the period when barrier splitting operations were cutting entries in the barrier block containing BPF25 to form 60 by 70 ft pillars. The actual mining period was from February 23 to March 4. The convergence graphs follow the pressure increase almost exactly, showing a brief increase in the convergence rate during barrier splitting, which then levels off.

As described by Holland and Thomas (1) the changes in pressure during the period February 23 to March 9 appear to be caused

by redistribution of load locally from the area of the newly mined entries to the new pillars. The 60 by 70 ft pillars were sufficiently strong to withstand the increased load and after the redistribution of load, both pressure increases and convergence took place at reduced rates until the beginning of May, shortly after pillar extraction operations began.

Stress changes and convergence took place over a much larger area during the pillar extraction phase, which began on April 24. The initial pillar extraction began in the pillars 9 rows away from BPF25 (in the area shown as gob at the upper center of figure 1). Pressure increases in BPF25, caused by pillar extraction, began as early as May 7. The rate of increase in pressure in BPF25 during pillar extraction was highest during the period May 21-27, when the bump cut pillar row was five rows away from it. Figure 1 shows the period June 6-12, during which the pressure measured by BPF25 reached its maximum value of 11,900 lbf/in². The maximum pressure reached in the pillar could not be determined, because the copper BPFs used at Olga failed at pressures of between 10,000 and 13,000 lbf/in². In the case of BPF25 the pressure dropped rapidly from 11,900 to 10,250 lbf/in² between June 13 and June 23. The pressure then gradually dropped during the period from June 23 to July 12 and stabilized at 9,000 lbf/in² between July 12 and August 13, when no mining took place. Significant increases in the convergence rate began well after the pressure increases, beginning when bump cut mining came within two rows of BPF25, about the same time that pressures in BPF25 began to drop. Convergence rates then continued to increase rapidly, reaching their (measured) maximum when the pillar wings were being mined. At this point the pillar had completely yielded and was carrying minimal roof load.

The BPF pressure data and the convergence measurements show that very high stresses can build up during barrier splitting, but that in general the cores of the pillars did not fail during barrier splitting. Some barriers at the edges of the gob had to be left in place because of minor bumps which threatened to become more severe if splitting continued. It appears that these pillars acted as abutment areas carrying a large portion of the load in the section from the immediate roof.

Data from extensometers installed in the coal pillars at 9-Right show that the outer portions of these pillars dilated much more than the centers, suggesting that the pillars consisted of a solid load bearing core surrounded by a yielding zone which provided some confinement pressure to the pillars. Holland and Thomas (1) have described this phenomenon and

suggested that in some cases removing the yielded outer portion of a pillar could lead to a pressure bump, even though the outside portion of the pillar may carry little vertical load.

As pillar extraction began, the stresses in the pillars increased to their maximum values, approximately 1 or 2 rows outby the rows where the first bump cuts took place. By the time of the third cut, when the pillar core was removed, the pillar had almost completely yielded and no longer supported significant load; at that point the convergence rate was very high but the risk of bumping was generally low. It appears that the pillar extraction method used at Olga allows gradual transfer of stress from active mining areas to areas outby the face where miners and equipment are less likely to be exposed to the hazards of bumping.

CONCLUSIONS

Studies at the 9-Right section of the Olga mine show that the primary factors influencing the occurrence of bumps there are:

1. Geologic - Massive and competent immediate roof and floor rock with a widely spaced jointing system.
2. Physical properties - High Young's modulus, and compressive and tensile strength of the roof and floor rock.
3. Overburden - Thick overburden ranging from 990 to 1,600 ft in the 9-Right Section.

The combination of these first two factors creates conditions which allow the roof rock to cantilever over the gob or to arch over large spans and in either case this allows high stresses to build up in the coalbed, either at the gob line or in the abutment areas. Secondary factors which may have influenced the occurrence of bumping were:

1. Pillar Splitting - The incidence of bumps was found to increase when medium sized barrier pillars were split into smaller (60 by 70 ft) chain pillars.
2. Horizontal In situ Stress - From comparisons with horizontal stresses measured in other mines in the region, horizontal in situ stress appeared to be unlikely to be a major influence upon the occurrence of bumps at the Olga mine.

The present pillar extraction system, developed over many years, has greatly improved the safety of miners and equipment and allowed higher coal recovery than block

pillar recovery methods would have allowed. The system which spreads out the pillar extraction process appears to transfer load away from the working faces and possibly reduces loads in the rows where men and equipment are present.

It is likely that additional design improvements based upon modeling using in situ stress, pressure buildup, convergence, geologic and physical property data could optimize the mining method and determine the safest retreat mining sequence.

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LONGWALL FACE BURSTS AND INADEQUATE CAVING: A CASE STUDY

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ABSTRACT

Deep coal mines with strong roof and floor strata frequently encounter face and rib bursts. The burst problem becomes more severe with increased depth. While the exact causes of bursts are often difficult to determine, localized high stress zones are common to their occurrence. This Bureau of Mines report describes a study at a mine using longwall systems that experienced both face and floor bursts. The study correlates shield loading, entry closure, and forward abutment pressure increases to observed burst occurrences. The results indicate that strong roof strata that overhang behind the longwall supports create additional abutment stress. When caving lags behind the face and the abutment zone does not advance ahead of the face during mining, stresses on the face may increase to a critical level, resulting in a burst. The critical level occurs when the stress in the abutment zone exceeds the ability of the coal and/or adjacent strata to store strain energy. Caving characteristics may be improved by inducing fractures in the roof or by providing sufficient support resistance to shear the roof. Data analysis of a major burst indicated a dramatic increase in floor heave, with a maximum occurring 80 to 100 ft in by the face, and a lag in the cave line behind the face supports. Analysis of the effects of caving characteristics and floor heave on burst occurrences is presented.

INTRODUCTION

Bursts, the instantaneous release of roof, rib, or floor material propelled into the mine opening, are a common problem in deep coal mines with strong roof and floor rock. Many studies have been conducted to understand their causes and to predict their occurrences. The exact causes are very difficult to

determine, and reliable prediction is difficult. Past research showed that several factors influence the occurrence of a burst, including high horizontal and vertical stress concentrations, mine geometry and orientation, physical properties of rock and coal, and geologic features (1). Each underground location has site-specific conditions that may contribute to a burst. Very few studies, however, have addressed roof caving behavior behind the powered supports and its influence on bursts.

Strong roof strata, a common contributor to bursts, may inadequately cave behind the shields as the longwall face advances. When the roof hangs behind the supports, stress on the face may increase to a critical level and the face becomes burst-prone.

The objective of this research is to understand the effect of inadequate caving behind longwall shield supports on burst occurrences in underground coal mines. The study included monitoring shield pressure, roof-to-floor convergence, roof-bed separation, and front abutment pressure changes as mining advanced in a deep longwall mine. The analysis included correlating the data to documented face and gate entry bursts.

TEST SITE BACKGROUND

A Bureau of Mines in-house research project was initiated to gather sufficient rock mechanics data in a deep burst-prone western coal mine at various stages of mining. This study provided a unique opportunity to observe preburst and postburst behavior and the effectiveness of various techniques to control or prevent bursts in underground coal mines.

The Mine

The mine, located in western Colorado, is operating at a depth of nearly 3,000 ft, with other active mine workings located about 400 ft above the coal seam being studied (2). The advancing longwall mining method is used to mine a 10-ft coal seam dipping at 12 degrees, with gate entries 16 ft wide. The long axis of the longwall panels is oriented parallel to the strike of the seam. The mine operates one main development section and one 800-ft-wide by approximately 3,900-ft-long longwall panel. Upslope from the longwall panel are a previously mined longwall panel and old room-and-pillar workings. The general mine layout is shown in figure 1.

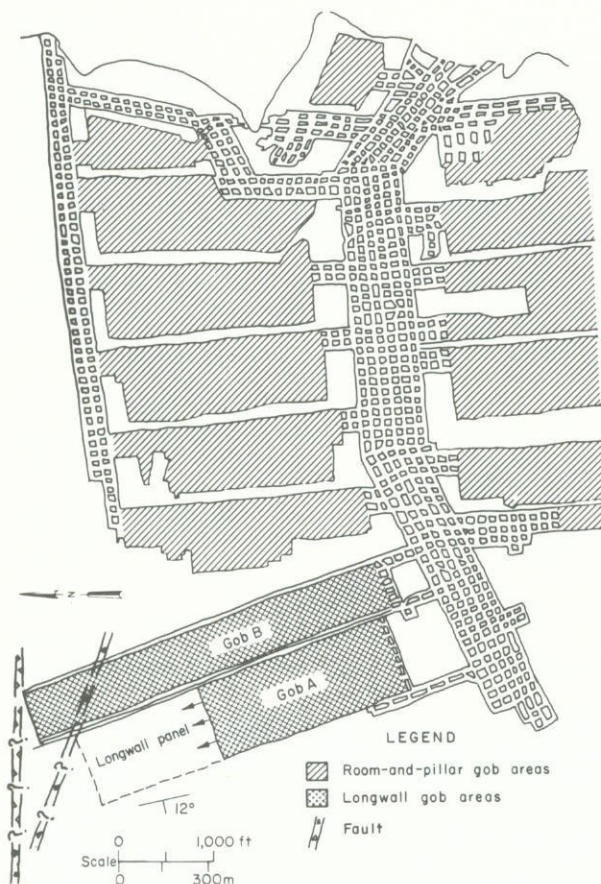


Figure 1. General mine layout.

Roof support in the entries is accomplished by mechanical anchor or resin-grouted roof bolts. Timber supports, trusses, and steel arches are also used in some areas of the mine.

Powered Support Specifications

The longwall face is supported by 162

Hemscheidt G 546-22.5/37 two-leg shields having an 82.3-in² cylinder area per leg, a 250-ton-per-leg yield load, and a 6,090-psi yield pressure (3). The maximum open height of each shield is 146 in, while the minimum open height is 70.8 in. The shields also have high-volume pressure relief valves to prevent shield leg damage during extreme bursts or floor heave. The maximum pressure exerted on the floor by the shield is 345 psi.

Mine Geology

The mine is in the Coal Basin Seam (4). The immediate roof, approximately 5 ft thick, is composed mostly of strong siltstone, shales, and sandstone layers. The immediate roof is overlain by a 9-ft-thick, competent sandstone layer that does not readily fracture and often forms cliffs where it outcrops at the surface. The mine floor consists of a strong shale-sandstone layer ranging in thickness from 4 to 11 ft in different areas of the mine, and a lower coalbed beneath the shale-sandstone floor which consistently measures 8 to 10 ft thick. Figure 2 shows the geologic column at the test site. Two well-defined faults were mapped across the longwall panels.

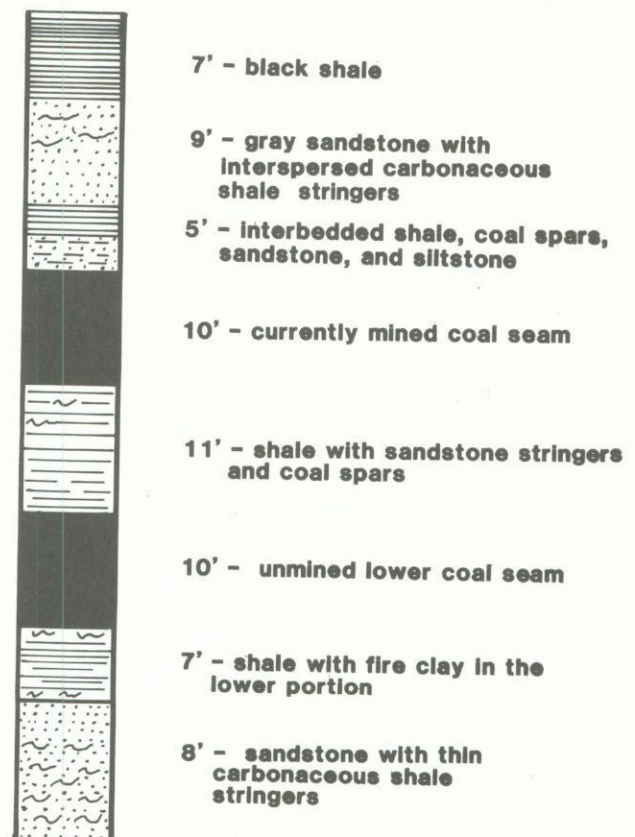


Figure 2. Geologic column at test site.

Burst Occurrences

Owing to the thick overburden and strong roof and floor, bursts have occurred in both room-and-pillar and longwall sections. Until 1983, stress relief was practiced only to destress longwall corners and mining sections at depths exceeding 2,000 ft.

Table 1. Summary of the major events at the test site.

	Event Number	Type of Event	Cause of Event	Date of Event	Location of Event (Between Shields)	Face Advance (ft)	Direction of Mining From - To
Before Destressing	1	Coal Burst/Face	Mining	1/13/83	25 - 36	271	HE* - TG**
	2	Coal Burst/Face	Mining	2/22	12 - 37	411	HE - TG
	3	Coal Burst/Face	Mining	2/25	35 - 46	430	TG - HE
	4	Coal Burst/Face	Mining	3/23	7 - 15	535	HE - TG
	5	Coal Burst/Face	Mining	4/15	25 - 39	605	HE - TG
	6	Rock Burst/Floor	Mining	4/20	125-162 + Tailgate	618	TG - HE
After Destressing	7	Rock Burst/Floor	Destressing (VF)	9/17	150-162 + Tailgate	1000	N/A
	8	Rock and Coal Burst/Floor & Rib	Destressing (VF & WI)	1/28/84	152-185 + Tailgate	1357	N/A
	9	Rock Burst/Floor	Destressing (VF & WI)	6/18	125-162 + Tailgate	1877	N/A

* HE - Headgate
** TG - Tailgate

VF - Volley firing
WI - Water infusion

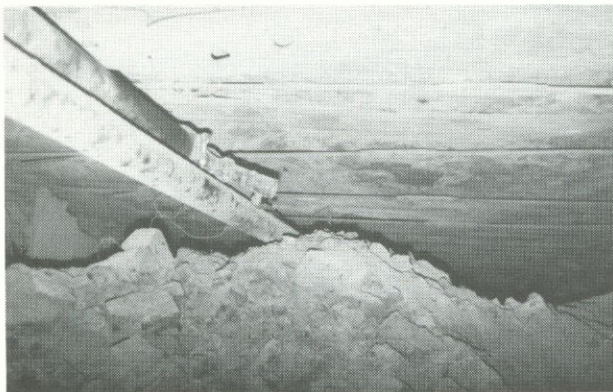
Soon after operation began on the new longwall panel, face bursts occurred. Between January 1983 and July 1984, nine major bursts were recorded. A summary of the bursts is presented in table 1; the bursts were either triggered by mining or induced during stress relief. On April 20, 1983, after 618 ft of longwall advance, a major burst occurred that affected both the tailgate entries and the face. The burst resulted in a rapid floor heave that extended approximately 1,200 ft along the tailgate entry and 300 ft along the face. In addition, it disrupted ventilation, stopped production, and damaged 40 longwall shield supports. Figure 3 shows the floor in the headgate

and tailgate entries after the burst. The floor in the tailgate entry heaved to within 1 ft of the roof in some areas; no significant roof separation was measured.

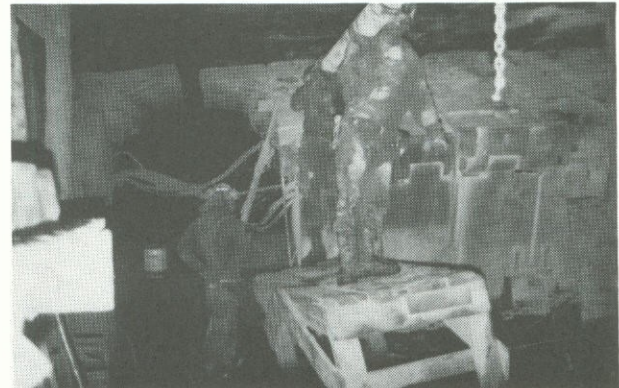
Burst Control

After the April 20 burst, mine management incorporated an extensive burst control plan to safely continue production. The program consisted of detecting high-stress areas by the drilling-yield method and destressing such areas using volley firing. After the program was incorporated into the mining sequence, the longwall panel was mined to completion, advancing 3,900 ft, with essentially no uncontrolled burst occurrences; nearly all face bursts were eliminated. Bursts that did occur were mostly in the tailgate entry and involved floor heave. The drilling-yield method involves drilling holes into the coal seam and recording the following information: volume of cuttings produced per length of hole drilled, occurrence of bounces, locations of gas in the hole, and squeezing of the hole on the drill rod. The information collected is used to locate high stress zones in the face. The area is classed as burst-prone if the volume of cuttings from a 2-in-diam hole exceeds 19 l/m of hole drilled, or if minor bursts or constant hole squeezing are recorded during the drilling (2).

Once the high stress areas are detected, they are destressed through volley firing, a method that uses explosives to fracture the coal face to a certain depth ahead of the face. This method, used prior to face advance or entry development, causes the abutment zone to advance away from the active working face. The blast holes are loaded with 3 lb of permissible explosives, stemmed, and detonated. The drill pattern



a. Tailgate entry after burst.



b. Headgate entry after burst.

Figure 3. Headgate and tailgate entries after the April 20 burst.

consists of a series of 2-in-diam holes, 13 to 15 ft deep, drilled on approximately 4-ft centers. Hole depth depends on the required daily advance of the face and on the location and magnitude of the stress abutment ahead of the face. Local conditions and site-specific experience dictate exact hole parameters. The corners of the advancing longwall face require a radial drilling pattern, using combinations of two or three holes angled at 10 degrees and 45 degrees, as shown in figure 4, to relieve high stress in both the face and rib area.

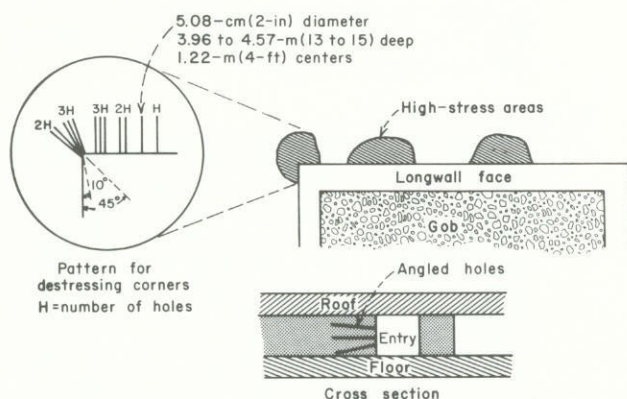


Figure 4. Volley firing drill hole pattern for longwall faces and corners.

ROCK MECHANICS INVESTIGATION

Since high-stress zones are a common denominator in the burst problem, a better understanding of the causes of high-stress concentration and stress redistribution during mining should lead to the development of control techniques to alleviate the strata pressures or to trigger bursts in a controlled fashion. Based on this rationale, an instrumentation program was developed to monitor stress changes, roof and floor deformations, and shield loading in a burst-prone mine. The data will be used to evaluate the following factors:

- Causes and location of high-stress concentrations,
- Relationship of burst occurrences to stress distributions,
- Development of systematic methods to prevent or control coal mine bursts.

Data Collection

Different types of rock mechanics data were collected.

1. Stress

- Magnitude and direction of the secondary horizontal principal stresses

- Stress redistribution caused by mining
 - Shield pressure loading
- Strata Movement
 - Roof separation
 - Roof-floor convergence
 - Panel-rib expansion
 - Geotechnical Data
 - Coal-rock properties
 - Geological mapping

Based on the need to gather data before and after a burst, all instruments were installed prior to advancing the longwall face. Shield pressure data collection did not begin until the face had advanced about 150 ft.

Instrumentation

1. Stress

- The magnitude and direction of the maximum and minimum secondary principal horizontal stresses were obtained using the Bureau of Mines overcoring technique with the three-dimensional borehole deformation gage. The overcoring was performed in the roof of the longwall headgate entry approximately 250 ft inby the longwall starting room, as shown in figure 5. A vertical borehole was drilled about 9 ft into the

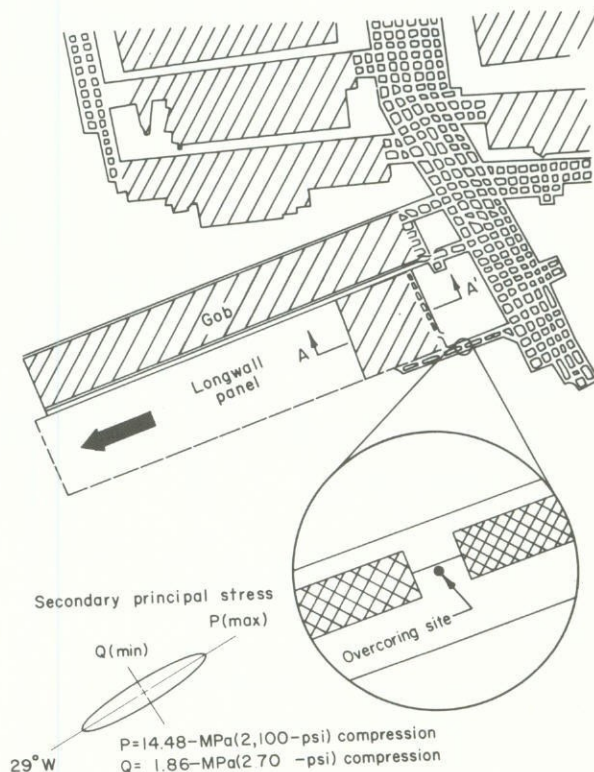


Figure 5. Overcoring site and test results.

roof before proceeding with overcoring tests, so the tests would not be influenced by stress concentration surrounding the mine openings. A total of four reliefs at 16-in intervals were collected; however, cores from only two reliefs were used since the other two cores fractured during recovery. The maximum principal horizontal stress, approximately 2,100 psi in compression, was measured parallel to the headgate entry axis.

- b. Borehole pressure cells (BPC's) were installed in the panel and packwalls at two stations in the tailgate entries and were oriented to measure vertical stress changes during mining. Stations 1 and 2 were located 950 ft and 2,000 ft inby the longwall starting room, respectively. Three BPC's were installed in each packwall. The cells were located approximately 2 ft from the packwall edges and in the packwall core. The BPC's in the panel were installed on approximately 5-ft intervals up to 20 ft from the rib and on 10-ft intervals between 20 and 50 ft in the panel. Each BPC was initially pumped to a setting pressure of 3,000 psi, approximately the premining overburden pressure based on 3,000 ft of overburden. No cells were installed at depths greater than 60 ft because

drilling into the panel became difficult at this depth. The instrumentation plans for both stations are shown in figure 6.

- c. The hydraulic pressure on each leg of seven different shields was continuously recorded using mechanical chart recorders with a range of 0 to 10,000 psi. The shields monitored were shields 5, 24, 44, 57, 84, 144, and 155. Each shield is approximately 5 ft wide, and the shields are numbered from the headgate to the tailgate starting with number 1 in the headgate entry. The face mining height was recorded to the nearest 0.1 ft when the shield pressure charts were changed.

2. Strata Movement

- a. Multipoint borehole extensometers (MPBX) were installed in the upper and lower tailgate entries at stations 1 and 2 to detect roof movements at different depths in the roof. The anchor depths were selected to coincide with the various geologic roof layers. Owing to the existence of a strong sandstone layer in the roof, roof layer separations were minimal; anchor depths are illustrated in figure 6. Four-point extensometers with anchors located at 5, 10, 15, and 20 ft from the rib were also installed in the panel at station 1 to measure the

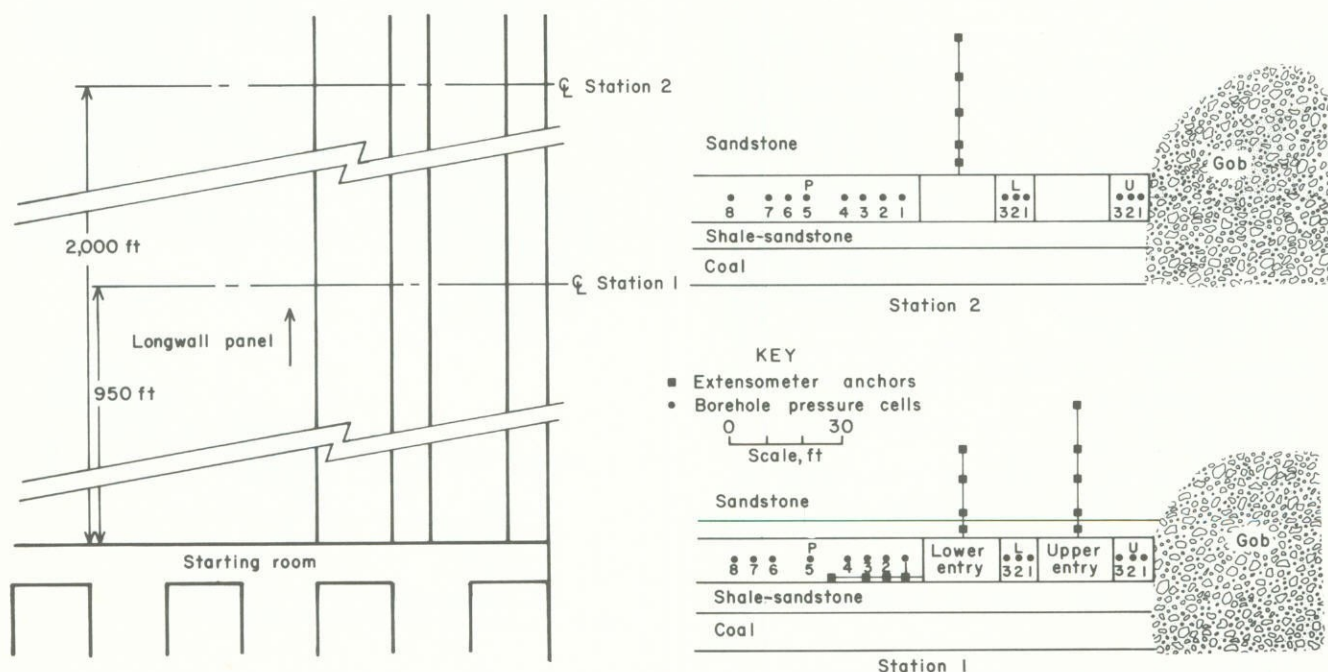


Figure 6. Instrumentations plan for sites 1 and 2.

lateral deformation of coal as mining advanced. Readings from the MPBX were recorded to the nearest 0.001 inch.

- b. Thirty-five roof-to-floor convergence stations were installed on 50-ft intervals along the lower tailgate entry. The first station was installed 450 ft ahead of the starting room, and the last station was at 2,000 ft ahead of the starting room. Each convergence station consists of a 3/4-in-diam by 22-in-long steel reinforcing rod grouted in a 1-5/8-in-diam borehole drilled in the floor directly beneath a roof bolt. A rod-type extensometer was used to measure the distance between the bolthead and the rod in the floor. The extensometer reads directly to the closest 0.01 in. The floor rods were installed at a depth of about 20 in to provide a solid anchorage in the floor.

3. Geotechnical Data

- a. Physical property testing was performed on core samples obtained from two NX-size holes drilled into the roof and floor of the third crosscut in the headgate entry. Core samples up to 32 ft above and 36 ft below the mined seam were recovered and tested in the laboratory. Table 2 shows the uniaxial and triaxial test results from the roof, floor, and coal samples. Table 3 shows the Brazilian test results.

Thirty in situ borehole shear strength tests were performed at 2- to 3-ft intervals in the roof and 1- to 2-ft intervals in the floor. The borehole shear tester (BST) provides in situ values for both angle of internal friction (ϕ) and cohesion (S_o) (5). Using the BST results, the in situ uniaxial compressive strength (C_o) and the in situ rock tensile strength (T_o) were calculated applying the following equations (6); results are presented in Table 4.

$$C_o = \frac{2 S_o \cos \phi}{1 - \sin \phi} \quad (1)$$

and

$$T_o = \frac{2 S_o}{\tan \phi + [(\tan \phi)^2 + 1]^{1/2}} \quad (2)$$

Table 2. Uniaxial and triaxial test results.

COAL										
Description	Sample #	Diameter (in)	Length (in)	Specific Gravity	Latent Pressure (psi)	Breaking Load (lb)	Compressive Strength (psi)	Young's Mod x 10 ⁶ (psi)	Poisson's Ratio	
Coal Bed Below Floor Rock Samples L/D=2	1	2.120	3.970	1.490	0	8300	2351	0.551	0.453	0.291
	2	2.120	3.950	1.540	0	5700	1615	0.442	0.342	0.230
	3	2.120	3.955	1.694	0	11900	3343	0.556	0.441	0.490
	4	2.120	3.965	1.500	250	11600	3286	0.462	0.362	
	5	2.120	3.965	1.311	250	8000	2266	0.417	0.382	
Mined Coal Bed Samples L/D=2	1	2.120	3.970	1.118	250	12400	3513	0.470	0.461	
	2	2.120	3.912	1.078	500	6250	1771	0.289	0.289	
	3	2.120	3.889	1.127	500	12500	3541	0.429	0.362	
	4	2.120	3.908	1.184	500	14700	4164	0.486	0.434	
	5	2.120	3.904	1.069	750	16300	4618	0.481	0.362	
	6	2.120	3.873	1.073	750	18300	5184	0.502	0.347	
	7	2.120	3.900	1.065	750	16700	4731	0.488	0.356	
	8	2.120	3.905	1.076	1000	19250	5453	0.495	0.340	
	9	2.120	3.918	1.057	1000	15700	4448	0.427	0.330	
	10	2.120	3.912	1.070	1000	21700	6147	0.689	0.401	
Coal Bed Below Floor Rock Samples L/D=1	1	2.120	1.958	1.472	0	7500	2125	0.412	0.344	0.214
	2	2.120	1.952	1.314	0	6800	1926	0.331	0.201	0.144
	3	2.120	1.960	1.587	0	14400	4079	0.592	0.340	0.152
Mined Coal Bed Samples L/D=1	1	1.950	1.975	1.071	0	4900	1641	0.312	0.256	0.192
	2	1.950	1.975	1.115	0	9800	3281	0.395	0.175	0.293
	3	1.950	1.975	1.069	0	3800	1272	0.298	0.184	0.206
	4	1.950	1.975	1.062	0	3600	1205	0.239	0.155	0.201
	5	1.950	1.975	1.075	0	3450	1155	0.137	0.054	0.097
ROOF/FLOOR										
Description/Depth	Diameter (in)	Length (in)	Specific Gravity	Latent Pressure (psi)	Breaking Load (lb)	Compressive Strength (psi)	Young's Mod x 10 ⁶ (psi)	Poisson's Ratio		
Roof Rock Samples L/D = 2	9'3" - 9'7"	2.145	4.285	2.564	500	89500	24767	4.298	3.986	
	9'7" - 9'11"	2.145	4.285	2.574	500	91800	25404	4.638	3.869	
	11'4" - 11'8"	2.145	4.305	2.531	1000	86000	23799	3.486	3.229	
	11'8" - 12'0"	2.145	3.908	2.543	1000	78000	21862	3.437	3.279	
	19'4" - 19'8"	2.145	4.275	2.613	1000	57000	15774	2.455	2.213	
	19'8" - 20'0"	2.145	4.280	2.629	1500	72500	20063	2.657	2.431	
	20'10" - 21'2"	2.145	4.244	2.608	1500	71900	19897	2.390	2.390	
	24'0" - 24'4"	2.145	4.283	2.648	0	74500	20616	4.009	3.384	.132 .05
	24'4" - 24'8"	2.145	4.280	4.689	0	81800	22636	5.704	5.140	.142 .141
	24'8" - 25'1"	2.145	4.280	2.706	0	107000	29610	6.102	5.751	.206 .175
Floor Rock Samples L/D = 2	6'3" - 6'7"	2.145	4.285	2.615	0	21200	5867	1.611	.957	.115 .051
	8'0" - 8'4"	2.145	4.260	2.671	500	119000	32931	5.505	5.505	
	9'8" - 10'0"	2.145	4.301	2.575	0	37700	10443	3.257	2.785	.313 .221
	21'0" - 21'5"	2.145	4.301	2.209	0	14700	4068	0.585	0.376	.151 .087
	22'3" - 22'7"	2.145	3.948	2.424	500	29800	8247	1.300	1.300	
	30'6" - 30'10"	2.145	4.300	2.493	1000	70500	19509	3.364	3.251	
	31'6" - 31'10"	2.145	4.280	2.513	1000	72200	19980	3.535	3.055	
	34'8" - 35'0"	2.145	4.295	2.460	1500	82000	22692	4.079	3.121	
Roof Rock Samples L/D = 1	2'0" - 2'2"	2.120	2.161	2.543		3040		422		
	5'9" - 5'11"	2.120	2.160	2.681		10730		1492		
	6'3" - 6'5"	2.120	2.167	2.650		6660		923		
	7'8" - 7'10"	2.120	2.168	2.665		5520		764		
	8'0" - 8'2"	2.120	2.162	2.654		12070		1800		
	8'2" - 8'4"	2.120	2.157	2.693		7150		995		
	18'10" - 19'0"	2.120	2.173	2.490		8190		1131		
	20'4" - 20'6"	2.120	2.165	2.540		5525		766		
	31'4" - 31'6"	2.120	2.182	2.514		8760		1206		
	31'7" - 31'9"	2.120	2.178	2.660		12140		1674		
Floor Rock Samples L/D = 1	2'0" - 2'2"	2.150	2.165	2.605		3730		510		
	5'3" - 5'6"	2.150	2.158	2.612		7200		987		
	7'0" - 7'3"	2.150	2.133	2.566		8740		1213		
	7'3" - 7'6"	2.150	2.174	2.627		10970		1494		
	8'4" - 8'7"	2.150	2.158	2.607		9090		1247		
	10'1" - 10'3"	2.150	2.156	2.612		4010		550		
	10'4" - 10'7"	2.150	2.167	2.616		9130		1248		

Table 3. Brazilian test result.

Description/Depth	Diameter (in)	Length (in)	Specific Gravity	Breaking Load (lb)	Indirect Tensile Strength (psi)
Roof Rock Samples L/D = 1					
2'0" - 2'2"	2.120	2.161	2.543	3040	422
5'9" - 5'11"	2.120	2.160	2.681	10730	1492
6'3" - 6'5"	2.120	2.167	2.650	6660	923
7'8" - 7'10"	2.120	2.168	2.665	5520	764
8'0" - 8'2"	2.120	2.162	2.654	12070	1800
8'2" - 8'4"	2.120	2.157	2.693	7150	995
18'10" - 19'0"	2.120	2.173	2.490	8190	1131
20'4" - 20'6"	2.120	2.165	2.540	5525	766
31'4" - 31'6"	2.120	2.182	2.514	8760	1206
31'7" - 31'9"	2.120	2.178	2.660	12140	1674
Floor Rock Samples L/D = 1					
2'0" - 2'2"	2.150	2.165	2.605	3730	510
5'3" - 5'6"	2.150	2.158	2.612	7200	987
7'0" - 7'3"	2.150	2.133	2.566	8740	1213
7'3" - 7'6"	2.150	2.174	2.627	10970	1494
8'4" - 8'7"	2.150	2.158	2.607	9090	1247
10'1" - 10'3"	2.150	2.156	2.612	4010	550
10'4" - 10'7"	2.150	2.167	2.616	9130	1248

where C_o = in situ compressive strength, psi,
 T_o = in situ tensile strength, psi,
 ϕ = angle of internal friction from BST, deg,
and S_o = cohesion from BST, psi.

Table 4. In situ physical properties from borehole shear test results.

Test Depth (ft)	Friction Angle (deg)	Cohesion (psi)	Uniaxial Compressive Strength (psi)	Tensile Strength (psi)
Roof				
3	37.3	176	711	175
6	44.5	261	1,245	219
9	38.9	61	255	58
12	29.2	609	1,888	715
18	33.4	51	189	55
21	28.6	523	1,762	621
24	36.1	447	1,758	455
27	23.0	556	1,680	736
30	39.1	9	38	9
Floor				
1.5	37.0	187	750	186
3	26.0	773	2,474	966
4	32.9	294	1,081	320
5	29.4	409	1,400	478
6	11.1	1,070	2,601	1,761
7	28.5	611	2,054	727
8	20.5	1,279	3,687	1,775
9	35.4	368	1,426	380
11	16.5	444	1,189	663
13	14.1	517	1,326	806
15	15.5	560	1,372	852
17	23.0	308	931	408
19	25.5	200	634	252
21	20.1	478	1,368	668
23	24.2	488	1,509	631
25	28.0	376	1,252	452
27	23.3	1,088	3,306	1,432
29	35.4	489	1,895	505
31	35.8	423	1,653	433
33	35.6	308	1,199	317
35	34.5	356	1,353	375

- b. Two major faults were mapped crossing the studied longwall panel at approximately 3,900 ft from the starting room.

DATA ANALYSIS

The primary emphasis of data collection was mainly directed toward characterizing roof caving, forward abutment pressure, and room closure influences on bursts.

Roof Caving

Different methods to predict roof caving are available (7). One of the most common methods, used for this study, is recording the shield leg pressure and calculating the time-weighted average resistance (TWAR) during the periodic roof weighting. The time-weighted average pressure of each shield leg is the area under the pressure-versus-time curve recorded for each cycle divided by the total cycle time. Figure 7 shows a typical pressure-time curve for a shield leg during a mining cycle. The variation in hydraulic pressure results from the forces applied to the shields through the roof and floor. Initially, the shield is set against the roof, point s. As the neighboring shields are lowered, advanced, and set, the pressure on the shield

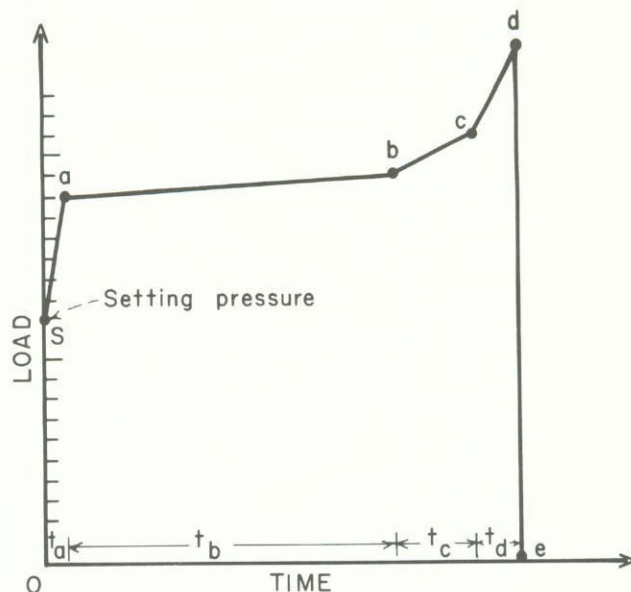


Figure 7. A typical pressure time curve for a shield leg during a mining cycle (7).

increases from point s to a. The increase in the pressure from point a to b is caused by the excess shield loading due to mining. When the shearer is cutting in the immediate vicinity of the shield, an increase in the pressure from point b to c is noticed. As the adjacent supports are lowered, advanced, and set, there is a significant pressure increase in the cycle shown from point c to d. Finally, as the shield is lowered so it can be advanced, the pressure drops from point d to e. Every shield cycle varies owing to the interaction between the roof, floor, and shield, the location of the shield with respect to face location, and changes in hydraulic pump pressure.

The time-weighted average pressure (TWAP) was calculated for each shield leg using the following equation:

$$TWAP = \frac{1/2(P_a + P_s) t_a + 1/2(P_b + P_a) t_b + 1/2(P_c + P_b) t_c + 1/2(P_d + P_c) t_d}{t_a + t_b + t_c + t_d} \quad (3)$$

Referring to figure 7, P_a , P_b , P_c , and P_d are the pressures at points a, b, c, and d, respectively; P_s is the shield setting pressure. t_a , t_b , t_c , and t_d are the times at points a, b, c, and d, respectively.

TWAP gives a good indication of the

pressure in each shield leg. To calculate the TWAR on the shields in tons, the following equation was used:

$$TWAR = (\Sigma \text{ TWAP on both legs}) \cdot \frac{A_x}{2000} \cdot \frac{e}{f} \quad (4)$$

where TWAR = time-weighted average resistance, tons,
TWAP = time-weighted average pressure, psi,

A_x = cross-sectional area of shield leg, in²,
= $\pi D^2/4$, where D is the diameter of shield leg, in²,

and e/f = leg efficiency specific to the support (dependent on mining height).

The calculated time-weighted average resistance, in tons for each shield, was plotted versus face distance in figure 8. The average periodic weighting interval

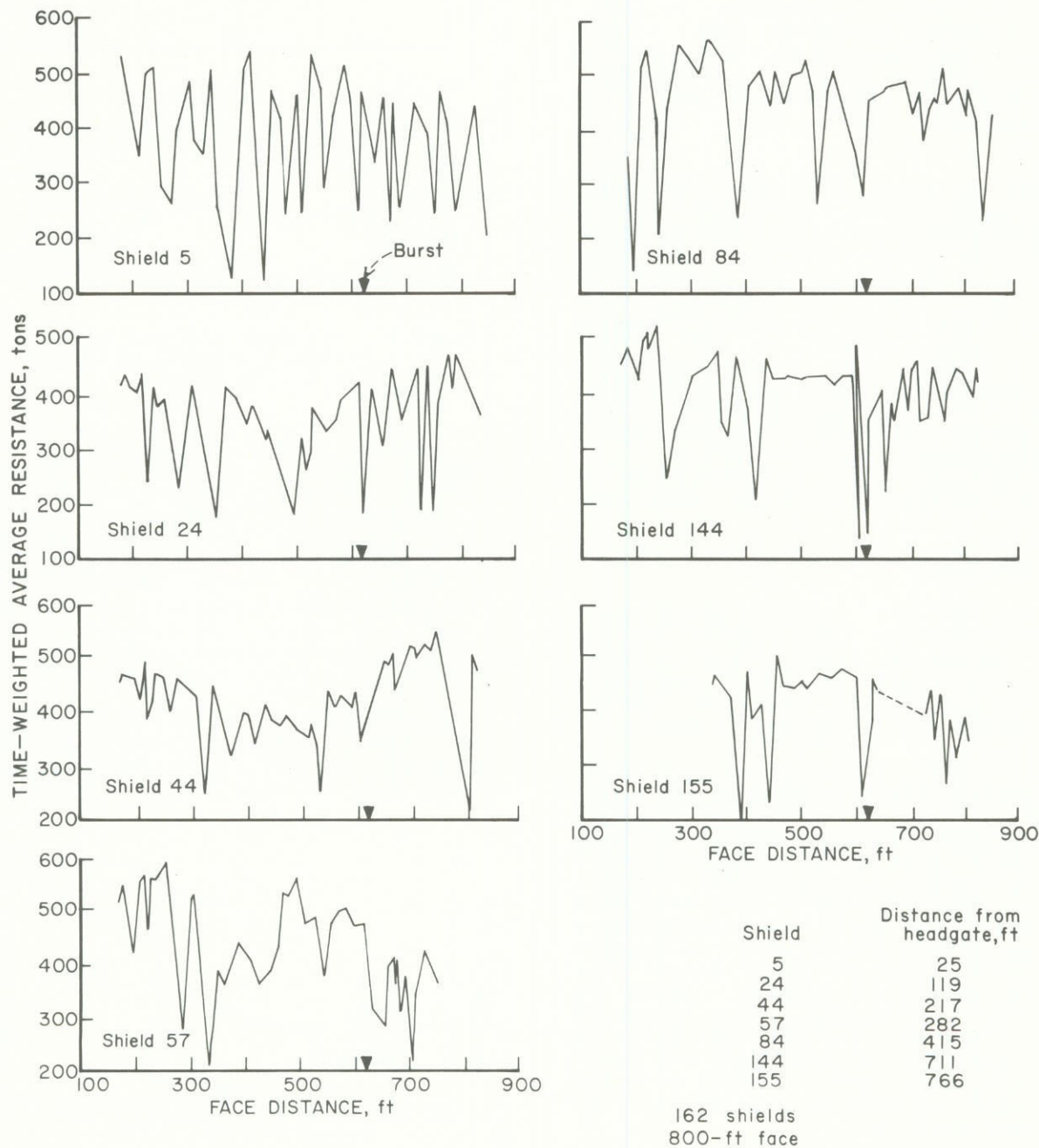


Figure 8. The weight average resistance versus face distance for each shield.

for all instrumented shields, which is the distance between the maximum TWAR, is approximately 40 ft. Data from shield no. 5, located approximately 25 ft from the headgate entry, indicate regular breaking and caving of the roof behind the face on the average of 40 ft; the peaks in the TWAR are spaced approximately 40 ft apart. Shields at the center of the panel also indicate fairly regular caving, but with an average periodic weighting interval of approximately 50 ft. Data from shields no. 144 and 155, which are very close to the tailgate entry, indicated that the roof was not caving regularly. In fact, the TWAR results indicated that a high load cycle, hypothesized as being due to non-caving of the tailgate roof extended for a distance of approximately 145 ft of face advance prior to a major burst in April 1983.

Analysis of these data shows that when the roof readily caves, the face is not overstressed and no major face bursts occur. On the other hand, as the roof hangs as a cantilever beam behind the shields, the weight of the beam is transferred onto the coal, and face burst conditions may exist. The exact cantilever length that causes the high-stress conditions is not easy to determine. The high loading cycle, which extended 145 ft, contributed to the high-stress conditions, and even shorter cantilever lengths may have contributed to high-stress conditions near the middle of the face.

Forward Abutment

Movement of the forward abutment as mining took place was monitored by borehole pressure cells, roof-to-floor convergence stations, and multiple-point borehole extensometers installed in the tailgate entry roof ahead of the face.

Figure 9 shows the effect of mining on the vertical pressure changes in the upper and lower packwalls. Borehole pressure cells installed in the upper packwall adjacent to a previously mined longwall, U1 and U2, showed no significant changes as the face advanced, while data from cell U3 adjacent to the upper entry indicated a gradual pressure decrease with an average gradient of 2.5 psi/ft face advance. In the lower packwall, however, cell L2 installed in the packwall core showed no significant pressure changes, and cells L1 and L3, adjacent to the upper and lower gate entries, indicated a gradual pressure decrease with an average of 3.25 and 1.25 psi/ft, respectively.

This behavior may be caused by the loss of confinement under the packwalls due to floor heave in the entries. Cells U1 and

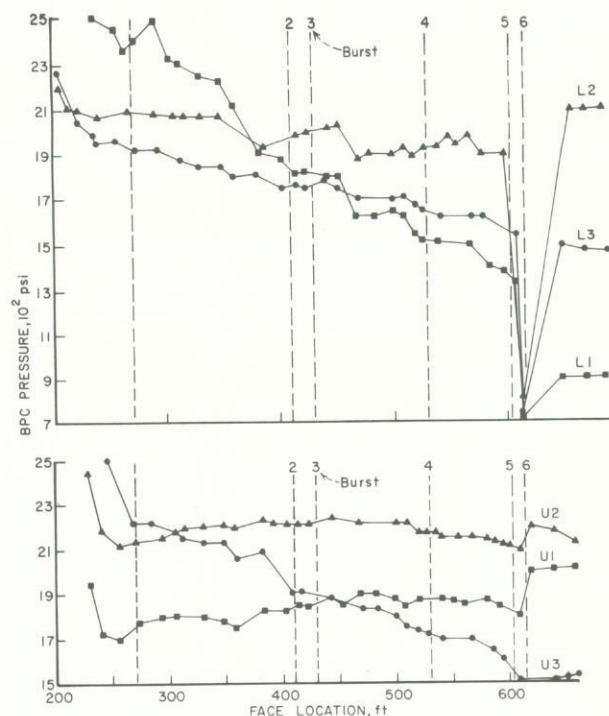


Figure 9. Pressure changes versus face location plots for upper and lower packwalls.

U2 are not affected because the upper packwall is partially confined by the gob from the previously mined panel and loaded by continuing roof deflection toward the gob. Prior to the April 20 burst, all cells showed a decrease in pressure; after the burst, all cells rebounded to the preburst pressure except that cells U3 and L1 did not regain the preburst pressure, further evidence of the effect of floor heave on the packwall pressure.

Instrumentation at site 1 indicated that the forward abutment extended approximately 500 ft ahead of the face, figure 10. The cell pressure in the panel

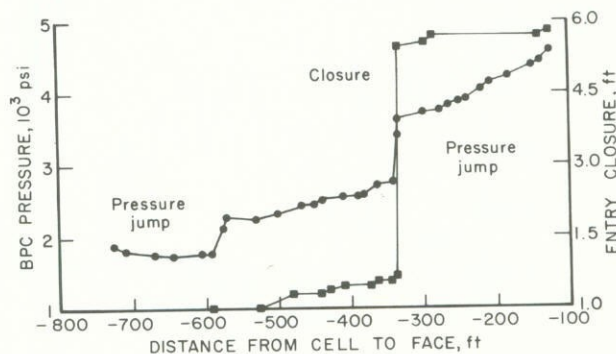


Figure 10. Pressure and entry closure versus distance from cell to face.

began to increase when the face was still 500 ft away. Of special interest is the sudden pressure increase resulting from the burst occurring after 618 ft of advance, when the face was 330 ft away from the cell. Postburst panel behavior, figure 11, shows not only dramatic

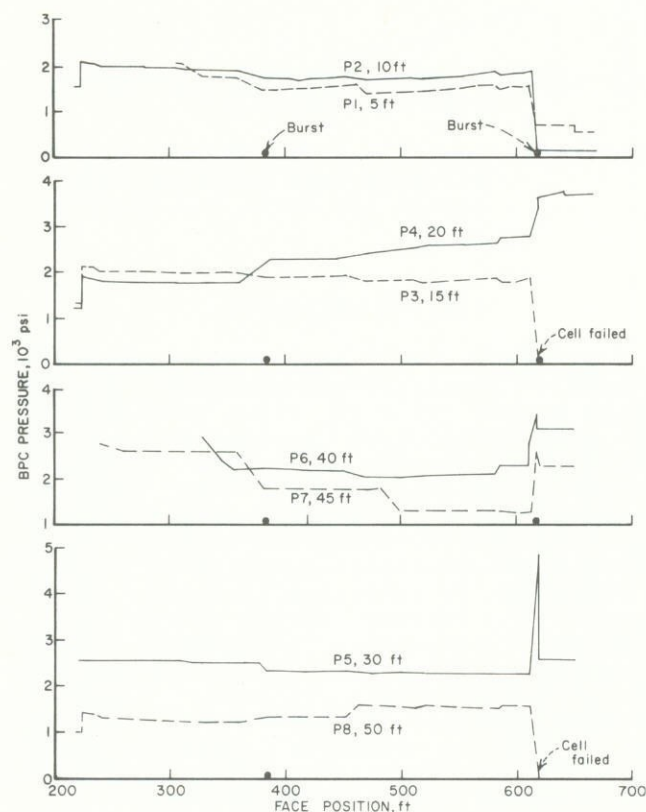


Figure 11. Pressure changes in the panel at station 1 versus face position.

pressure increases, but also pressure redistribution. Panel stresses increased significantly from 20 to 40 ft inside the rib, while those cells installed at depths of less than 20 ft showed pressure decrease. Reduction of the structural integrity and load-carrying ability of the panel edge is confirmed by rib expansion measurements shown in figure 12. Cumulative rib expansion was approximately 1.85 in at the collar and decreased with distance into the rib. Readings before and after the burst differed at the collar by 0.85 in and decreased with depth into the rib. Subsequent readings from this instrument were not possible because of the floor heave in the tailgate entry.

Owing to the tailgate floor heave caused by the burst, access to areas ahead of the face was not permitted. This made

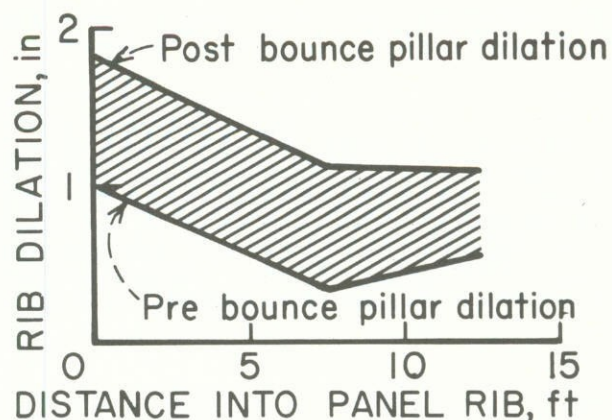


Figure 12. Panel rib dilation before and after the April 20 burst.

it difficult to further record data from station 2. However, subsequent readings indicated no burst effects on station 2, approximately 1,400 ft away from the burst location. Figure 13 shows the difference in stress levels before and after a burst obtained from instruments in the panel at stations 1 and 2.

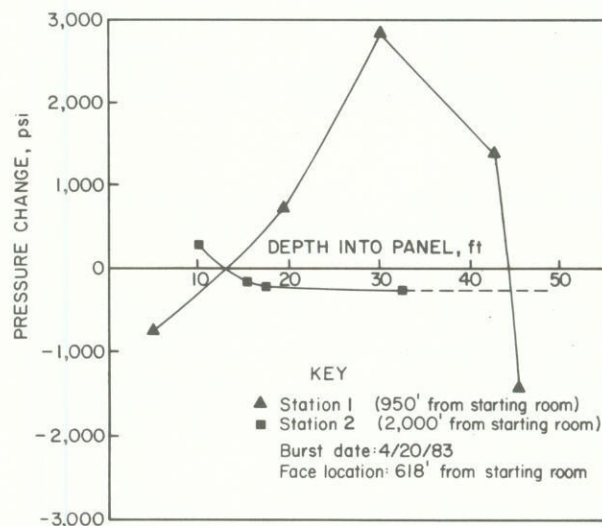


Figure 13. Pressure change from cells in panel at stations 1 and 2.

Room Closure

Approximately 5 ft of roof-to-floor convergence at station 1 resulted from the April 20 burst. Convergence data from 10 other stations, located on 50-ft centers from 850 ft to 1,300 ft in by the starting room, indicated a mean forward abutment extent of 570 ft ahead of the face. Analysis of abutment-induced closure rate determined a mean closure rate of 0.004

ft/ft advance, but did not provide any possible precursors to the major burst.

Similarly, roof-layer movement, obtained from the multiple-point extensometers located in the roof of the upper and lower entries of the tailgate at station 1, indicated a nearly uniform rate of immediate roof deflection with approach of the face. The roof-layer separations versus face advance for three horizons above the coal seam are shown in figure 14.

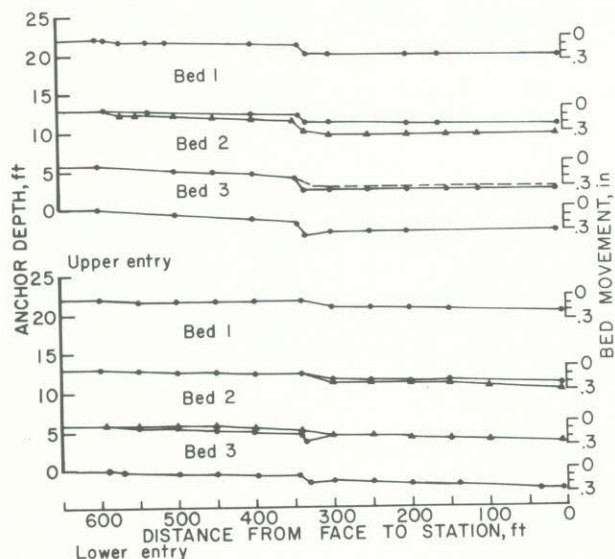


Figure 14. Roof layer separation versus distance from face.

As observed from the pressure cell data, the burst-induced pressure increase was accompanied by increased roof movement. Preburst roof deflection measurements indicated that deflection was greater for the lower roof beds, and that for anchors at the same depth, those installed in the upper entry generally exceeded lower entry roof movement. As the upper entry is adjacent to the previously mined gob and is farther out under the cantilever beam extending from the panel toward the gob, greater roof deflection should be expected.

Data analysis obtained before and after the April 20 burst, figure 15, indicates that floor failure may have centered about 80 to 100 ft inby the face. Floor failure may be partially due to the horizontal stress conditions, lack of roof caving and high abutment pressures, lateral loading from the previously mined longwall panel, or a combination of these and other factors.

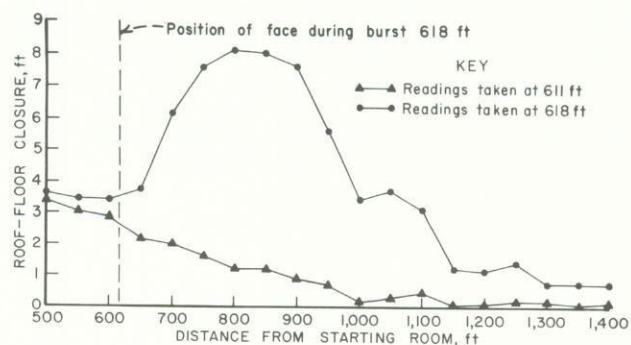


Figure 15. Roof-floor convergence versus distance from starting room before and after the April 20 burst.

FAILURE MECHANISM

Conditions at this mine are very complex. Contributing factors include depth; strong roof in close proximity to the coal seam that apparently overhung behind the shields near the tailgate; lateral loading from the uphill, previously mined longwall panel; a floor comprised of a soft coalbed overlain by a strong shale-sandstone layer which varying in thickness at different locations and is capable of storing strain energy; and a principal secondary horizontal stress component oriented approximately along the axis of the tailgate entries. While a detailed analysis is premature, the field data suggest a possible explanation for the April 20 burst that generally agrees with observed and measured behavior.

The forward abutment extended to approximately 570 ft ahead of the face, with significant pressure increases occurring 20 to 40 ft inside the panel rib. Abutment pressure increases consisted of two components: (1) a gradual increase associated with load transfer of the roof onto the panel, and (2) two dynamic load increases that followed documented burst occurrences.

Superimposed onto the forward abutment loading is the effect of an apparent 145-ft-long cantilever roof beam extending behind the near-tailgate shields. Concurrent with the forward abutment loading is the lateral thrust from the previously mined panel onto the tailgate packwalls and panel rib. Lateral loading effects are most evident from roof deflection measurements that indicate roof inclination toward the previously mined gob and less evident from packwall pressure measurements. Prior to the April 20 burst, upper packwall pressures exceeded those measured in the lower packwall, but both revealed higher core pressures than near-rib pressures.

Following the burst, the lower packwall lost pressure, while the upper one remained essentially undisturbed.

As entry closure measurements indicated that mining-induced convergence preceded the longwall face, a possible explanation for packwall behavior is presented. The shale-sandstone rock split is not only loaded by the horizontal stress field, the principal secondary components being oriented approximately along and normal to the entry axes, and combined forward and lateral abutments, but is also loaded from below by a soft, possibly plastic, coalbed. With approach of the face, floor movement from under the packwall ribs increased and contributed to decreases in packwall rib pressure.

The burst, occurring after 618 ft of advance and presently conjectured as centered approximately 80 to 100 ft in by the face, imposed a sudden, dynamic load onto the already stressed tailgate entry. The ensuing burst resulted in a sudden uplift of the rock split and underlying coalbed. Destruction of the floor was accompanied by further dilation of the panel rib and panel rib pressure redistribution. Load-carrying capacity within 60 ft of the rib was generally lost with the exception of a 20-ft-wide zone from 20 to 40 ft within the panel. Load transfer to other areas of the panel was not measured.

Postburst packwall response is apparently the result of the floor behavior. The lower packwall, adjacent to the heavily damaged lower tailgate entry, showed lower postburst pressures, which may be explained as due to reduced bearing strength when the floor broke. The packwall essentially settled into the shattered floor debris. The upper packwall, somewhat confined by the adjacent gob and subjected to less floor heave in the upper tailgate entry, retained its preburst integrity and pressure levels. While some upper entry floor heave occurred, any pressure drops associated with heaving are believed to have been offset by the additional measured settling and rotation of the immediate roof onto this packwall. Postburst roof behavior suggests that the roof was supported by the upper packwall and also by the structurally competent 20-ft zone within the panel rib.

The April 20 burst is hypothesized as resulting from a complex combination of mining-induced and tectonic stresses, a hanging roof, and additional loading from lower coal bed plastic flow. High vertical stress conditions resulted from overburden stress, additive forward and side abutment loads, and apparent

cantilever effects. Lower coal bed deformation contributed to the floor failure by inducing additional flexure stresses within the rock split.

As critical pressure levels were reached, the floor violently failed, drastically reducing confinement to the panel and the bearing capacity of the packwall-floor components. Loss of confinement under the panel rib was accompanied by increased dilation and loss of structural integrity, as evident from the postburst panel pressure redistribution. Relationship between packwall behavior and floor heave is supported by convergence and pressure measurements, as detailed above.

Loading due to noncaving of the roof apparently contributed to the high-stress conditions; failure of the sandstone roof member may have imparted a sudden load onto the tailgate entries and panel. This preliminary analysis suggests that high stress, combined with dynamic loading, triggered the burst.

RECOMMENDATIONS FOR MINING UNDER STRONG ROOF

As the longwall face advances, the roof layers behind the supports separate and cave. Depending on the geology, physical properties, and thickness of roof layers, the immediate roof may cave into the gob area immediately following the advance of the shields. The main roof breaks and falls periodically along the direction of the face advance and imposes periodic roof weighting (impacts) on the face. Depending on the exposed roof length and the type of floor rock, the impacts may exert a dynamic loading to the shields and face. Consequently, the powered supports may be damaged because the impact loading occurs in a very short period of time, and the yielding values do not react in time. If the floor rock is soft, the powered supports may sink into the floor under the roof pressure. This will reduce the supporting capacity of the shields and becomes a problem in advancing the supports. When the roof is thick, strong, and does not readily cave, it overhangs behind the powered supports and forms a cantilever beam extending into the gob; initially, the beam deflection is small, and no major increase in the abutment stress is noticed. However, as the face advances, the strong roof gradually overhangs for a long period of time or over a large span. Its weight, supported by the shields and face, causes the shield leg pressure and the front abutment stress to increase. When the stress in the coal exceeds the ability of the coal to store strain energy, a burst occurs.

Current methods that address the problem of high stress buildup in the face area include detection and destressing of high-stress zones by volley firing, auger drilling, or hydraulic fracturing. Such methods have proven effective in reducing burst occurrences. After destressing the coal face, the structural integrity of the coal is lost, and the abutment stress is advanced ahead of the face; however, the immediate cause of the high-stress condition has not been removed. By destressing the face to a depth of 5 ft, for example, the length of the roof beam has increased by 5 ft. This situation adds loads to the shields and to the abutment in the coal panel 5 ft ahead of the current face. If the roof does not cave behind the shields by the time the face advances 5 ft and the face is destressed again, even more load is applied to the shields and panel. Eventually, the cantilever beam becomes so long and has stored so much energy that it fails; when the beam fails in this manner, a significant amount of load and energy has to go somewhere--to the shields, panel, etc.--and bursts may occur.

Another method practiced at this mine was destressing the floor ahead of the face in the tailgate entry. When the floor in the tailgate entry is destressed, the floor heaves, which causes the pillars or packwalls to sink or lose confinement. This causes the roof to sag toward the gob from a previously mined panel, and the load carried by the pillars or packwalls will be transferred to the panel and shields, which adds to the burst conditions.

From this study, correlation between burst conditions and caving of the roof was shown. A conclusion is that when the roof does not cave regularly, burst problems have been recorded. The apparent solution is to induce regular caving in roof strata that do not readily cave.

There are two basic strategies to approach this problem: 1) Increase shield setting pressures to fracture the roof, and/or 2) induce fracturing of the roof by some other means. European experiences with hydraulic fracturing have shown the beneficial effects of fracturing the roof strata over the coal seam ahead of the face (8). However, with hydraulic fracturing, it is difficult to control the areal extent of the fracture area; fracturing too large an area may create, rather than reduce, ground control problems. Another method (7) uses large-hole blasting to initiate the cave behind the shields. Depending on the roof thickness and rock physical properties, the hole specifications were as follows:

- hole length = 13-23 ft
- hole diameter = 1.6-2.4 in
- inclination angle = 60°-65° from horizontal
- horizontal hole spacing = 20-33 ft
- distance from blast hole to canopy = 1.3-2.0 ft

A promising technique currently under investigation at the Bureau of Mines is the use of expansive grout, which was proven successful in fracturing large rocks without the use of explosives in the laboratory and in surface mines. The expansive-grout method uses lime compounds that, when mixed with water and confined in a small borehole, generate expansive force of up to 9,000 psi by a chemical hydration reaction. The expansive force increases at a rate of 2,000 psi per day and allows fracturing to proceed without direct attention of workers. To create fractures in rock, holes may be located to act independently of each other or may be positioned so that the stresses from adjacent holes interact to generate cracks expanding from hole to hole along the line of the holes drilled. Required equipment includes a rock drill, small slurry pump, plugs to stopper filled holes, and tubing to bleed off air as holes are drilled. This method has the capability of fracturing rock without the disadvantages associated with blasting, such as noise and vibration, and it can be used in gassy mine environments without the added risks of explosives.

Even though induced caving may be an effective technique to control bursts in underground coal mines, more field data are required to evaluate the effectiveness of different fragmentation methods given specific underground conditions.

CONCLUSIONS

This case study data analysis indicates that the burst on April 20, 1983, resulted from high forward abutment pressure compounded by noncaving of the strong sandstone roof layer. Shield loading measurements, apparently the best indicator of abnormal conditions, revealed that while the headgate and center portion of the face underwent periodic weightings, the near-tailgate face area experienced a substantial period of high loading prior to the burst. While the exact cause of the burst is unknown, a combination of high local stresses and the presence of both strong roof and floor contributed. Floor heave in the tailgate caused loss of confinement of the pillars or packwalls, and the load carried by these pillars was transferred to the panel, which added to the burst conditions.

Closure data suggest that the burst centered on an area 80 to 100 ft ahead of the face, but discrete closure and closure rate measurements did not provide any indicators (precursors) to the event.

Even though current techniques for detecting and destressing high stresses in the face area have proven effective to control bursts during face advance, they do not eliminate the basic cause of the high stress, the ability of strong, near-seam strata to accumulate strain energy. A promising technique is the use of expansive grout to induce controlled fractures without the use of explosives. Induced fracturing may be an effective burst control technique; however, extensive future research and field evaluation are needed to answer the many questions that exist.

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MINE LAYOUT DESIGN FOR COAL BUMP CONTROL

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ABSTRACT

A comprehensive study consisting of stress determinations, core logging, laboratory testing, and numerical analyses was conducted to investigate the cause and potential alternatives to the coal bump conditions that developed in the Second North Panel of the Little Dove mine near Huntington, Utah. Roof rock characteristics were such that a cave did not develop above this 420 ft wide panel. After 700 ft of pillar retreat, severe coal bump conditions developed during pillar mining. The coal bump conditions resulted from excessive pillar stress which was caused by load transfer from the extracted, noncaving, portion of the panel.

Site specific data and numerical modeling techniques were used to back-analyze the bump prone condition. This back-analysis lead to the identification of critical stress, closure, and energy release levels. These critical levels were then used to evaluate alternative pillar mining sequences in the hope that the potential for coal bumps could be reduced. It was found that while alternative sequences offered some potential improvement over the pillar mining sequence used in the mine, alternative pillar mining sequences would not significantly reduce the potential for bumps.

For the roof characteristics at this mine, coal bumps are best avoided by developing narrow panels or significantly wider panels, which incorporate properly sized barrier pillars. Narrow panels, on the order of 300 ft, would not cave, but panel pillar stresses and the potential for bumps would be moderated by the reduced panel width. Wider panels, of approximately 650 ft, would allow a cave to develop, also reducing panel pillar stresses and coal bump potential.

INTRODUCTION

The Little Dove mine is one of six mines operated by the Utah Power and Light Company in the Wasatch Plateau near Huntington, Utah. Within the Little Dove mine, the Blind Canyon Coal Seam is mined by room and pillar development with pillar extraction. In the summer of 1986, severe coal bump conditions developed during pillar extraction in the Second North Panel. This paper describes an investigation which: 1) identified factors contributing to the formation of bump prone conditions, 2) evaluated pillar extraction alternatives for developed reserves, and 3) projected optimum panel layouts for underdeveloped reserves. Core logging, laboratory testing and in-situ stress determinations were conducted in order to characterize the geomechanical conditions leading to the occurrence of coal bumps. This information was used as input to numerical analyses of the Second North Panel geometry and pillar extraction sequence. Following this back-analyses of bump prone conditions, numerical analyses of alternative mining sequences and panel/barrier pillar designs were conducted.

BACKGROUND

Sudden, violent failures that occur in the proximity of underground coal mine excavations are commonly referred to as coal bumps. Other terms, such as rock burst, crump, mountain shot, mountain bump, pillar burst, pressure burst, quake, and bounce are used interchangeably.

A combination of geologic and mining factors is responsible for the formation of coal bumps. Reported geologic factors include: 1) a depth of cover exceeding 500 ft, and 2) the existence of strong roof, floor, and coal rocks which will not yield readily (1). Related mining factors include the mining sequence and panel

layouts which produce areas of high stress concentration. Since the operator does not have control of geologic factors, mining factors are the only variables that can be selected to minimize the occurrence of coal bumps.

Panel width, barrier pillar sizes, panel pillar sizes, and panel orientation are geometric factors which impact mining induced stresses, and thus, the development of coal mine bumps. In order to identify the critical levels of such factors, mine maps were reviewed to identify locations where either recovery was significantly reduced or where coal bumps were reported. From this review, the following geometric factors were identified for panels in the Blind Canyon coal seam under 1200 ft of cover:

- o The larger the panel width, the better the caving conditions. Good caving conditions minimized the occurrence of bumps and enhanced resource recovery.
- o Odd sized pillars or unmined stumps in a wide panel hindered the cave, lead to bumps, and reduced resource recovery.
- o Barrier pillars, wider than 70 ft, minimized load transfer from previously mined panels to active panels and reduced the occurrence of bumps.
- o Reported coal bumps were associated with the extraction of 80 ft by 80 ft panel pillars. The use of 60 ft by 60 ft panel pillars minimized the occurrence of bumps, but lead to poor resource recovery in two panels.
- o Thirty foot wide fenders, associated with the extraction of 80 ft panel pillars were more likely to experience bumps than 20 ft wide fenders, associated with the extraction of 60 ft panel pillars.

While coal bump conditions occurred at several locations in the mine, the most severe coal bump conditions were associated with pillar extraction in the Second North Panel. The Second North Panel was 420 ft wide consisting of four 80 ft square pillars and five entries. The barrier pillar that separated the Second North Panel from the previously mined Third North Panel was 50 ft wide.

Each row of four pillars in the Second North Panel was extracted according to the sequence shown in Figure 1. Mining cuts 1 through 12 extracted the two pillars on the right. This same sequence was then repeated to extract the two pillars on the left. Seven rows of four pillars were

extracted in this manner. However, a cave did not develop in the gob. This resulted in an area 420 ft wide and 700 ft long which transferred its load to the remaining panel pillars, the barrier pillar, and the solid block.

In the eighth row of pillars of the Second North Panel, mining cuts one and two were made as indicated in Figure 2. A high stress concentration existed to the right of cut one. Therefore, the mining sequence was modified as indicated in Figure 2. While mining cut 5, a roof fall trapped the continuous miner; this was followed by violent failure of the 3 ft thick mudstone roof along a portion of cut 2, and the occurrence of a significant coal bump within the fenders.

Factors Related to the Second North Coal Bump

The major factors related to the Second North coal bump include panel geometry, the lack of roof cave in the gob, geology and material properties of the roof, and load or stress transfer. The panel geometry is described in the previous section. This section discusses the geology and mechanical properties of the Second North Panel and the stress distribution which lead to the coal bump.

The material above the Blind Canyon coal seam consists of competent sandstones and interbeds of siltstones, sandstones, and shales. The total percentage of sandstones in the overburden is between 34 to 45 percent, with 75 percent of the sandstone occurring as thick sandstone beds (2). The lithology in the Second North Panel was evaluated by drilling vertical core holes into the roof and floor (see Figure 2). The lithology and Rock Quality Designation (RQD) observed in these vertical core holes is illustrated in Figure 3, which demonstrates the occurrence of thick-bedded strata adjacent to the Blind Canyon Seam. Laboratory tests on core obtained from the roof confirmed the existence of strong/stiff roof members with unconfined sandstone strengths averaging 14,500 psi and mudstone/siltstone strengths averaging 10,250 psi. Young's moduli for these materials was also relatively high for coal measure strata, averaging 5.7 million psi for sandstone and 2.8 million psi for mudstones/siltstones.

A series of thick, competent roof members can resist breakage above narrow panels and prevent the formation of a good cave behind the pillar line. Without a good cave, excessive loads and high stress concentrations are transferred to the barrier pillar(s) and to the panel pillars yet to be extracted.

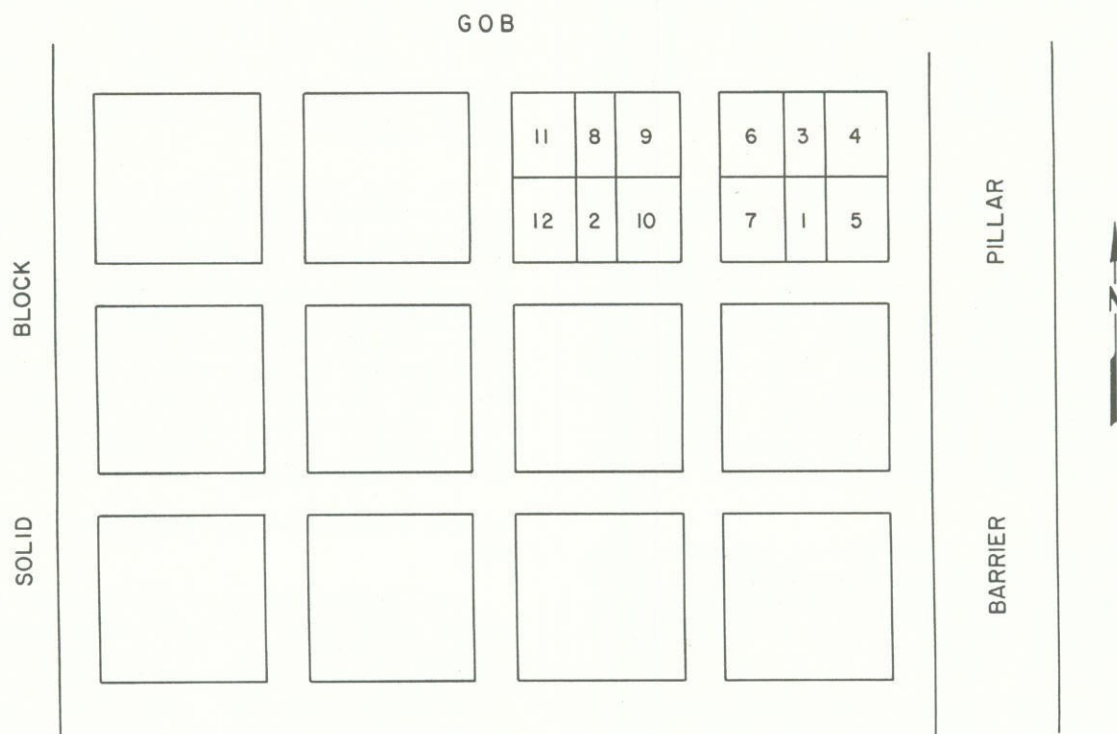


Figure 1 Second North Mining Sequence for Two Pillars - This Sequence is Repeated for the Two Pillars to the Left

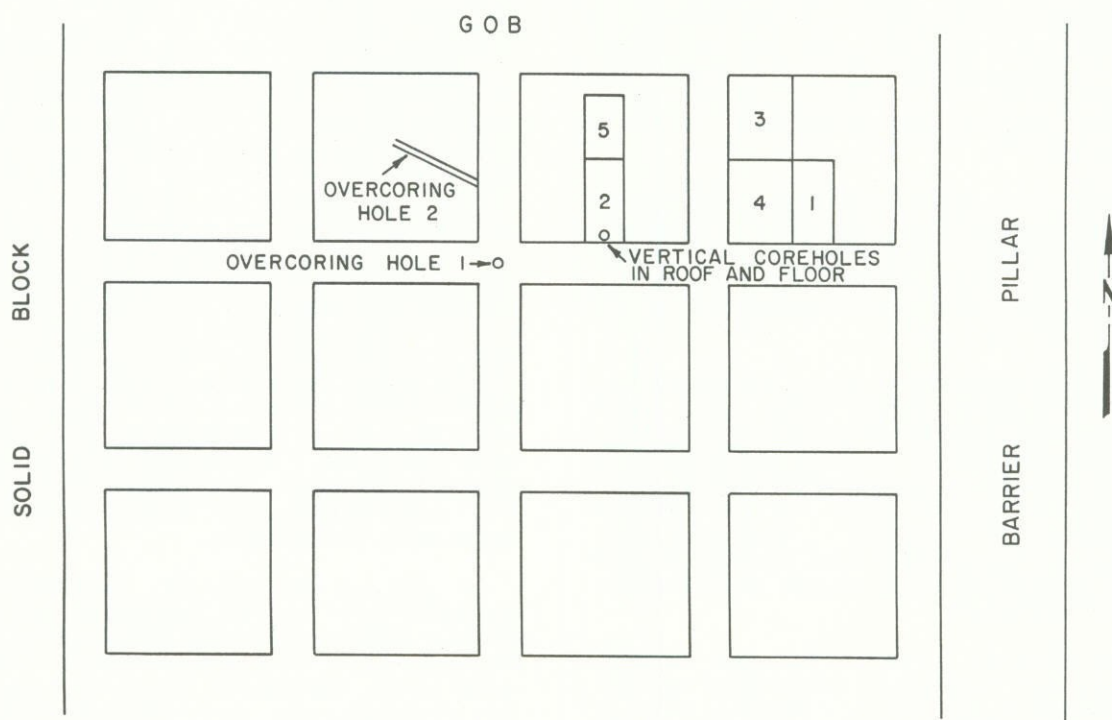


Figure 2 Mining Sequence and Hole Locations in the Coal Bump Area of the Second North Panel

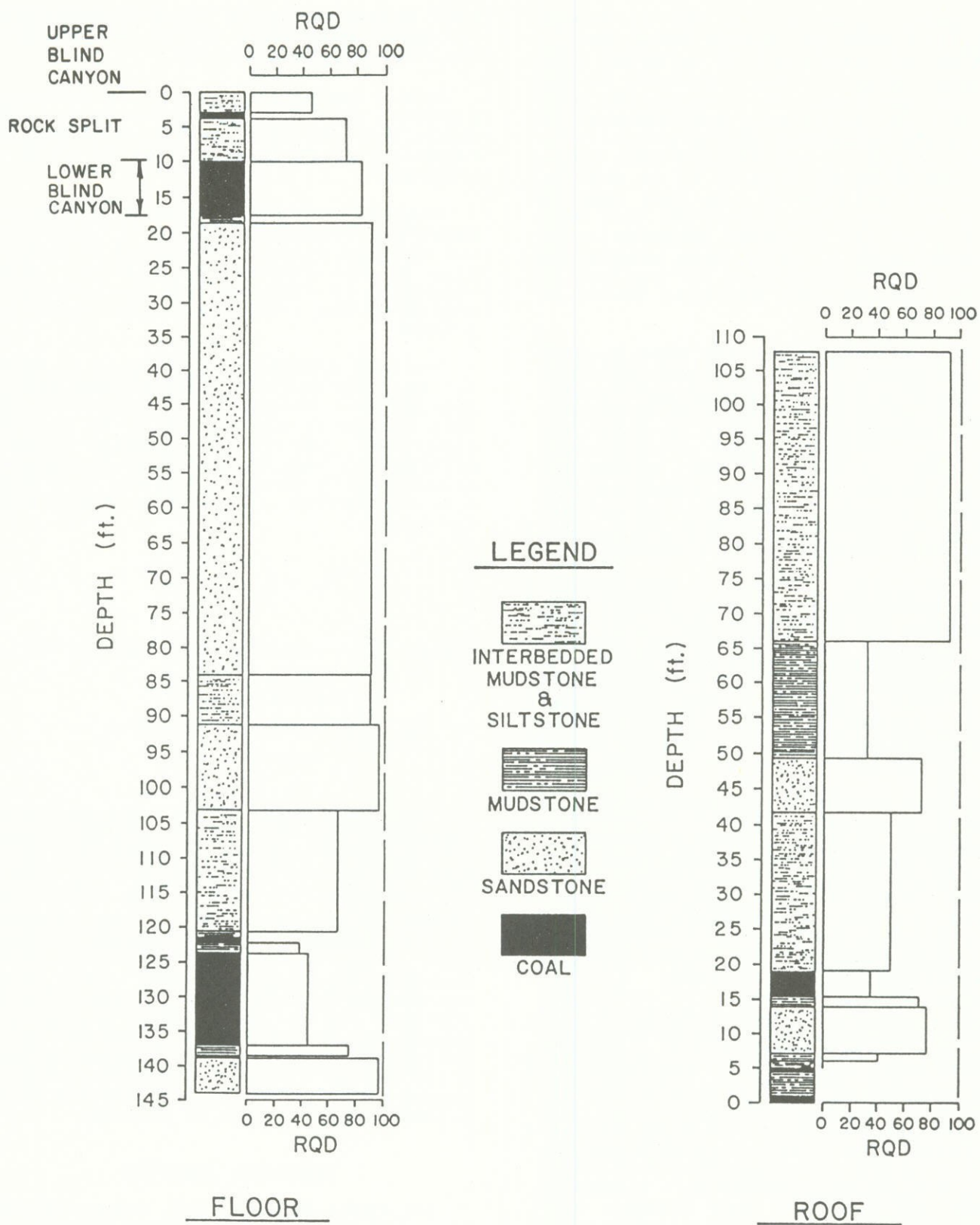


Figure 3 Roof and Floor Lithology and RQD in Second North Panel (see Figure 2)

In order to quantify the stresses associated with the Second North Panel, stress relief overcoring (3) using the USBM deformation gage (4) was conducted in two boreholes (see Figure 2). Overcoring hole 1 was drilled vertically into the roof to quantify the horizontal stress in the roof. Overcoring hole 2 was drilled near horizontal and above an 80 ft by 80 ft pillar. Hole 2 was drilled in the roof rock to determine the stresses above the coal pillar. For the specific orientation of overcoring hole 2, overcoring activities produced stress information for a plane, 15 degrees from vertical striking North 25 degrees East.

In the vertical overcoring hole 1, poor core recovery in the immediate mudstone roof inhibited the overcoring process and prevented successful stress determinations. At a distance approximately 5 ft above the roof, the rock type changed to a sandstone which produced excellent core recovery. Five successful stress determinations were conducted in this sandstone. Horizontal principal stresses resulting from these measurements are presented in Table 1.

As indicated in Table 1, the maximum horizontal stress decreases rapidly with increasing distance from the roof. The rate of stress decrease can be used to estimate a maximum horizontal stress of approximately 1500 psi in the immediate roof. This relatively high horizontal stress resulted from high loads on the pillars and the cantilever action of the uncaved roof over the extracted pillars.

The core log for overcoring hole 2 is shown in Figure 4. Four overcoring attempts were conducted between 3.9 and 7.2 feet. Of these four attempts, the core failed prematurely during reliefs 1 and 3. However, overcoring attempts 2 and 4 (Figure 4) produced excellent overcoring data. In order to rapidly advance hole 2, 6 in. diameter core with no center hole was drilled from 7.2 to 16.8 feet. In this interval, some core discing occurred; however, core recovery indicated that overcoring should continue to be relatively successful. Core discing is the phenomenon by which rock, under stress, fractures into discs or wafers of relatively uniform thickness during the drilling process. The EW pilot hole was extended from 16.8 to 21.9 ft in hole 2, and overcoring attempts were initiated. Between 16.8 and 19.25 ft, 10 overcoring measurements were attempted. However, due to severe core discing, none of these attempts produced useable overcoring data. The discs in the fine-grained sandstone were regularly spaced with a thickness of 1 to 1.5 inches. The discs in the dark, sandy shale that occurred at 19.25 ft were much thinner than those in the sandstone. The abrupt decrease in the thickness of the discs indicated that the

rock stress magnitude had increased and/or the dark, sandy shale was not as strong as the fine-grained sandstone.

Hole 2 was extended to 26 ft with continuous, thin discing in the dark, sandy shale and the occurrence of borehole crushing or spalling in the 24 to 26 ft interval.

Table 2 summarizes the stress determinations obtained in hole 2. Note that the maximum stress is rotated toward the area of the extracted pillars. This rotation is evidence of the cantilever action of the roof above the extracted pillars.

The core discing that occurred in hole 2 can be used to infer some stress related information. Discing is an indication that rock stress is high relative to the rock strength. Laboratory studies of core discing (5) have resulted in an empirical relationship between stress and rock characteristics. This relationship takes the form:

$$S_r = k_1 + k_2 S_a \quad (1)$$

where: S_r = radial stress required to cause discing
 S_a = axial stress
 k_1, k_2 = constants depending on rock type

Of the rock types tested by Obert and Stephanson (5), Nova Scotia sandstone with a compressive strength of 14,200 psi is the most similar to the sandstone of hole 2. The constants k_1 and k_2 for Nova Scotia sandstone are 6500 psi and 1.0, respectively. These constants and an estimated axial stress of 500 psi, indicate that the maximum compressive stress which caused the discing in hole 2 was on the order of 7000 psi. If the relationship between this maximum stress and the vertical stress is similar to the results presented in Table 2, it can be inferred that the vertical stress acting on the central portion of the pillar is on the order of 4900 psi.

NUMERICAL ANALYSES

The three-dimensional displacement-discontinuity program EXPAREA (6) was used to calculate stresses, entry closures, and released energy for the different mining sequences. Table 3 shows the material properties and conditions used in the analyses. Properties were based on previous laboratory measurements (7) and measurements from the Second North Panel.

Table 1 Horizontal Stresses - Hole 1

Depth of Measurement (ft)	Principal P (psi)	Stresses Q (psi)	Bearing of P
6.3	663	262	N12 E
7.3	337	121	N44 E
8.3	230	191	N49 E
10.4	141	62	N59 E
11.4	120	63	N6 E

Table 2 Calculated Stresses - Hole 2

Measure-ment Depth (ft)	Relief No.	Principal P (psi)	Stress Q (psi)	Inclin-ation of P Toward Gob (degrees)	Verti-cal* Stress (psi)	Hori-zontal Stress (psi)	Shear Stress (psi)
5.1	2	3,990	515	30	3,085	1,420	-1,525
6.7	4	3,863	465	16	3,592	736	-920

* Actually 15 degrees from vertical due to hole inclination

Table 3 Material Properties and Stress Field

Property		
Coal seam modulus		0.35×10^6 psi
Roof and floor rock modulus		2.0×10^6 psi
Roof and floor rock Poisson's ratio		0.20
Seam depth		1200 ft
Initial vertical stress		1280 psi
Seam thickness		8 ft
Coal strength:		
Core element		4900 psi
Rib element		2820 psi

Post-Failure Characteristics	Modulus	Ave. Pillar Residual Strength
16 ft wide fenders	431,000 psi	2100 psi
30 ft wide fenders	230,260 psi	2820 psi
50 ft wide square pillars	137,920 psi	3520 psi
80 ft wide pillars	N/A	4380 psi

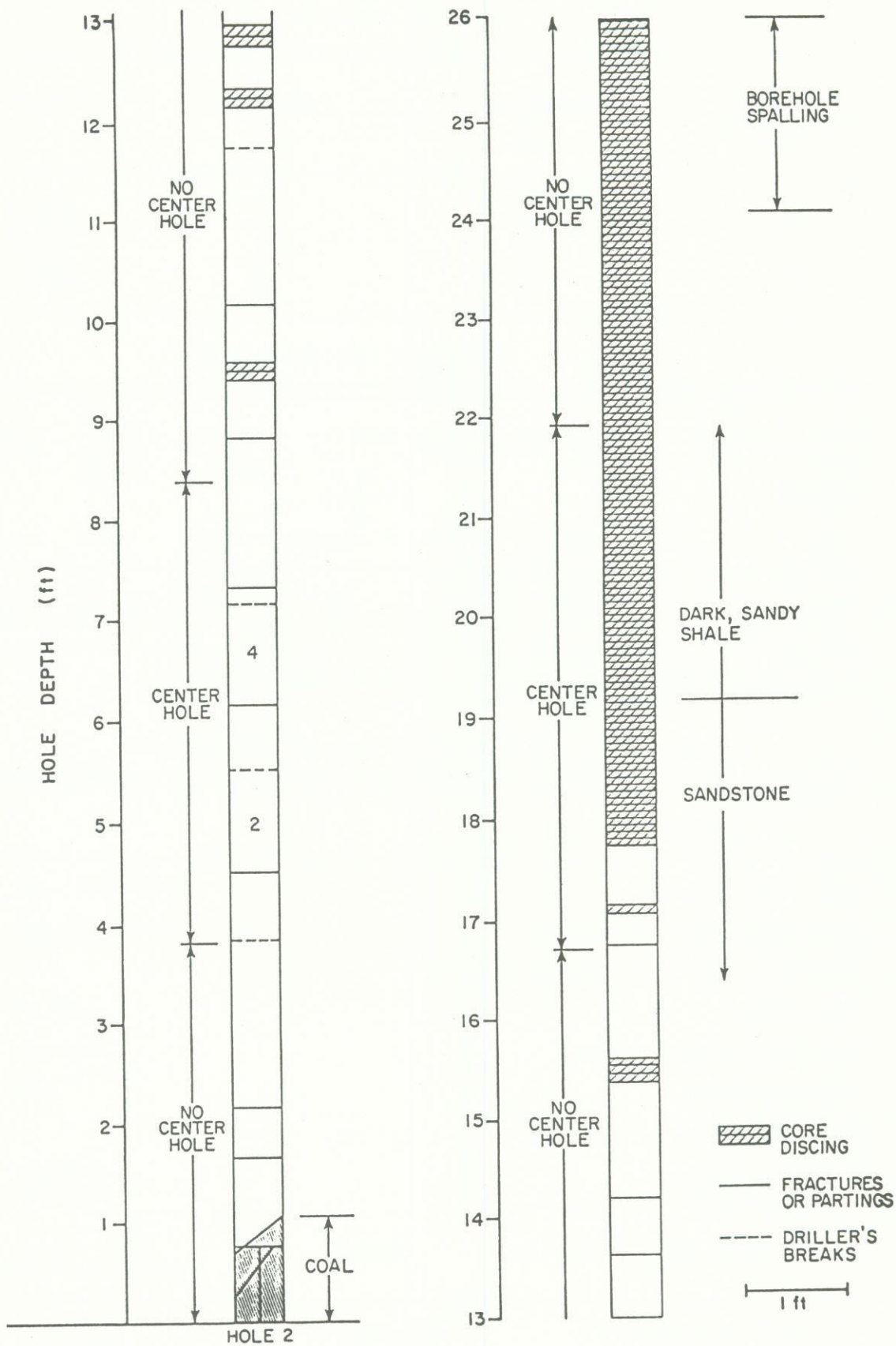


Figure 4 Fracture Log, Overcoring Hole 2

Coal peak strength and post-failure properties were estimated from laboratory tests of the Blind Canyon Seam (8), pressure measurements in coal pillars (7), and the overcoring stress measurements previously described.

The pillar peak strength was estimated from the overcoring measurements performed in the Second North Panel. These measurements indicated vertical stresses of 2820 and 4900 psi for the rib (outer 10 ft) and pillar core, respectively. Therefore, an 80 ft square pillar had an average peak pillar strength of approximately 4380 psi.

Pillar yielding was modeled by assuming either an elastic-plastic behavior or by a strength reduction to a residual value (Figure 5). All mining geometries were analyzed using the elastic-plastic pillar behavior, which required far less computer time than the residual strength behavior. Several analyses were repeated for the residual strength pillar behavior to determine the sensitivity of the results to post-failure behavior.

Cook et al. (9) have evaluated bump prone conditions from the standpoint of the energy changes that necessarily occur when an excavation is created. As a cut is made, a portion of the gravitational potential energy of the overburden rock is transformed into stored energy (in the form of stress concentration) and released energy. The released energy is dissipated by gradual crushing or by violent bumps.

The South African experience (9) has established a correlation between the maximum energy release rate and the severity of rock bursts in gold mines. This approach does not attempt to determine the amount of energy which can be absorbed non-violently. Crouch and Fairhurst (1) and Starfield and Fairhurst (10) propose an algorithm for such calculations based on the local stiffness of the mine roof in comparison to the post-failure behavior of coal.

In this study, energy release has been calculated by the following relationship suggested by Hardy and Christianson (11), (see Figure 6);

$$E = \frac{1}{2} \sum_{i=1}^n \sigma_i (\delta i_B - \delta i_A) A_i \quad (2)$$

where: A_i = area of each element (10 ft x 10 ft)
 σ_i = pre-cut stress on coal element i
 δi_A = closure at element i after cut is made

δi_B = closure at element i before cut is made

the energy release was calculated for those elements that are mined during any particular mining cut. Also, an average energy per cut was calculated by dividing the total energy release by the number of elements within a cut, this allowed a comparison of cuts of different sizes.

Critical levels of stress, closure, and released energy associated with bump prone conditions at the Second North Panel were determined using the mining geometry prior and subsequent to the occurrence of the coal bump. The vertical stress levels prior to the bump are shown in Figure 7 (assuming elastic-plastic behavior). Each number in this figure represents a 10 ft square element, the numerical value indicates the vertical stress magnitude at each element. Following underground observations, some load transfer through the gob was permitted for the Third North Panel, but no cave was modeled in the Second North Panel.

Mining geometries before and after cut 5 (see Figure 2) were modeled. Figure 8 shows the vertical stress levels for both elastic-plastic and residual strength pillar behavior. This back-analysis of the coal bump condition allowed the identification of critical levels of selected parameters as given in Table 4. The average pre-cut stress is the vertical stress on those elements to be extracted in the next cut; the closure is the average roof to floor closure within the cut; and highly stressed elements are those elements that had a vertical stress greater than 4900 psi. Both the elastic-plastic and the residual strength pillar behaviors indicated the potential for coal bumps. However, the residual strength pillar behavior more accurately predicted the location of the severe bump that occurred.

In order to compare modeling results obtained for the coal bump conditions described above to other pillar extraction sequences, additional analyses were performed. Three pillar extraction sequences were evaluated. These included:

- o Extraction Sequence 1 - normally used in the Second North Panel (Figure 1).
- o Extraction Sequence 2 - based on the South African experience that a uniform rate of energy release is more desirable than a highly variable rate of energy release. This sequence is illustrated in Figure 9.

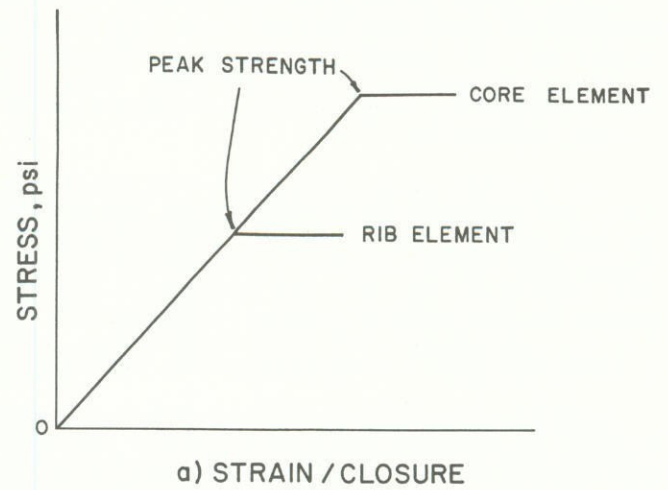
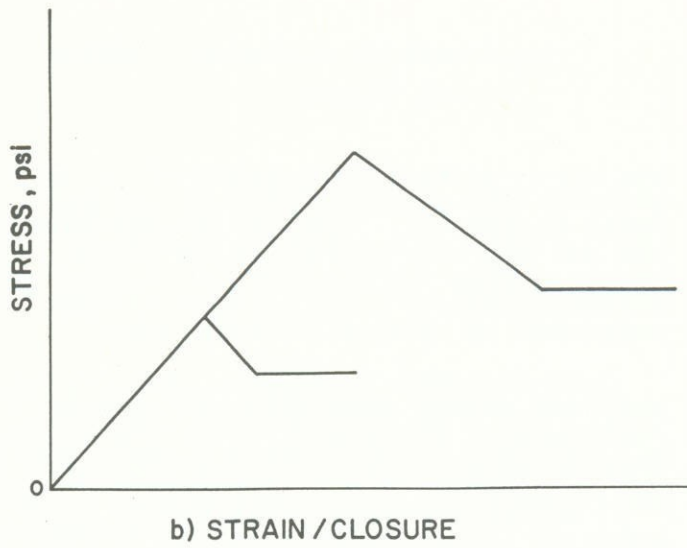


Figure 5 Modeled Coal Behavior

- a) Elastic-Plastic
- b) Residual Strength

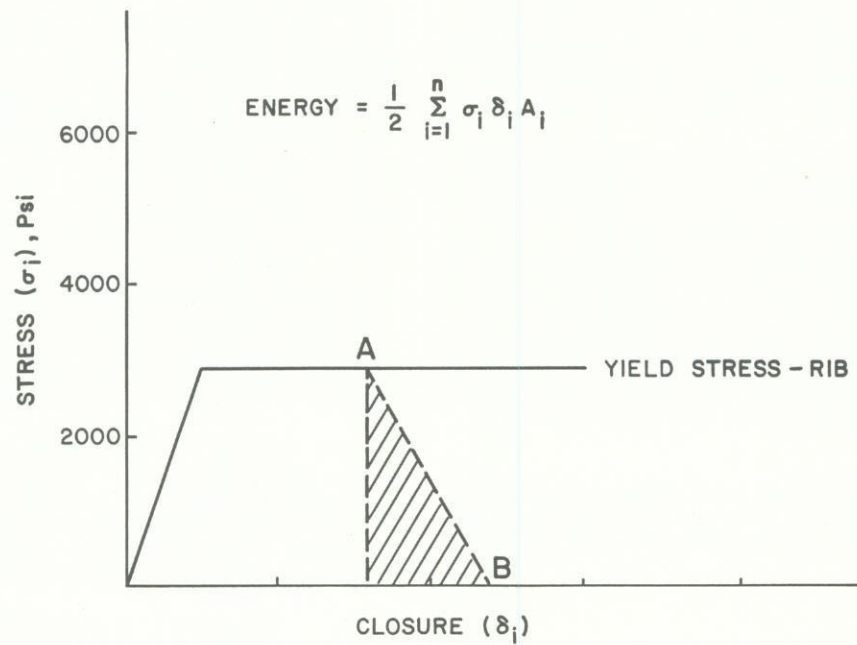


Figure 6 Energy Released Used in the Computation

VERTICAL STRESS LEVELS

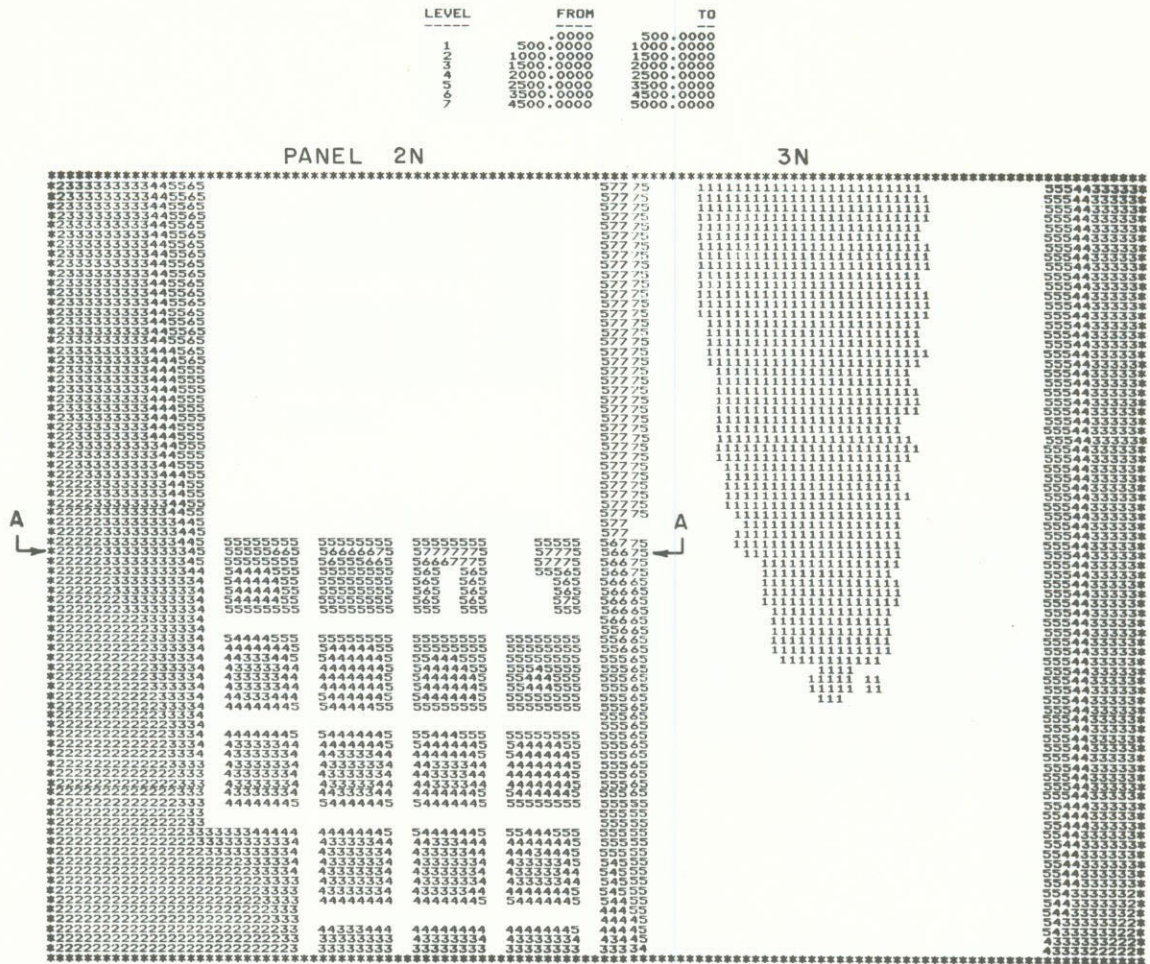


Figure 7 Vertical Stress Levels Prior to the Coal Bump

AFTER CUT 5

AFTER CUT 5

(b) Residual Strength

42

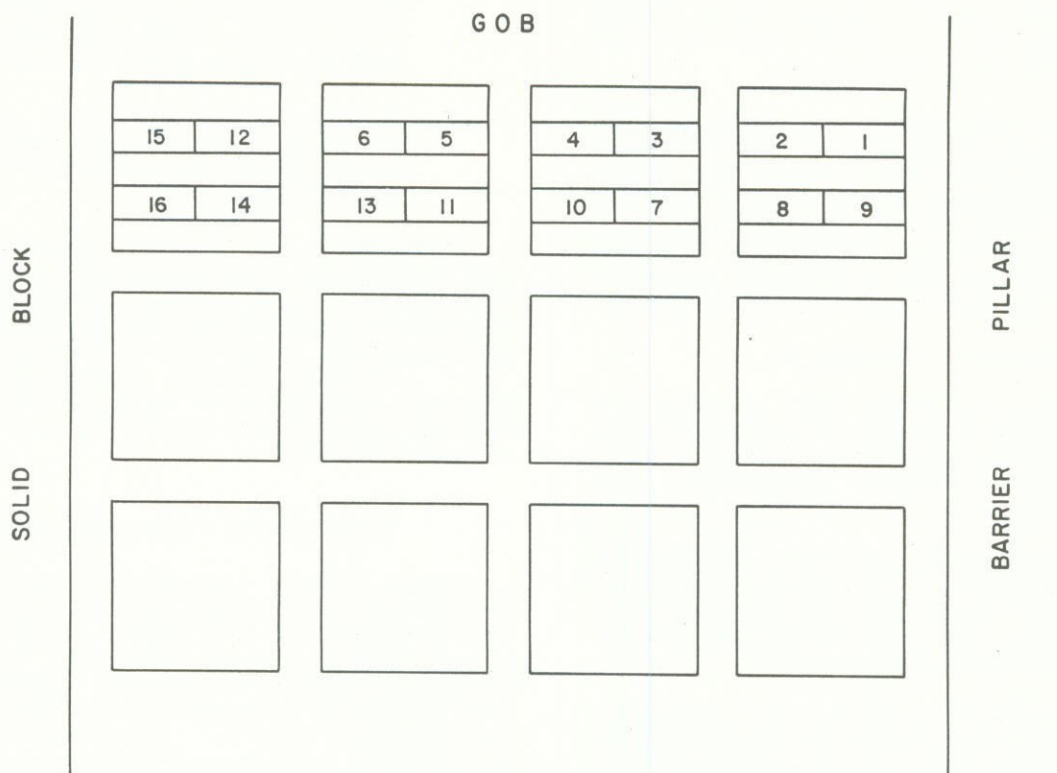


Figure 9 Mining Sequence 2 - Uniform Energy

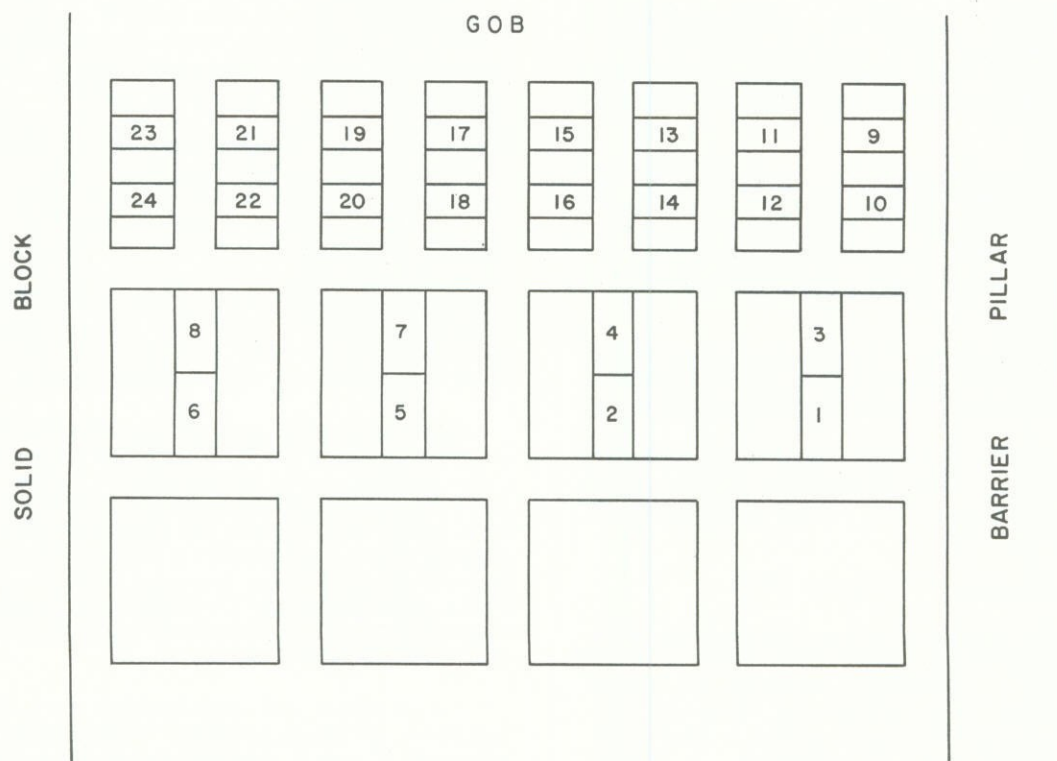


Figure 10 Mining Sequence 3

- o Extraction Sequence 3 - used to help control coal bumps in a West Virginia mine. This sequence is illustrated in Figure 10.

The average pre-cut stress, closure, number of highly stressed elements, and released energy were evaluated for each of the three extraction sequences and compared to the critical level criteria established for the coal bump conditions (Table 4). These comparisons lead to the following conclusions for each of the sequences considered:

Extraction Sequence 1 (Figure 1)

- o Pre-cut stresses were 5 percent higher for Sequence 1 and closures were up to 20 percent higher, this indicates a higher potential for roof falls.
- o The number of highly stressed elements and the released energy are increased by 10 percent for Sequence 1, thus, the potential for bumps during mining is increased.
- o By not leaving stumps to moderate stresses and closures at the face line, the potential severity of bumps would be greater with Sequence 1.

Extraction Sequence 2 (Figure 9)

- o Stresses, closures, and released energy were relatively constant for each cut, thus, mining would take place under more uniform loading conditions.
- o Stresses, closures and released energy are similar to critical levels when mining cuts 4, 9 and 10. Therefore, the development of instabilities would be most likely at those times.
- o This sequence eliminates highly stressed core elements by creating small fenders, this enhances safety.
- o This sequence is an improvement over Sequence 1, but does not eliminate the potential for bumps and/or roof instabilities.

Extraction Sequence 3 (Figure 10)

- o For this alternative, stresses are significantly lower than the critical level. However, stress is near the peak strength of the 30 ft fenders when mining cuts 9 through 24. This could result in roof control problems during the mining of these cuts.

- o For cuts 9 through 21, closures are significantly greater than the critical level, thus, the potential for roof failures would be increased.
- o Due to shorter cuts and lower stresses, the energy release rate is lower than the critical level for this alternative.

In summary, it can be concluded that for the conditions that existed in the Second North Panel, none of the three extraction sequences considered eliminated both the potential for bumps and the potential for roof instabilities. Therefore, mine planning should focus on alternative panel layouts which would avoid the occurrence of critical levels.

MINE LAYOUT CONSIDERATIONS

Panel pillar extraction transfers the weight of undermined rocks to panel boundaries, including the face. At the critical panel width, load transfer to the face creates stability problems. Critical width panels can be avoided by the use of narrow, subcritical panels isolated by barrier pillars of sufficient width, or by the use of super-critical panels which allow the formation of a good cave with associated stress relief at the face.

Currently, there are no well-defined guidelines for panel width selection that take into account the overburden characteristics. British experience suggests the width of subcritical panels be 1/4 the depth or less (12). Measurements in Western U. S. coal mines (13) indicate that a good cave will develop when the panel width is 55 percent of the depth. Therefore, following these general rules of thumb, the 1200 ft depth of the Little Dove mine indicates subcritical and super-critical panel widths should be on the order of 300 ft and 650 ft, respectively, as compared to the critical width of 420 ft that existed in the Second North Panel.

Barrier pillars play an important role in minimizing load transfer from mined out panels to active panels. The resources left in barrier pillars, however, dictate the need to optimize their width. British experience suggests that the width of barrier pillars be one-tenth the depth. This guideline indicates 120 ft wide barrier pillars would be appropriate for the Little Dove mine.

In order to evaluate the effect of barrier pillar width, the model shown in Figure 7, was reanalyzed with the barrier pillar width increased from 50 ft to 140 ft. The 140 ft width was selected because the Fourth North Panel would conveniently accommodate a 140 ft barrier pillar.

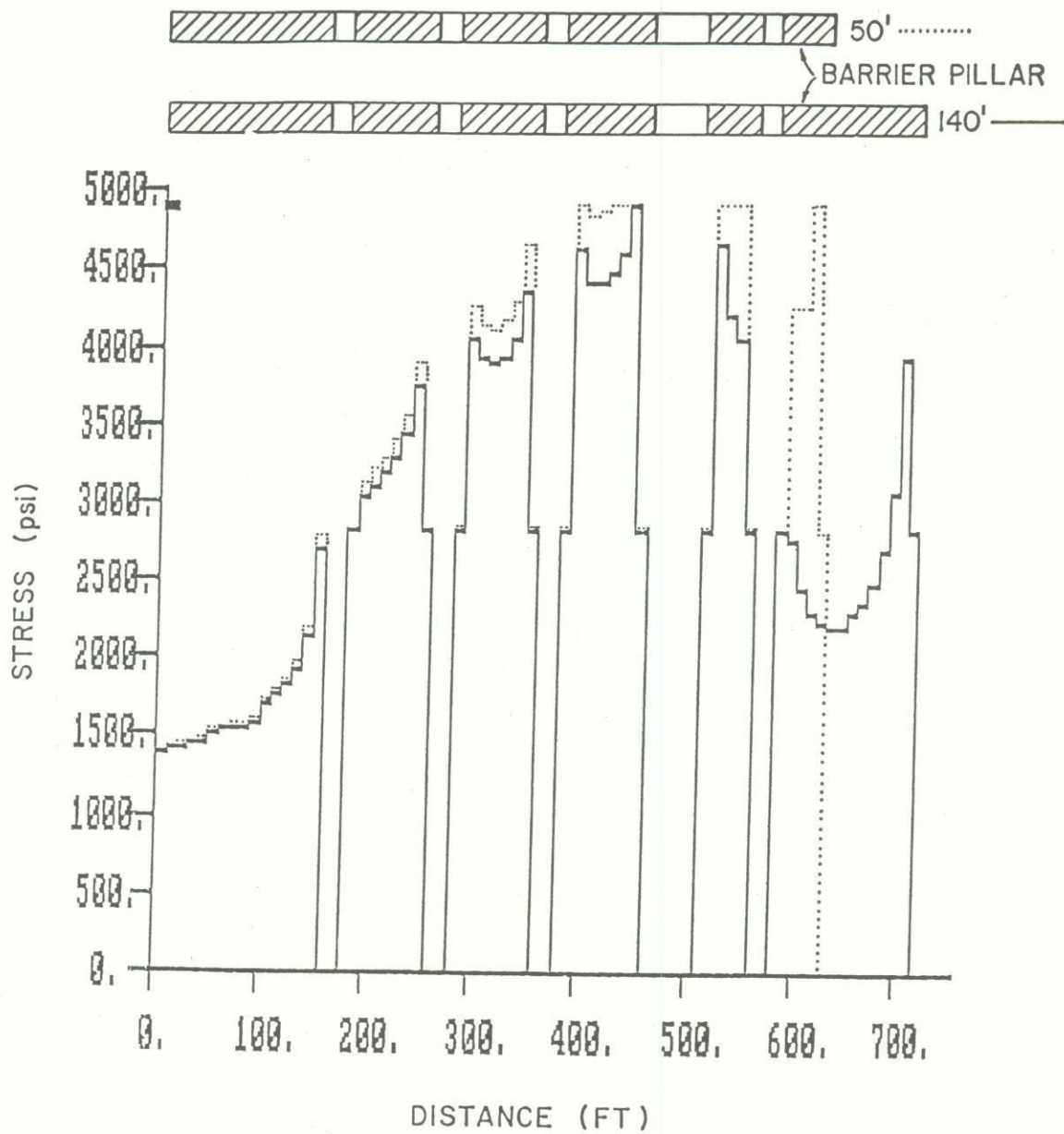


Figure 11 Comparison of Vertical Stress Levels for Two Analyses with Different Barrier Pillar Sizes (Along Section AA of Figure 7)

Figure 11 shows the stress profile along Section AA of Figure 8 for the two barrier pillar widths. This figure shows that an increase in barrier pillar width decreases the stress on panel pillars (especially those that are being extracted). This stress decrease would reduce the potential for coal bumps.

SUMMARY

The coal bump conditions that occurred in the Second North Panel of the Little Dove mine resulted from a combination of panel geometry and roof rock characteristics. The 420 ft panel width inhibited cave development in the competent roof members. Without a cave, panel loads were transferred to the panel boundaries creating coal bump conditions at the face.

Numerical analyses indicate that once coal bump conditions were established at a panel length of 700 ft, variations in the pillar extraction sequence do not significantly reduce the potential for coal bumps during pillar mining. Therefore, design activities should give careful consideration to the selection of panel widths and barrier pillar widths. These widths should be selected to avoid or minimize the development of conditions which lead to coal bumps.

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ABSTRACT

China has at least 200 collieries suffering from the problem of instantaneous outbursts of coal and gas. The strong outbursts often occurred in active tectonic zones and areas of high stress concentration. In this paper, the author describes some characteristics of these strong outbursts and analyses the effects of ground stress, gas pressure and the properties of coal in outbursts.

INTRODUCTION

In China, most coal mines are gassy. The instantaneous outbursts of coal and gas have occurred in 17% of the coal mines. About 10,000 outbursts, large and small, up to 12,780 tons of coal and rock have occurred in the last 35 years. The strong outbursts with more than 10,000 tons of coal and rock occurred 54 times in 1958-1980.

The total amount of coal and rock ejected in these outbursts was about 107,000 tons. Among them, the ones with intensity of 1,000-3,000 tons were 46 times; 3,000-5,000 tons, 5 times; 5,000-10,000 tons, 2 times; more than 10,000 tons, 1 time.

The ejected gas was almost always CH_4 and only in two outbursts it was CO_2 . However, the statistics of the ejected gas in every outburst was not exact, and varied between 0.02 and 3.50 million cubic meters.

According to the characteristics of the outbursts, i.e. ejected distance, angle of repose and sizing of the coal piles, those occurrences were typical of the outbursts. Sudden squeeze-outs and extrusions of coal had never exceeded 1000 tons. Those strong outbursts are very dangerous to coal mines, because they eject a lot of gas that can move upwind for a considerable distance. According to previous studies, those strong outbursts have the following characteristics.

A. Geological Conditions

According to the statistics, 85% of the outbursts occurred in bituminous and anthracite coal seams of the Carboniferous-Permian formation. In Sichuan, Kweichow, Hunan, and Jiangsi Provinces, there is always a thick limestone layer of higher strength and lower permeability above the outburst-prone coal seams. The limestone is capable of maintaining a high gas content in the coal seams and a strong elastic energy in the strata.

In addition, almost all outbursts were located in the faults and other areas of abnormal geological structures where the coal was crushed. In some areas, the coal formed mylonitic bands.

About 10% of the outbursts occurred in gas-rich and bituminous coal seams of Jurassic formation. In Liaoning province, magmatic intrusions were frequently found in outburst prone coal seams. It can not only cause the coal seam to fold, thin-out and fracture but also transform it into a fragile or hard natural coke. Outbursts usually occurred in zones of fragile coal.

After intrusion the strength of coal becomes ununiform. When a drift is driven into this area, an abrupt change of gas pressure and ground stress occur easily in front of the working face. This is an important factor to outburst initiation. In fact, the abnormal geological conditions provide the bases for outburst.

B. Ground Stress

In outbursts, the ground stress is the main energy source for initiating outbursts. Ground stress consists of primitive in-situ stress and mining induced stress. As observed in the measurements of ground stress in China, the horizontal stress is always more than the vertical gravitative stress in active tectonic areas. It is always 1.5-3.5 times, even 8 times as high as the vertical gravitative stress. However, the difference between them decreases with depth (Liao and Shi, 1983).

Among the strong outbursts, three occurred at the depths of less than 100 m; 16 at 100-200 m; 11 at 200-300 m; and 24 at more than 300 m.

In Hunan Province, most strong outbursts occurred at the depths of less than 200 m. In Zanbei Colliery, for example, a strong outburst with 1,300 tons of coal and rock occurred in a crosscut where the depth was only 95 m, and was not affected by mining activities in the neighborhood. It is reasonable to conclude that this outburst might be initiated by the horizontal ground stress because the vertical gravitative stress was only 2.5 MPa, that was insufficient to initiate an outburst.

The strong outbursts usually occur in structural zones formed by compression. For instance, on August 8, 1975, when the K₁ coal seam in Sanhui No. 1 colliery, Sichuan Province, was excavated by shock blasting at the main drift, a strong outburst with 12,780 tons of coal and rock as well as 1.5 million m³ of gas occurred. In this area, there existed a thrust fault, the coal seam was under strong compressive ground stress. It made the coal seam to become thicker and softer and splitted. The seam inclination increased from 32° to 55°, and its thickness from 2.5 m to 5 m.

The first outburst took place at the footwall of the fault. After the crosscut has cut through the fault plane into the hanging wall of the fault 450 tons of coal and 25,000 m³ of gas were ejected from boreholes when hydroboring was employed as a preventive measure. Under the protection of the hydro-boreholes, the crosscut was driven safely for 27 m in the coal seam. At this time the roof of the coal seam was exposed at the top of the crosscut. However, the second strong outburst with 2,807 tons of coal and rock ejected occurred as soon as the hydro-boreholes was completed. It was reasoned that there existed not only a large ground stress but also a strong abutment pressure from the first outburst (Figure 1).

After mining, the ground stress redistributes. The abutment stress is 2-3 times as high as the virgin stress in front of the working face. When the abutment stresses from two mining operations superimpose the increment of abutment stress will reach 4-5 times of the virgin stress which is very dangerous. For instance, on January 20, 1968, a strong outburst with 5,000 tons coal in Nantong No. 1 colliery occurred in a heading being driven into the abutment of the upper protective seam (Figure 2).

Therefore the ground stress is the key to outburst initiation.

C. Gas Pressure and Gas Content

The potential energy of the gas contained in the coal seams is the main energy source for outbursts. It acts to crush, eject, and transport coal during the process of outbursts. As observed in practice, the gas pressure in the

outburst-prone coal seams is always larger than 1 MPa (150 psi). In China, the highest gas pressure measured in Beipiao Mining Area is 8.26 MPa (1239 psi). Under this condition the gas content of the coal seams is about 15-30 m³/t.

Under the same depth, the gas pressure is approximately uniform in a small area. Up to now, we haven't found any place where the gas pressure is abnormally high. However, the relative gas outflow (m³/t) of the outbursts is much larger than the gas content of the coal seams. According to some statistics, the average relative gas outflow of the outburst is 200 m³/t. The largest is about 700 m³/t. This illustrates that the amount of gas outflowed is not only from the ejected coal in the outburst but also from the coal seam around the outburst cavity where the permeability of coal is increased greatly due to destress. Sometimes, the width of the destress zone around the outburst cavity is larger than 10 m.

The measurements of the velocity of gas desorption from coal particles (0.2-0.3 mm) show that the percentages of gas emission from coal particles in the first minute are 2-21%. As the severity of coal crushing increases, the velocity of gas desorption increases, as shown in Figure 3 (Wang, 1980).

As observed in practice, a strong outburst always consists of several outbursts. Each outburst may last from several to tens of seconds. There will be a relative quiet period between two consecutive outbursts. This has been proven by the statistics. This process may repeat several times until the ground stress and gas pressure are insufficient to initiate an outburst. The total processes may last more than 10 minutes. The gas energy of the first outburst comes from the free gas in the coal seam. Whether the subsequent outbursts can be initiated or not depends on the ability of gas desorption.

Based on the theoretical analysis, gas energy is the main energy source for outburst. For example, on December 4, 1977, when the K₁₀ coal seam in Zhong Liang Shan colliery was excavated by blasting, an outburst with 817 tons of coal and 38,500 m³ of gas occurred. The characteristics of the outburst-prone coal seams were: thickness, 2 m; seam inclination, 60°; specific gravity of coal, 1.35 t/m³; depth, 320 m; gas pressure, 1.95 MPa; duration of the outburst, 39 seconds.

A brief knowledge about the energy sources of the outburst can be obtained by the following simple calculations.

1. Elastic energy of coal

$$W_t = \frac{2.94\sigma^2}{2E} (1-2\nu) \quad \text{MJ/m}^3$$

where σ -- gravitative stress of rock, 8 MPa,
 E -- Young's modulus of coal, 500 MPa,
 ν -- Poisson's ratio, 0.3

$$W_t = 0.0753 \text{ MJ/m}^3$$

The elastic energy released from the roof and floor is $W_2 = 10.98 \text{ MJ}$

3. Gas energy

Since the duration of the outburst was only 39 seconds, the physical process of gas expansion can be considered as adiabatic. Under this condition one may consider that only the free gas was involved in the outburst. The free gas content was $1.63 \text{ m}^3/\text{t}$.

$$W_3 = \frac{0.098P_1V_1}{n-1} \left(\frac{P_0}{P_1} \right)^{\frac{n-1}{n}} - 1 \quad \text{MJ}$$

where P_0 -- gas pressure in coal seam, 1.95 MPa
 P_1 -- gas pressure in the entry, 0.1 MPa
 n -- adiabatic constant, 1.31
 V_1 -- volume of free gas, 1336 m^3

The gas energy is therefore $W_3 = 422.35 \text{ MJ}$

The gas energy is about 88% of all energy for this outburst. According to the analysis, gas energy is the main energy source for outburst, while the elastic energy of coal, roof and floor is the main one for crushing coal seam.

D. Properties of Coal Seams

Among the 54 strong outbursts, three occurred in the coal seams with thickness of less than 1 m; 8 in 1-2 m; and 43 in seams thicker than 2 m; 15 occurred in the coal seams with inclination less than 30° ; 19 in $30-45^\circ$; 20 in larger than 45° .

The strength of coal is the resisting force to outburst. It is an important factor. In China, coal has been divided into five categories according to their strength. Table 1 shows some properties of coal measured in BeiPiao Mining area.

As the severity of crushing increases, the coal strength decreases. As a result the coal is easy to break. All strong outbursts occurred in the fourth and fifth categories of coal. Up to now, the outbursts have not occurred in the first and second categories.

The third category of coal appears as broken pieces. Both the fourth and fifth categories look like soil. They are the most dangerous coal once saturated with gas.

Generally speaking the permeability of the outburst-prone coal seams is very low, i.e. less than 0.05 mili Darcy. In some notorious outburst coal seams, it is less than 0.005 mili Darcy. Thus the gas pressure gradient in front of the coal face is sufficiently high to initiate outbursts during development.

Under high ground stress in particular the outburst-prone coal seams are highly mobile. Therefore after the outburst, the volume of outburst cavity is always much smaller than that

of the coal and rock ejected from the outburst. Sometimes the outburst cavity is so small that it could not be found after outburst.

If hydroboring is applied in the outburst-prone coal seams, the outburst cavity is always smaller and filled with broken pieces of coal (Figure 4).

The flowability of coal is very important in the initiation and development of outbursts. It increases greatly the magnitude of outburst and the quantity of gas emission. Therefore, under the same condition the intensity of outburst increases with the thickness of the coal seams.

E. Type of Drift and Method of Development

Among the strong outbursts, two occurred in the raises, 11 in the drift headings, and 41 at the crosscuts. 76% of the strong outbursts occurred at the crosscuts when intersecting or cutting through the outburst-prone coal seams.

When intersecting the coal from the crosscut, the ground stress and gas pressure undergo a great change. This is conducive to an outburst. In addition, since the cross-sectional area of the crosscut is generally larger, it is very convenient for transporting coal by the gas storm. Under such conditions it is difficult to form a barricade of coal pile in the crosscut for outburst prevention.

During the process of cutting through the outburst-prone coal seam, the gas concentration in the mine air flow is always smaller, because the permeability of the coal seam is very low. But large ground stress and high gas pressure still exist in the surrounding strata of the crosscut. This is a dangerous condition for crosscut development. Thus in China, preventive measures are required for development work between the areas from 2 m before and 2 m after intersecting the coal seam.

Among those outbursts, 51 or 94.4% were caused by shock blasting. This demonstrates that the effects of blast vibration play a commanding role in inducing the outbursts. The maximum amount of explosive that induced the outbursts was 231 kg, while the smallest was only 0.6 kg.

In the dangerous areas, if the coal seam is not discharged to form a distressed zone within 5-8 m surrounding the crosscut, a strong outburst will occur easily. It requires a period of time for the ground stress to crush the coal. Generally speaking, outbursts occur several to tens of seconds, or even tens of minutes, behind coal crushing. Sometimes, if the ground stress is very large, the gas pressure is high, and coal strength is weak, the outburst can occur in normal operations without blasting.

CONCLUSIONS

The problems of outbursts are serious in China. In 1958-1980, the strong outburst with more than 1,000 tons of coal and rock ejected occurred 54

Table 1 Coal Properties in BeiPiao Mining Area

Parameters	Units	Category				
		First	Second	Third	Fourth	Fifth
Cohesion	MPa	2.48	1.73	1.05	0.73	
Angle of friction	degree	38.8	37.5	33.3		
Coefficient of coal strength, f		0.69-2.20	0.25-1.33	0.13-0.52	<0.1-0.33	<0.1
Porosity, K	cm ³ /g	0.0121	0.0131	0.0216	0.0314	
Desorption constant, B	%	3.8	5.3	4.8	10.6	24.6
AP		0.5 - 2.8	0.5 - 8	1 - 19.3	3.8-21.7	16.7-22.1

times. They displayed the following characteristics.

1. The strong outbursts always occurred in the structurally disturbed areas and the abutment zones. The ground stress was the key to outbursts.
2. In strong outburst-prone coal seams, the gas pressure was larger than 1 MPa.
3. There are 5 categories of coal. The strong outburst occurred in the fourth and fifth categories.
4. The flowability of coal plays an important role in outburst. The absorption and mechanical properties of coal were the bases of outburst initiation.
5. Most strong outbursts occurred at the crosscuts. Blasting has a strong effect on inducing outbursts.

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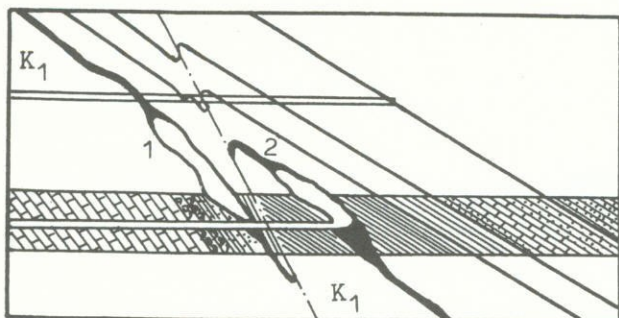


Figure 1. Locations of two outbursts in Sanitui No. 1 Colliery,
1 - Aug. 8, 1975 2 - May 7, 1976

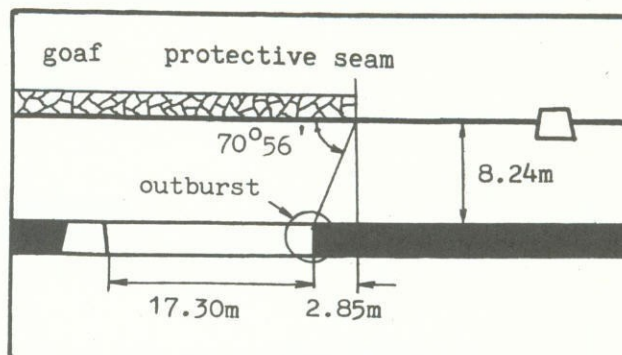


Figure 2. Schematics showing the location of outburst in Nantong No. 1 Colliery on Jan. 20, 1968.

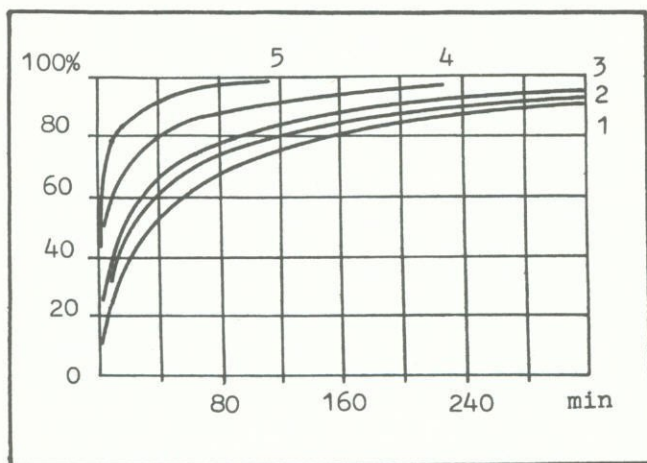


Figure 3. Velocity of gas desorption of Beipiano Coal. Partical sizes 0.2-0.3 mm; virgin gas pressures, 0.5-1 MPa.

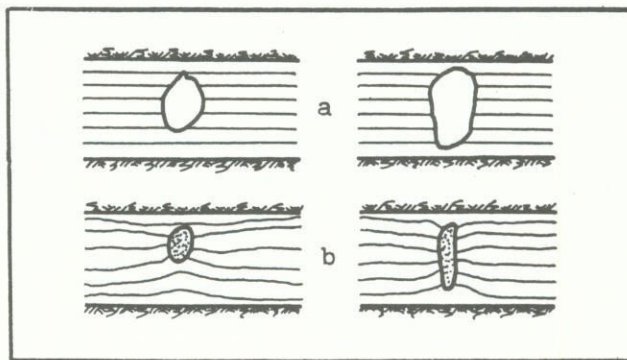


Figure 4. Flowability of coal.
a. immediately after hydroboring
b. long after hydroboring

ROCK BURST OCCURRENCES IN THE COEUR D'ALENE MINING DISTRICT OF NORTHERN IDAHO

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ABSTRACT

Mining induced rock bursts have been a severe operational hazard associated with the deep silver mines of the Coeur d'Alene mining district for over 30 years. Recent studies during the past few years have led to a better understanding of the different types of rock bursts and their causative mechanisms. This has resulted in ground control measures that more effectively minimize burst occurrences as well as contain burst damage. Case histories of burst incidences and examples of control measures are presented.

REVIEW OF ROCK BURSTING

Most of the hardrock mining districts throughout the world that experience rock bursts have many similarities. They are in an initially high stress environment owing to depth, tectonic forces, or a combination of both. The shape of the orebody is tabular, and the thickness of the vein or the width of the seam is usually < 5 m. The direction of the major principal stress is near perpendicular to the tabular dimension. The extraction ratio is high, usually $> 80\%$. The wall rocks, and often the vein or seam material, are very hard, brittle, and strong as well as being very old geologically--mostly of Pre-Cambrian age ($> 1 \times 10^9$ yr). In addition, the geology is usually very complex with respect to folding, faulting, and metamorphism.

This combination of high stresses, narrow veins, high extraction ratios, and strong, brittle rocks in a complex geologic environment is almost always certain to lead to the occurrence of rock bursting in deep mines. This same combination of characteristics with relatively weak rocks results in coal bumps or bounces in coal mines at moderate depths.

Definition of a Rock Burst

Definitions to characterize the occurrence of a rock burst have been proposed by workers in the field as well as by governmental agencies. A number of such definitions are presented. It should be noted that these definitions do not

identify the conditions or causes for the occurrence of a rock burst.

"That phenomena which occurs when a volume of rock is strained beyond the elastic limit, and the accompanying failure is of a nature that accumulated energy is released instantaneously" (USBM, 1968).

"An instantaneous failure of rock causing an expulsion of material at the surface of an opening or a seismic disturbance to a surface or underground mine" (Ontario Ministry of Labor, 1983).

"A sudden and violent failure of a large volume of overstressed rock, resulting in the instantaneous release of large amounts of accumulated energy" (MSHA, 1984).

As might be expected, these definitions include everything from the popping off from the face of a few small pieces of rock to large seismic events, a local Richter earthquake magnitude (M_L) ≥ 4.0 , that may occur near an underground operation and do no observable damage. The South Africans now tend to classify an event a rock burst only if it does measurable damage to the mine, usually $M_L \geq 0.5$. They refer to all occurrences that release kinetic energy as seismic events, from splitting and popping to very large mining-induced seismic events. On the other hand, civil engineers (particularly Norwegians) refer to the small-scale seismic disturbances encountered in tunnelling as rock bursts, even though there is no measurable damage and the M_L is $\ll 0.5$.

The Rock Burst Phenomena--Large Seismic Events

Underground and close to the source, rock bursts are heard as loud, distinct blastlike noises. From a farther distance, rock bursts sound like deep, dull thudlike noises. When a large volume of rock has been dislodged by a burst, there is generally a shock wave of air that is felt in nearby interconnected openings. For this reason,

most underground miners refer to rock bursts as "air blasts." On the surface, rock bursts sound something like a sonic boom, but the ground and surface structures shake similar to what would be felt from a small, nearby earthquake.

Underground damage from rock bursts is basically from two different mechanisms: the effect of the burst itself and the effect of the resulting seismic wave. Rock bursts occurring in overstressed structures, particularly pillars, result in the breaking up of the volume of rock involved into small fragments and various degrees of damage to the walls and supports in adjacent openings. Damage from the action of the passing seismic wave results in the slabbing off of large pieces of rock from the walls and backs of openings. Supports in openings, particularly timber or steel, can be severely damaged by the effects of the passage of the seismic wave. This type of damage can be very widespread or limited depending on the magnitude of the rock burst.

The seismic wave generated by a rock burst can be detected on surface seismographs over large distances. Small- to medium-sized bursts usually can be detected within a 100-km radius, while large bursts have been detected 1,000 km away.

Small Seismic Phenomena

Popping, spitting, and sometimes spalling are stress-induced surface rock failures that also result in generating seismic waves. The volume of rock involved in such small-scale failures is usually $< 0.5 \text{ m}^3$, and no air blast or shock wave is created. The seismic and audible signals generated generally cannot be detected or heard at distances much beyond 50 m from the source.

"Bumps" or "knocks" are terms applied to noises heard in the mine but generally are neither heard nor felt on the surface. If they occur very close to an opening, they can displace a volume of rock in the 1 to 2 m^3 range.

Types of Rock Bursts

Many terms have been used to describe the wide range of effects associated with rock bursts. The two most commonly used terms are "strain bursts" and "crush bursts" (Witwatersrand Rockburst Committee, 1924). Strain bursts refer to the small and localized bursts that can occur around any excavation, whereas crush bursts are generally large and associated only with areas of extensive mining.

These terms suggest different mechanisms that release seismic energy during a rock burst. The three most commonly considered mechanisms are stored strain energy, change in potential energy, and slippage along faults. To get an understanding of these mechanisms, it is instructive to look at how they relate to mined openings.

Development Openings

Single openings in rock, e.g., shafts, tunnels, or drifts, are surrounded by a small volume of highly stressed rock which results in an accumulation of stored strain energy at or near the face. The popping and spitting that is commonly encountered in very brittle rock is a result of the instantaneous release of small amounts of stored strain energy. The rock ahead of the advancing face or heading is not able to adjust to the rapid stress increase, and therefore, these small-scale failures occur, usually along geologic structures such as joints.

When single openings intersect geologic structures, e.g., faults, dikes, or hard layers, a larger volume of highly stressed rock may result. Hence, a larger amount of energy may be released. Bumps and small rock bursts can occur. In these cases, the fracturing of a larger volume of rock results in releasing potential energy from the stress field as well as stored strain energy.

At Hecla's Star Mine near Burke, Idaho, two bursts in the range of $0.5 \leq M_L \leq 1.0$ occurred during the driving of new level development on the 7700 level. This level is ~2,350 m below the surface and was 60 m below the deepest mining level. The wall rocks are a series of thin- and thick-bedded quartzites with argillaceous interbeds. The compressive strength of the quartzite beds is in the 140 to 180 MPa range. The in-situ horizontal stress parallel to the beddings on the 7700 level was found to be ~80 MPa, and the horizontal stress perpendicular to the beds was ~60 MPa.

The first burst occurred when the drift intersected a lamprophyre dike, and the second burst occurred when the drift subsequently intersected a stronger and more massive quartzite layer. A plan view of the 7700 level showing the geology and the location of the bursting is shown in Figure 1. Rocks of ~2 to 3 m^3 were displaced from the back and face from each burst. Both bursts occurred during blasting. Destressing ahead of the face was used until the drift had passed through these burst-prone zones.

Mining Openings

Multiple openings from mining create highly stressed ground between and around these openings, which can involve a large volume of rock. The most classic type of rock bursting is pillar bursting associated with narrow vein or seam mining. When the pillar fails, a large amount of stored strain energy is released. In addition, the failure of the pillar results in instantaneous closure of the walls or roof and floor. Such small displacements over a very large area cause an additional large amount of potential energy to be converted to kinetic energy and released. This potential energy can be gravitational or tectonic. Pillar bursting is most common when the extraction ratio is $\geq 80\%$, i.e., a small pillar area is left supporting a large mined-out area. Pillar bursts are generally in the range of $1.0 \leq M_L \leq 3.5$ and

and present a serious hazard to underground personnel.

As an excavation progresses, the stresses change and the rock deforms around the mined opening. For any type of seismic phenomena to occur, including rock bursting, the stress changes due to excavation must result in an unstable equilibrium of the rock mass. It should be pointed out that rock bursts or seismic activities are a consequence of an active excavation process. When the excavation is halted or completed, the seismic activity ceases.

Rock bursting in the Coeur d'Alene mining district of northern Idaho is mainly located in pillars that result when mined openings converge. Typical burst-prone geometries depict the problem at Hecla's Star and Lucky Friday mines (Langstaff, 1980) as shown in Figure 2.

On the 7700 level of the Star Mine, ~2,350 m below the surface, converging first floors burst when the small pillar between drifts was reduced to 7 m. The burst ($M_L = 1.0$) fractured the pillar and displaced ~100 ton of rock off the backs and walls of the stopes. The burst came with blasting. Similarly, when the stope was mined up to within ~15 m of the 7500 level, a sill pillar burst occurred with magnitude $M_L = 2.6$. This burst displaced ~500 ton of rock in the stope and in the crosscut on the level above. Destress drilling from the crosscut above was being carried out at the time of the burst.

The compressive strength of Star Mine quartzite rock is in the 140- to 180-MPa range. The in-situ horizontal stress parallel to the bedding on the 7700 level is ~80 MPa, and the horizontal stress perpendicular to bedding is ~60 MPa. In achieving nearly 100% extraction, it is clear that sill pillars will fail as the pillar size is reduced. If the pillar does not break up gradually through self-destressing, it will fail violently by bursting. Pillar destressing is designed to allow the pillar to fail gradually.

Extensive Mined Out Areas

As mined out areas become very extensive, > 1 km², stress transfer and changes due to mining can cause displacements on nearby to distant fault structures. The energy release associated with slippage along faults can be very large, M_L to > 5.0 , resulting in damage to surface structures but often does little or no damage underground. Such failures can occur around soft as well as hard rock mines and have also been induced around surface mines and quarries.

Most mining-induced seismic events are caused by slippage along preexisting faults. The disruption of the regional stress field by an extensive mined out area either increases the driving force along the fault or decreases the resisting force normal to the fault. The net result is the same, a sudden displacement along the fault releasing large amounts of seismic energy.

Fault-slip seismic events have become the predominant type of rock burst accompanying mining on the lower levels of Hecla's Lucky Friday Mine. Thin argillaceous interbeds between quartzite layers are favorably oriented for fault-slip movement, as depicted in Figure 3. Damages from large bursts of this type is primarily a result of seismic wave reflection off of stope and access opening walls. Where the fault intersects a cross-cut above, it is not uncommon to have track heave of 30 cm.

DEALING WITH THE OCCURRENCE OF ROCK BURSTING

While there is no theoretical or practical way to eliminate rock bursting, measures have been developed over the years that have been found to minimize both the occurrence and effects of bursting.

Geometric Considerations

The most common means of minimizing high stresses around mining openings is by using geometries and excavation sequences that do not create large burst prone pillars and that transfer stress to the abutments in a manner that minimizes high stress concentrations. In practice, this is achieved by utilizing longwall-type sequences. In addition to minimizing stress concentrations, these stair-stepped geometries also reduce exposure, since workers are spread out in a number of small stopes rather than being concentrated in long flat-backed overhand or underhand stopes. Any pillars left must be large enough so that they do not become burst prone, and should be oriented so that stope span is reduced, as well as stope closure. Such a pillar is called a stabilizing pillar, and if left systematically throughout an orebody will greatly reduce the occurrence of rock bursts.

Geologic Considerations

As mining experience in an orebody or district is gained, relationships between rock bursting and certain geologic features become evident. Dikes, faults, specific stratigraphic horizons, or a geologic unit are examples of geologic features that result in high stress concentrations and are very rock burst prone when intersected by either mining or development openings. Service or access openings should not intersect or be placed in such features unless necessary. Mining through such features should be carried out so that stress increases are minimized.

Extraction Ratio

As a face is advanced in highly stressed ground, a fracture zone is developed ahead of this face. While most of this fracturing occurs within a few hours after the blasting of the face, the fracture zone boundary continues to migrate slowly for ~24 h. The depth of this fracture zone depends on the face height or width and on the abutment stress. This fracture zone can be

thought of as a self-distressed zone between the opening and the solid intact rock.

Too rapid a face advance does not allow time for this fracture zone to occur gradually and reach stress equilibrium. In order to restore equilibrium, the development of this fracture zone may take place in an unstable manner in one large and rapid step--a rock burst.

In most mines, stope faces prone to rock bursting are advanced on a single shift basis. The face advance is kept as small as possible, generally not more than the stope height or width. This is to allow the wallrock time to adjust and reach a self-distressed equilibrium state. It should be pointed out that most rock bursts occur during or immediately following mine blasting due to the almost instantaneous stress transfer and loading of the rock around the new face or heading.

Detection of Rock Bursting

Most mines experiencing rock bursting have installed some type of seismic or microseismic monitoring system. Very large mines tend to do seismic monitoring, while smaller mines tend to do microseismic monitoring.

Seismic monitoring on a mine-wide basis detects only large seismic events and rock bursts. The rapid location of rock bursts and an estimate of the damage are vital in initiating rescue operations. Data from seismic monitoring are also used to assess the relative seismicity and potential rock burst hazard presented by various geologic discontinuities in different parts of a mine. This information is used to replan or reschedule mine layouts to minimize the occurrence of bursting (Spottiswoode, 1984).

Microseismic monitoring detects very small-scale seismic events as well as the large seismic events and rock bursts. The data are used to delineate high stress buildup or potential rock burst zones prior to the occurrence of bursting. Knowledge of a particular problem area and its causes allows control measures to be taken to minimize both the occurrence and effects of a rock burst (Blake, 1982a). Recognition of either burst-prone geologic features or mining geometries allows mining plans to be modified or changed prior to the occurrence of bursting.

Rock Burst Prediction

Rock burst prediction was the goal of microseismic monitoring when this technique was first developed by the U.S. Bureau of Mines (Obert, 1941). While the microseismic method has been successfully used to detect and delineate burst prone mine structures, use of microseismic data to determine if a structure is going to burst and when it will burst has not been possible except in rare instances.

The low accuracy and reliability of the microseismic method with respect to prediction

should not be surprising. Earthquake prediction has a very similar record. Because of changing stress, mining geometry, and local geologic conditions, the occurrence of every burst is almost unique. For this reason, readily recognizable microseismic and seismic precursor patterns do not precede the majority of rock bursts (Blake, 1984).

Preventing Rock Bursts

In mining, it is not possible to eliminate the occurrence of all rock bursts. For some special geometric relationships, however, it has been possible to avoid the occurrence of a burst by destressing.

The idea of destressing is to blast fracture the rock in a high stress or potential rock burst zone to reduce the stress. In addition to reducing the stress in a particular mine structure, destress blasting also results in modifying the material properties of the structure so that its mode of failure is changed from brittle elastic to more plastic behavior. The destress blasting reduces the stress and dissipates stored strain energy by inducing fractures that allow yielding (Blake, 1982b).

The most successful application of destressing has been with respect to development headings, i.e., tunnels, drifts, and shafts. The usual procedure is to drill and blast at least two parallel holes loaded halfway ahead of the face. Popping, bumping, and small bursts ahead of the face have been controlled by this type of destressing.

Destressing of burst-prone pillars has also been successfully carried out. The pillar is line-drilled, and the holes are blasted to fracture it and allow it to yield.

Destressing does result in stress transfer to adjacent solid rock. Hence, the timing and sequencing of any destressing must be done with care. The transfer of stress from destressing can cause a similar or worse problem in adjacent or nearby highly stressed structures.

Reinforcement to Contain the Effects of Bursting

Traditionally reinforced backs and walls of openings will not hold up under the effects of a nearby rock burst. Such reinforcement is designed to support dead loads and not shock loads as is the case for a burst. Rigid support or reinforcement is not able to yield and, hence, fails or deforms almost instantaneously, allowing the back and/or walls to come in and fill the opening.

Very often there is nothing that can be done to eliminate the occurrence of all rock bursts, but steps can be taken to minimize the damage resulting from bursting. The use of fully grouted rebar- or threadbar-type rock reinforcement and the use of wire mesh will contain the effects of small nearby seismic phenomena as well as large distant rock bursts. The use of fiber-reinforced shotcrete has also been found to prevent and contain the effects of popping and spalling. For

DISCUSSION

An overall strategy for dealing with rock bursts in the deep mines of the Coeur d'Alenes has evolved over the years. The initial step was modify stope geometry and mining sequences to avoid long burst-prone sill pillars. This was followed by utilizing microseismic monitoring to identify burst-prone structures, and destress blasting to cause yielding rather than allowing violent failure. Recognizing that bursting cannot be eliminated has led to better rock reinforcement practices to contain the effects of a burst. Recent efforts have consisted of developing underhand mining techniques to recover sill pillars as well as establishing a primary underhand longwall mining geometry. Consideration is now being given to leaving permanent stabilizing pillars in low grade sections since this is the only proven practice that significantly reduces the overall rock burst rate.

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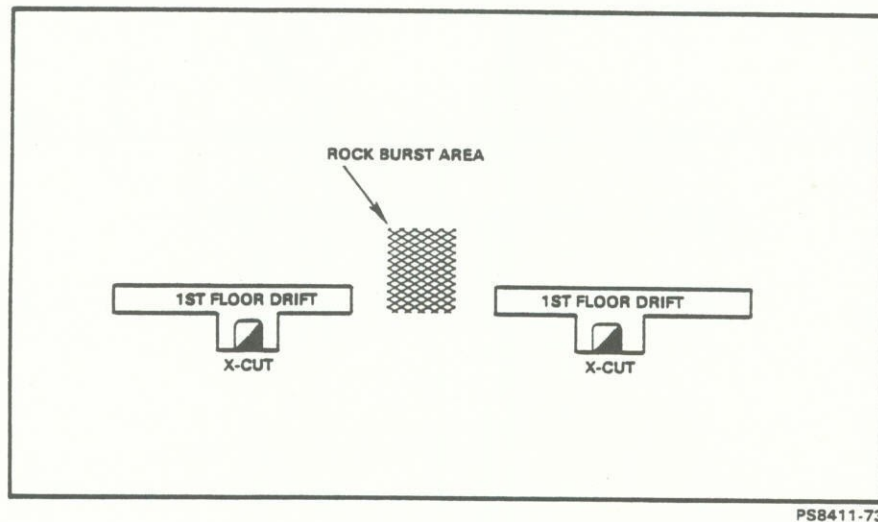
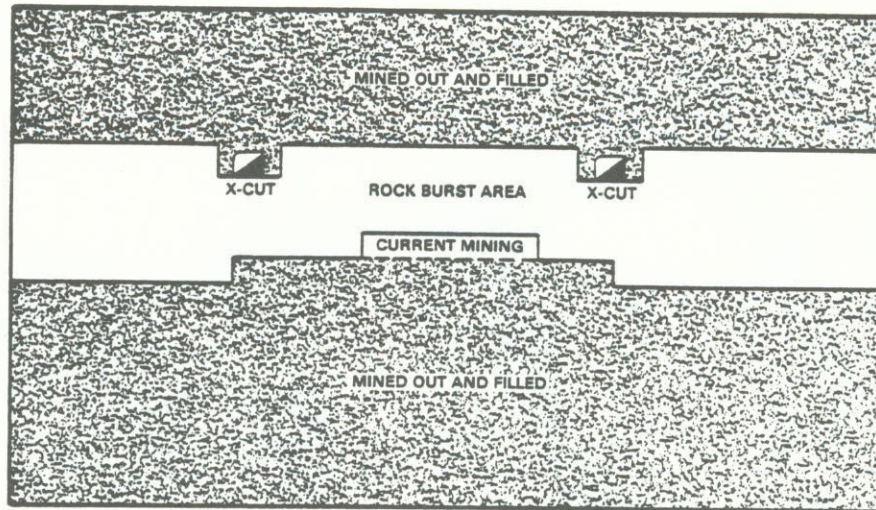


Figure 2. Burst Prone Geometries at Star Mine (Langstaff 1980).

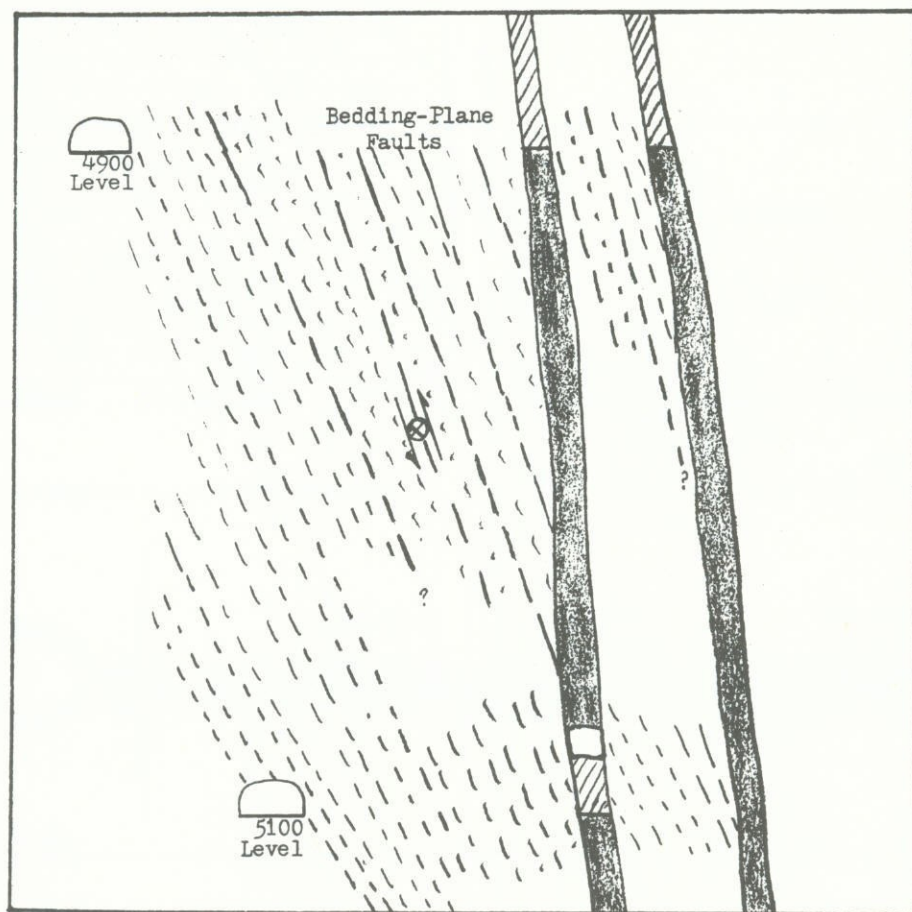


Figure 3. Fault-Slip Rock Burst at Lucky Friday Mine.

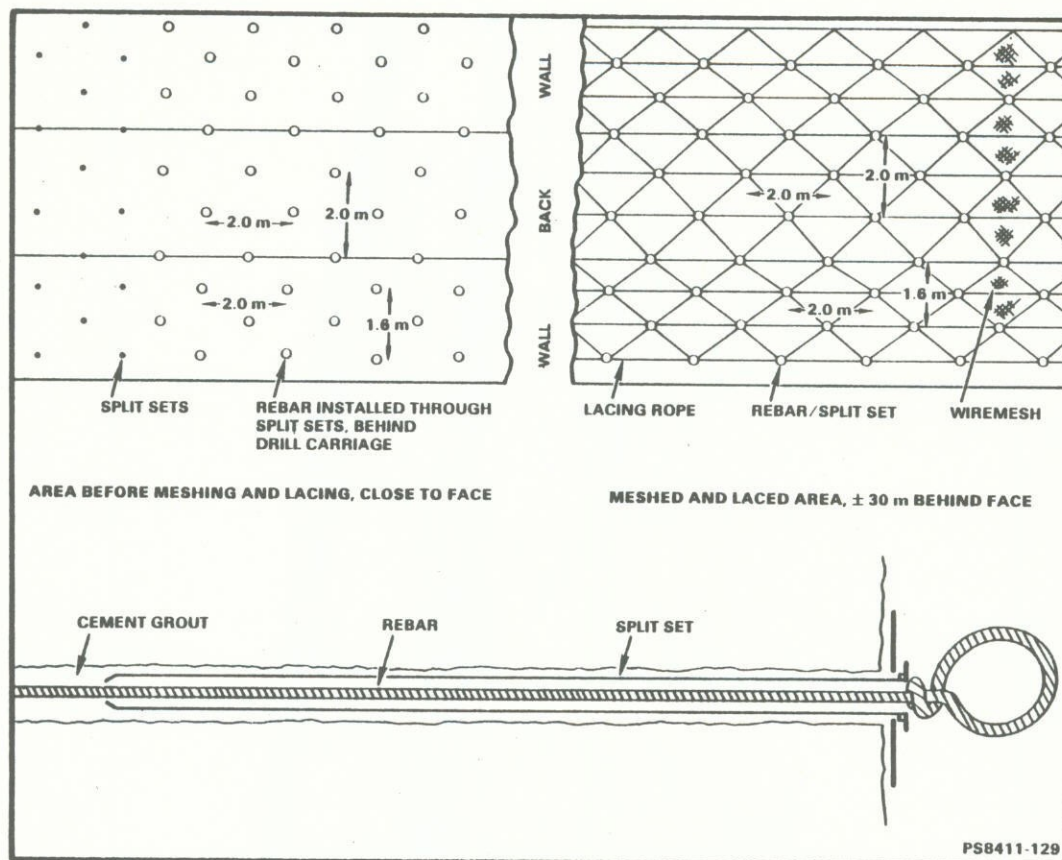


Figure 4. Reinforcement to Contain the Effects of Rock Bursting (Jacobs and Van der Merve 1981).

A HISTORY OF ROCK BURST RESEARCH IN THE COEUR D'ALENE MINING DISTRICT

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ABSTRACT

This paper reviews the history of rock burst research in the Coeur d'Alene Mining District of northern Idaho over the last 30 years. Rock bursting is a problem because of a combination of geological factors in the region--complex structure, high horizontal stresses, and strong, brittle rock that readily stores strain energy. The Bureau of Mines has cooperated with industry to study rock burst phenomenon for many years. Early work focused on methods to mitigate bursting problems within the context of existing mining methods. In the 1970's, researchers began investigating methods for altering or controlling conditions that lead to rock bursts. These control measures become part of the stope development that precedes mining or are changes in the mining process itself. Although, researchers continue to develop microseismic techniques, the major research effort focuses on developing a new mining method to reduce the number and severity of rock bursts in order to improve safety and increase productivity in the Coeur d'Alene Mining District.

INTRODUCTION

Rock bursting has been a problem in the Coeur d'Alene Mining District for nearly 50 years. The Bureau of Mines has cooperated with industry to study rock burst phenomenon for much of that time. Early work focused on microseismic techniques and methods to mitigate bursting problems within the context of existing mining practices. While research today continues to develop microseismic techniques that will reliably predict rock bursts, the major effort focuses on developing new mining methods that will reduce the number and severity of rock bursts in production stopes.

Minerals were discovered in the Coeur d'Alene Mountains just over 100 years ago, and since then, the area has been mined almost continuously. Although the

first claims filed in the area in 1879 identified galena deposits along the South Fork of the Coeur d'Alene River (1), mining did not begin in earnest until placer gold was discovered in Prichard Creek in 1883. A gold rush followed, and thousands of people came to the area searching for wealth. The colorful history of this mining district has been chronicled in several books describing intrigues typical of the early mining days in the Old West (2, 3). One such story tells of the discovery of the Bunker Hill lode by a jackass. The owner of the jackass, a prospector named Noah S. Kellogg, awoke one morning to find that his pack animal had wandered off. His three-day search for the donkey led him to a far mountain side where he found the animal standing on the outcrop of what became one of the most productive deposits in the district. The Bunker Hill Mine has produced over 35 million tons of lead-zinc-silver ore over its 95-year history (4).

The Bunker Hill and other major producing mines in the district--the Sunshine, the Lucky Friday, the Galena, the Star-Morning, the Coeur, and others--have produced metals valued at over \$4 billion, making the Coeur d'Alene Mining District one of the richest in the world (5).

After discovery of the rich silver-bearing veins, mining in the district rapidly progressed from open pit "glory holes" to underground. The geology of the region presents a serious challenge to the underground miner. It is very complex as a result of several episodes of folding and faulting. The high tectonic stresses responsible for folds and faults have left the region with a nearly horizontal principle stress (6). The veins of ore are generally located in bands along fractures and fault zones and appear primarily in quartzite host rocks (7).

All of these factors result in mining conditions which promote rock bursts even at fairly shallow depths. The first rock burst fatalities in the district were recorded in 1941 on the 2900 level of the Sunshine Mine (8). Since then, all the mines in the district have experienced rock bursts, some more than others, with the frequency and severity increasing with mining depth.

The Coeur d'Alene district has some of the deepest mines in North America, a few of which reach depths of 8,000 ft. High stresses associated with mining deeper ore bodies are a prime factor in the recent increase in rock bursting in the Coeur d'Alene Mining District and, indeed, throughout the world. However, high stresses alone will not produce a rock burst. The geologic structure or the rock fabric must be capable of storing strain energy until it is released in the form of a rock burst. Strong, brittle rock which tends to store strain energy to its rupture point or faults which store large amounts of strain energy before slipping will cause bursts in high-stress environments. Unfortunately, all of these conditions--high stresses, stick/slip faults, and strong, brittle rock--are prevalent in the Coeur d'Alene region. The quartzite host rock, the high horizontal stress, and a complex structure, in combination with present economic factors, have caused the closure of all but three of the mines.

Research into rock burst phenomena has had a long history in the district, and the purpose of this paper is to review that history and research trends up to the present day. It should be obvious that the problem has not been solved, but there have been successes. We will point out the successes and failures, but more importantly, we will try to show the evolution of the research and what direction it is likely to take in the future. We believe that the rock burst problem can be solved by a thorough understanding of the phenomenon and its controlling mechanisms. This is not to say that all rock bursts can be eliminated, but by understanding the nature of bursting, we can control the mining process to eliminate the devastating results that too often accompany them.

ROCK BURST PREDICTION

In 1938, the Bureau began research to determine if there were a relationship between the velocity of seismic waves through a rock mass and the state of stress in that rock mass. While experimenting with sensitive seismic equipment, it was discovered that rock under stress emits sub-audible or microseismic signals in

addition to the audible sounds usually associated with breaking rock. The generation of these microseismic signals in stressed rock inferred a relationship between the time rate of the signal and the state of stress in the rock (9). This relationship was soon verified during a seismic investigation of pillars in a deep copper mine in northern Michigan. In the course of this investigation, the ground being monitored became seismically active for about 15 minutes before a rock burst occurred. This unexpected event led to a hypothesis that microseismic activity might be used to predict or warn of an imminent rock burst (10). Thus began the development of the microseismic method as a rock burst prediction tool.

Although microseismic techniques were first mentioned as a method of studying rock bursts in the Coeur d'Alene district soon after the first fatalities in 1941, it was not until the mid-1960's, when methods for seismic source location became available, that microseismic techniques were actually applied. Since that time, research into microseismic methods has resulted in major advances in equipment and signal processing (11). Several types of microseismic systems have been developed to monitor rock bursts in the district. Mine-wide systems monitor the source locations and magnitudes of events; these systems generally utilize 20 to 30 single-axis geophones to triangulate source locations and a triaxial sensor for magnitude determination. Concentrated networks of single-axis geophones surrounding individual stopes have also been used to monitor and locate events of very small magnitude.

Twenty years of research has not developed a microseismic technique that can be used to predict rock bursts. There have been successes in predicting bursts just prior to their occurrence in at least two mines in the district, but the techniques have not proven to be reliable. Although a rapid increase in the number of microseismic events at a location sometimes precedes a burst, at other times an identical pattern will provide a false alarm, while in other instances bursts occur without any warning at all. Microseismic monitoring systems that report location, frequency, and magnitude alone do not appear to be adequate to predict rock bursts reliably.

The failure of present systems led researchers back to the laboratory to search for a precursor which may be hidden in the acoustic waveform of the microseismic signals. Waveform attributes under examination include rise time, peak amplitude, decay time, duration, and total energy. This work has been made possible

by advances in digital processing of acoustic signals. New processing equipment is capable of storing individual waveforms of microseismic events at rates of 500 samples per second for a 16-channel system (12). Other fields of measurement, such as x-ray spectroscopy, electromagnetic radiation, and high-frequency acoustics, are also being investigated in the search for a reliable precursor. If a precursor can be identified in the laboratory, field data will be gathered and studied to see if it can be translated for practical application in mines. The search to discover a reliable precursor to catastrophic rock failure is an important part of the Bureau's rock burst research, but several other areas that fall under the general category of control measures have been investigated as well.

CONTROL TECHNIQUES

Several control techniques for reducing rock bursts in underground metal mines have been investigated in the Coeur d'Alene Mining District. Each of these control measures attempts to alter the conditions that, as we currently understand them, combine to produce a rock burst. Definitions used to describe a rock burst vary, but their common features include the sudden, violent failure of overstressed rock near mine openings, and an instantaneous release of strain energy (13-16).

Control measures are aimed at preventing a build-up of strain energy to dangerous levels. There are two basic ways to accomplish this: 1) change the physical properties of the rock so that it cannot store strain energy, and 2) mine in ways that prevent high levels of stress from being concentrated at critical points. The goal of all control techniques is a gradual release of the strain energy produced as a result of mining--a release that allows the rock to yield harmlessly over a period of time.

Initially, all control measures were applied within the context of standard mining practices. The standard mining method used for many years in the Coeur d'Alene district was (and is) cut-and-fill stoping. This is a method typically used to excavate steep, narrow, high-grade vein deposits in strong, competent wall rock. Access drifts are developed from a shaft on levels, usually 200 ft apart vertically, and crosscuts are driven from the drifts to the vein at regular intervals. The resulting stope dimensions are generally 200 ft high by about 300 ft wide and these blocks of ore are mined upward from the lower drift in 10-ft-high lifts. Thus, a stope mined from the 4900 level would be mined upward to the

4700 level. As each lift of the stope is completed, tailings from the mill, referred to as "sandfill," is placed in the mined-out stope to form the floor of the next lift. Figure 1 is a cross section of a cut-and-fill stope showing the drifts on the development levels, the nearly vertical vein, crosscuts to the vein, and the mined-out, filled area below the active stope.

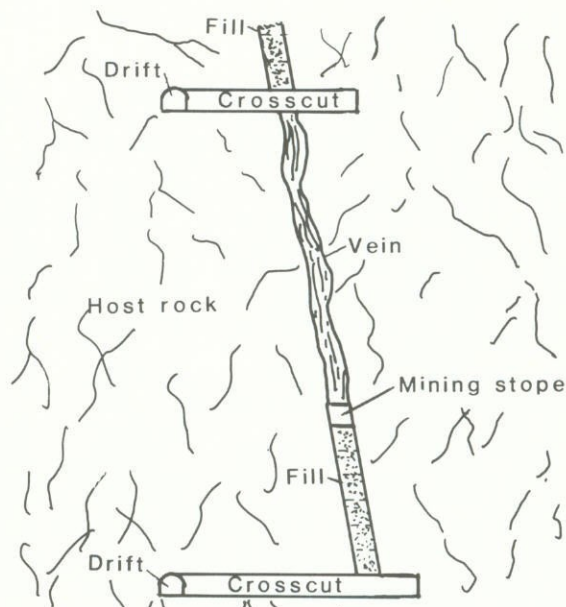


Figure 1. - Cut-and-Fill Stoping.

As each lift is mined and filled, the horizontal stresses are redistributed in the intact vein above and the fill below. Initial convergence of the wall rock compacts the sandfill to eventually limit further wall convergence. While the modulus of the sandfill is possibly as little as one-one-hundredth that of the wall rock, the support it provides is significant in terms of the stress induced in the remaining vein (17). The fill provides more than just passive wall rock support and a surface from which to continue mining: it plays an active role in the control of the induced stresses.

Remedial Measures

Rock bursts may be triggered at any stage of the cut-and-fill mining process. Bursts may occur in any of the development drifts or raises as they are being driven or in the mining stopes. If a build-up of stress can be detected soon enough, either by microseismic systems or audibly by the miners themselves, remedial measures can be taken. The most popular of these measures has been destressing. For the purposes of this discussion, the term "destressing" is applied to the drilling and blasting of relatively small problem areas where high stress concentrations

have been identified. The wall rock, or the vein itself, is drilled and blasted so that it fractures uniformly, and thereby weakens the rock and relieves the stress. Because miners must work within the confines of the highly stressed drift or stope after the problem area has been identified, destressing is a hazardous task. This is illustrated by the fact that the most recent rock burst fatality at one mine in the district occurred during a destressing operation.

Over the years, a number of other measures designed to control stress concentrations were attempted within the constraints of the cut-and-fill mining method. These included stope sequencing to control the mine geometry, end-of-shift blasting to trigger stress releases in unstable areas during idle mining periods, controlled excavation to allow a gradual release of strain energy, and active and passive support systems to limit drift damage in the event of a burst. These approaches were successful to various degrees; however, they all exacted a cost in terms of productivity. These methods are remedial in nature and slow the mining process, thus adversely affecting production. The fact that, even with these remedial measures, rock bursts continued to be a serious problem indicated that a different approach was needed.

Preconditioning

Researchers next began looking at ways to alter the physical properties of the rock itself. Bureau and industry personnel investigated the concept of uniformly fracturing massive areas of rock where high stress would likely concentrate under normal mining conditions. This technique is referred to as "preconditioning." In practice, preconditioning is accomplished in much the same way as destressing--that is, by drilling and blasting rock. However, while destressing attempts to release high levels of strain energy already existing in localized areas, preconditioning attempts to fracture and weaken large blocks of rock to prevent strain energy from building to dangerous levels in the first place. Preconditioning is introduced into the mining cycle as a development step. Either the vein or the walls can be preconditioned, depending upon which is more likely to store strain energy and burst.

Preconditioning, initially referred to as pillar destressing, was first used in the Coeur d'Alene district during the late 1960's at the Galena Mine (18). Microseismic monitoring of a pillar at the Galena indicated development of a potential burst zone in a 135- by 30-ft block of ore. Blast holes were drilled up through the

pillar on a 5-ft spacing and the pillar was softened by blasting. Seismic velocities measured through the pillar before and after the blast demonstrated a reduction of the pillar stiffness, and subsequent mining of the pillar took place without mishap.

The success at the Galena Mine led to further demonstrations of preconditioning at other mines in the district. The most ambitious of these took place at Hecla Mining Company's Star Mine in the late 1970's. Two experiments were conducted at the lower levels of the Star in an area where the number of rock bursts was increasing (19, 20).

In the first experiment, the vein was preconditioned in one area while the wall rock parallel to the vein was preconditioned in another. Seismic velocity, stress, and displacement measurements all indicated significant fracturing in both preconditioned areas prior to mining the stopes; however, the effectiveness of preconditioning was not the same in both cases. Where the wall rock parallel to the vein had been preconditioned, difficult mining conditions were still encountered. Where the vein had been preconditioned, mining was not accompanied by the usual rock popping or seismic activity. This success of the vein preconditioning led to a second experiment.

The second test took place directly below the previous one, but this time only the vein was preconditioned. The extensive drilling and blasting that took place during this test was completed in two phases and spanned a 2-year period. Some microseismic activity and two large rock bursts accompanied subsequent mining of the stope, but overall the preconditioning was considered effective in reducing the frequency and severity of bursts.

A technical disadvantage became apparent, however, during the Star Mine tests. For preconditioning to be effective, the vein had to be fractured quite extensively. This presented ground control problems during mining of the preconditioned stope. Since cut-and-fill mining proceeds upward into the ore body, the preconditioned ore is above the heads of the miners in the stope and is more likely to collapse than if the ore has not been preconditioned. The search for a solution to this situation led to a new mining method: undercut-and-fill stoping, also called underhand stoping. As the name implies, the backfill is placed above the active mining cut. Since the fill is above the mining stope, a preconditioned ore body is not a problem. Of course,

extra measures must be taken to keep the fill in place, so cement is added to the sandfill and steel mats or timber are anchored to the walls above to help support the back. Undercut-and-fill mining was originally developed to mine naturally fractured vein-type ore bodies, so there are many similarities between it and the more traditional cut-and-fill stoping method. The one outstanding difference is that mining proceeds downward with cemented fill placed above the active mining area.

In order to test the combination of preconditioning and undercut-and-fill stoping, the Bureau sponsored another demonstration project. First, undercut-and-fill stoping in other parts of the world was studied to determine how best to apply this mining method in the Coeur d'Alene district. A mining plan was then presented to the major mining companies in the district; Hecla Mining Company again offered to test it in their Star Mine (21).

The site at the Star Mine was to be a newly-developed block of ore, but this plan was abandoned when a large rock burst caved mining stopes below a sill pillar in another part of the mine. Only four lifts remained in the sill pillar on the 6700 level when it burst. Large expanses of the vein had been mined and filled above and below the remaining pillar, and the converging walls had created extremely high stresses in the vein. Rock bursts are common in this situation and are called sill pillar bursts. Mine management decided to stop mining the sill pillar with the overhand cut-and-fill method and test the combination of preconditioning and undercut-and-fill stoping. Blast holes were drilled through the remaining ore and detonated. Once the remaining pillar was heavily preconditioned, mining began from the top down using the underhand mining method.

The novelty of undercut-and-fill mining presented some problems for the miners, but overall the project successfully demonstrated that the new mining method, used in combination with massive destressing, could control rock bursts. Even though the pillar was highly burst prone, only one moderate burst occurred during the project. That burst was located in the wall of the stope and caused limited damage.

The Star Mine undercut-and-fill project was the last preconditioning test in the district. The tests at the Galena and the Star demonstrate that preconditioning can effectively reduce the incidence and severity of rock bursts in a stope. The major disadvantage of preconditioning is the high cost. The drilling techniques are

labor- and equipment-intensive and explosives are expensive. These costs must be added to the already high cost of cut-and-fill mining.

New Mining Methods

The success of the final Star Mine demonstration led some to consider that undercut-and-fill stoping was an inherently safer mining method for controlling rock bursts. Numerical analysis showed, however, that this is not the case. Undercut-and-fill stoping, by itself, would not eliminate, or even reduce, rock bursting. The major rock burst problem created by traditional cut-and-fill stoping results not from the direction of the mining, but from the creation of pillars between levels. Figure 2 shows a simplified vertical section of several stopes and illustrates how pillars of ore are isolated between sandfill areas when mining takes place on multiple levels. As the pillar size is reduced, the remaining sill pillars become highly stressed and bursting increases sharply. If undercut-and-fill stoping is substituted into the same multi-level mining scheme, without preconditioning the ore, the results will be the same. Pillars will be isolated at the bottom of the stoping levels instead of the top. The development of an inherently safer mining method requires a system that eliminates these highly stressed, burst-prone pillars from the mine.

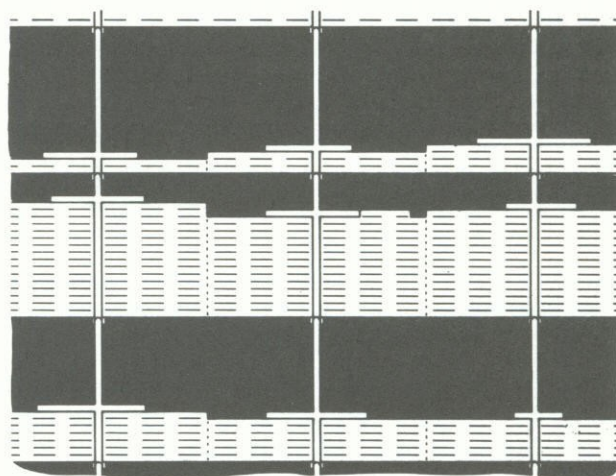


Figure 2. - Multiple-Level Mining.

One mining method that meets this need is single-level underhand longwall stoping. Mining of individual stopes in this method is similar to undercut-and-fill stoping; however, all of the stopes in the mine advance side-by-side on the lowest level of the mine. Mining always continues downward into virgin territory

without creating sill pillars. In order to push the stress wave ahead of the longwall and away from the stopes, this method requires a high-modulus backfill.

Finite-element studies designed to compare the strain energy release rates for multiple-level mining, whether overhand or underhand, and single-level longwall mining clearly show the advantage of the longwall method. For the multi-level systems, the energy release rate remains relatively constant while mining the first 150 ft of the level, then it sharply increases as the sill pillar gets smaller. This analysis is corroborated by an increasing number of bursts when the last 50 ft of a sill pillar are mined. Using the single-level longwall system, the energy release rate remains constant throughout the extraction of a 200-ft section of ore.

To test the underhand longwall concept, the Bureau began a cooperative project with Hecla Mining Company and the University of Idaho at the Lucky Friday Mine in 1984. This field trial, referred to as the Lucky Friday Underhand Longwall, or LFUL, continues today at the Lucky Friday even though production at the mine stopped in mid-1986. Rock bursts in other parts of the mine were partly responsible for the mine closure, and some believe the reopening of the mine to full production depends upon the successful development of a new mining method.

Upon what does the successful development of the LFUL depend? A major problem involves the fact that it is a single-level mining system. By contrast, in multiple-level mining, production stopes are scattered throughout the mine, and if more tons/day are needed, additional stopes can be added. With the single-level longwall system, however, the total mine production must be concentrated on one level. This means that the number of stopes will be limited to four or five at most, and production from these stopes must be maximized for the mine to be profitable. This means mechanization.

The spiraling ramp shown in figure 3, a schematic of the experimental LFUL stope, is a drastic departure from traditional mining practices in the Coeur d'Alene district. It was driven to accommodate rubber-tired drill-jumbos and load-haul-dump units. This was one of the few real attempts to mechanize mining in the district with relatively large equipment. Therefore, the success of the LFUL not only depends on the elimination of serious rock burst problems at the Lucky Friday Mine, but also on increasing productivity so that four mechanized stopes can produce the tonnage needed for the mine to be profitable.

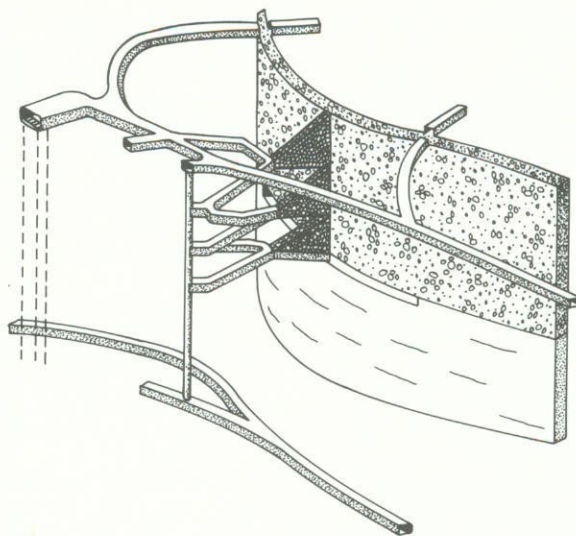


Figure 3. - The LFUL Experimental Stope.

Preliminary numerical modeling indicates that success also depends upon the development of a high-modulus fill. Since fill is placed above the active mining area, the cemented fill must provide a safe and sound roof for miners in the stope. But this is only part of the problem; the fill must also be capable of supporting the walls in order to control the stress concentrations of the extended longwall. The situation is similar to a hanging roof on a coal mine longwall. When the roof above the gob refuses to cave, the cantilevered strata create high stress concentrations at the longwall face. In the case of the underhand longwall, the strong wall rock cannot be expected to break; therefore, the vertical "cantilever" of the converging walls must be supported by a very stiff cemented fill. The success of the underhand longwall method depends heavily upon the successful development of a stiff fill support system. Otherwise, high stress concentrations at the longwall could create a serious rock burst problem.

Problems in designing an adequate filling system stem from the material being used and the placement system. The most economical fill material is mill tailings to which cement and water are added. Because the particle size of the tailings is very small, it is difficult to create the cement matrix around each particle necessary for good consolidation. Water is also a problem. Too much water in the mixture lowers the strength, but since the fill must be pumped into the stope, maintaining the correct water/cement ratio is critical. So an important part of the LFUL research has

been to develop both a mix design and a transportation and placement system for a stiff fill.

Research at the Lucky Friday Mine has been temporarily halted by a reorganization at the mine. Three cuts have been removed from the LFUL stope, two of which have been filled using a temporary cemented fill system. Instruments placed in the fill indicate that the mix design used in the first two cuts is not providing adequate support to meet the anticipated needs of the underhand longwall. An improved fill performance is expected once the stiff fill system becomes operational. The experimental stope and the ramp system have been very stable without the stiff fill; however, a relatively small area of the vein has been mined. A 400-ft-long by 30-ft-high section of the vein has been mined to this point. Productivity with the mechanized equipment has been encouraging and development of the stiff fill system is nearing completion. Overall, researchers are optimistic about the future of the system, and Hecla was considering the addition of a second stope to the experimental longwall just prior to shutting down the LFUL stope.

FUTURE RESEARCH

It should not be surprising that the Bureau feels that rock burst control in the Coeur d'Alene Mining District depends on successful accomplishments in the two major research areas we are pursuing. The first major area is the development of predictive capabilities, microseismic or otherwise. This is very important because it is probably impossible to completely eliminate bursting with control measures. Passive control measures, such as cable lacing, can limit the hazards of flying rock and caving if a burst occurs, but support measures cannot protect against massive rock bursts. A reliable prediction system that can warn of imminent bursting is critical to the safety of the miner.

An aspect of microseismic research that should not be overlooked is its value in analyzing rock burst source mechanisms. Our current understanding of the factors which contribute to bursting have presented several possibilities regarding ways to control bursts, such as preconditioning. Further understanding of rock burst source mechanisms will present new avenues of research. Rock burst mechanisms are currently being studied in the mine and in the laboratory. Laboratory work is focusing on fracture mechanics and associated acoustical emissions while field work is concerned with assessing microseismic activity around the experimental LFUL stope to identify source

mechanisms and evaluate the potential of the mining method to cause bursts.

Developing an inherently safer mining method to control rock bursts is the second important research area. Development of predictive capability alone will not solve the rock burst problem in the Coeur d'Alene Mining District. Bursting is not only a very serious safety problem, but is a productivity problem also. A foolproof predictive system would not, by itself, eliminate or even reduce the underground damage that accompanies a burst. Very few of the rock bursts in the district kill or injure miners. Aside from psychological effects, the greatest impact of the vast majority of bursts is on production. Lost production due to rock burst damage, or even the threat of further bursting, adds costs which can shutdown a mine. This point emphasizes the importance of engineering an inherently safer mining method for the district, which in turn will result in increased profitability. The LFUL project is an attempt to develop a mining method that can be utilized profitably.

A standard geomechanical research approach is being employed to evaluate the LFUL. This approach allows alternative mining methods to be evaluated as well. All of the rock mass characteristics of the site are being gathered and fed into a finite-element model of the stope geometry. Output from the model predicts energy release rates, ground movements and fill pressures, and contours stress concentrations. These calculated responses are then compared to the actual response measured in the field. The finite-element model may have to be refined if there are discrepancies, but once the two match, the "calibrated" model can be used to improve the LFUL design for optimum performance. An example of the type of problem to be examined is the design of stope intersections in the underhand longwall. It may be necessary to leave yield pillars between stopes or to stagger the level of adjacent cuts to prevent high stress concentrations at intersections. Finite-element studies will indicate how stresses are likely to be distributed for different configurations.

The calibrated model will also be used to study other mining methods for rock burst control. An accurate numerical representation of the mine site will provide an opportunity to numerically "mine" with a variety of mining methods. Results will indicate how stresses are likely to be transferred and dissipated under different mining methods and conditions. Some type of bulk mining method, for example, may be possible that

would not only better manage the stress concentrations, but also remove men from the stope and reduce their exposure to rock bursts. Problems with numerical modeling of underground structures stem from inaccurate input for the structural and material models. A calibrated model will ensure the reliability of the modeling approach. This type of research provides an inexpensive way to test many theories and implementation techniques before putting them to a real test in the mine.

SUMMARY

The Bureau has cooperated with industry to study rock burst phenomena for over 30 years in the Coeur d'Alene Mining District. Early work focused on methods to mitigate bursting problems within the context of the existing mining method--cut-and-fill stoping. These efforts included developing microseismic techniques to locate and predict where and when a burst was likely to occur, controlling mining rates and stope sequences to release strain energy gradually, and drilling and blasting to destress hazardous areas. Although remedial control methods were sometimes effective in reducing rock bursts, they had a detrimental effect on productivity.

Recent research in the district has investigated methods for altering or controlling the conditions that can lead to rock bursts. These designed control measures have become a part of the planning and development stages and are aimed at either changing the properties of the rock mass so that it cannot store strain energy at dangerous levels or improving mining techniques to reduce stress levels in critical areas. Designed control measures include preconditioning to alter rock stiffness and engineering new mining methods to lower rock stress around mining areas. Preconditioning has proven to be very effective in several mine tests; however, it has also been very expensive. The largest current effort, the LFUL project, is testing a new mining method to reduce the number and severity of rock bursts in and around production stopes.

A new mining method is important not only to make the mines in the Coeur d'Alene district safer places to work, but also to make them more profitable. Lost production due to rock burst damage or the threat of further bursting can shutdown a mine. Inherently safer methods are not the sole solution, however. Bursts that occur during stope development, for example, are difficult to prevent with control measures. Research continues in developing microseismic and other techniques that will reliably predict rock bursts. By developing safer mining methods which remove the miner from burst-prone areas and

developing a predictive capability for areas where removal is not possible, we can greatly reduce, if not eliminate, the safety hazards and productivity losses associated with rock bursts in the Coeur d'Alene Mining District.

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CHEMICAL DESTRESSING TO ALLEVIATE ROCKBURSTS

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ABSTRACT

Destress blasting is commonly used in the mines to minimize the occurrence of rockbursts. The fracturing created by destressing in the vicinity of the face reduces the stress concentration by pushing the zone of high stress deeper into the rock mass. But the destressing practice by blasting may be devastating near the faults or other sensitive high stress zones. To avoid severe rockbursts in such situations, the application of chemical destressing has been proposed in this paper.

A laboratory investigation was conducted on the rocks in the Sudbury basin to examine the change in the material properties with an aim of assessing the feasibility of destressing by chemical means. The favourable change observed in the rock properties led to the suggestion that the injection of surfactants in boreholes would enhance fracture propagation similar to what is currently done by blasting but in a safer, inexpensive and non-violent manner.

The current destressing practices in Ontario mines have also been reviewed in this paper.

INTRODUCTION

Destressing is commonly conceived as a blast fracturing technique to relieve stress in a potential rockburst zone (1). The fracturing reduces the load carrying ability of the rock which results in a stress reduction in the destressed zone (2). The purpose of destressing is to:

- a. Push the pressure front further ahead, presumably away from the mining area.
- b. Reduce the stress and the stored strain energy.
- c. Change the failure mode from brittle elastic to elastoplastic or plastic.
- d. Allow the gradual yield of the fractured zone rather than sudden and violent failure.
- e. Propagate minor bursts (3).
- f. Distribute stress more evenly and over a large area than prior to destressing.

The change in the stress concentrations around mine excavations has been displayed in Figure 1.

Review of Destressing Practices in Ontario:

Destress blasting is done to fracture a burst prone rock mass subjected to high stress concentrations. The practice in Ontario mines differ not only from mine to mine but in different locations of the same mine. The following examples reflect an important segment of the destressing practices under different mining situations (4).

A. Production Mining

Size of holes:

- Diameter - 1 1/2" - 1 3/4"
(38 mm - 44 mm)
Depth - 10' - 90'
(3 m - 27 m)
Spacing - 10' - 15'
(3 m - 4.6 m)

Direction of holes:

Horizontal holes: to destress ground adjacent to stopes and pillars

Vertical holes: to destress the ground above the stope

Charging: The explosives commonly used for destressing are Cilgel 70%, Amex and Tovex, etc. Most of the time holes are loaded completely.

Blasting: Destress holes are usually blasted with the stope blast. However if a large area is to be destressed, it is done prior to resumption of mining.

B. Development: The typical information about the destress blasting in development openings has been given below and shown in Figure 2.

Diameter of holes - 1 1/2" (38 mm)
Depth of holes - 10' (3 m)
Spacing of holes - 10' (3 m)

About 5' (1.5 m) from the toe of the hole is charged with explosive.

- C. Shaft Sinking: The destressing practice in a shaft sinking operation in Ontario has been presented in Figure 3. The shaft was sunk by benching method with 6 feet (1.8 m) benches.

Depth of the shaft: 5100' (1530 m)
Number, size and orientation of holes:
Three 1 1/4" (32 mm) holes 8'
(2.4 m) deep drilled at - 45° into
the wall.
Two 12' (3.6 m) long holes drilled in
the shaft bench.

SCOPE AND OBJECTIVE OF THE APPROACH

It is obvious that the concept and mechanics of destress blasting are relatively simple but its application in the mining environment is neither straightforward nor consistently successful. The stress shift by destressing, even though small, may cause problems in the nearby areas, particularly when those areas are critically loaded (5). The sensitive areas of special concern are the vicinity of faults, dykes and other highly stressed rock mass at the verge of unstable equilibrium.

It has been observed that the incidence and severity of rockbursts increases in the vicinity of the faults. A fault is a pre-existing plane of weakness in an otherwise undisturbed rock mass. During the process of mining, the induced shear stresses build up gradually along the plane until a critical time when suddenly a comparatively large rock mass slips along the plane triggered by a shock. Destress blasting near faults may provide the triggering mechanism and create a dangerous situation by slippage along the fault plane (Figure 4). The rockbursts which result in particularly hazardous situations are generally associated with well defined faults or dykes and cause devastating damage to a small portion of the mine (6).

The need for the avoidance of such serious rockbursts can be fulfilled by a technique which can favourably change the rock mass properties without causing a significant shock. Chemical destressing appears promising and should be pursued in a sensitive high stress situation where a shock of large intensity can trigger the rockburst.

Anderson and Grew (7), Dunning and Dunn (8, 9) suggested that the stress corrosion theory of crack propagation may have important geophysical applications. The effect of chemicals on the geologic material has been extensively studied for different purposes. Two schools of thought are prevalent. The first by Rehbinder et al. (10)

stated that the material strength is altered by a change in the surface free energy of the developing crack or fracture due to the absorption of the chemical. This concept was supported by Boozer et al, (11) and Swolfs (12). The second theory of Westwood (13) proposed that the changes in the material properties caused by the surfactant are a function of the zeta potential of the surfactant material system. The changes in the strength of the rocks have also been linked to zeta potential by other workers (14, 15).

This is beyond the scope of this paper to go into the depth of these theories. The objective of this study was to examine the influence of chemicals on the rock properties from the point of view of destressing the mine workings. In an endeavour to achieve this objective, a laboratory investigation was conducted on the rocks in the Sudbury basin to examine the effectiveness of suitable chemicals to cause favourable changes in the material properties.

EXPERIMENTAL

The tests were conducted on the specimens of Grano-diorite, quartz-diorite, diabase and rhyolite rocks from Sudbury basin in Canada. Out of each pair of specimens, one was soaked in Aluminium Chloride (0.1% concentration) for 24 hours prior to testing. The selection of Aluminium Chloride as a chemical suitable for these rocks was made on the basis of the work done by Rehbinder et al, (10). The second specimen was subjected to testing without any treatment. The following properties were compared with and without treatment.

- (i) Burst Energy Release Index
- (ii) Compressional Wave Velocity
- (iii) Compressive Strength
- (iv) Strain Energy Per Unit Volume
- (v) Modulus of Rigidity

The compressive strength, strain energy per unit volume, modulus of rigidity and compressional wave velocity were determined by standard methods.

Burst Energy Release Index:

The important feature of a rock burst is the violent fracture of the rock which is accompanied by the release of energy in amounts sufficient to damage the surroundings. This suggests the search for a parameter which represents the energy released at the time of rock fracture and can be used as a relative measure of the burst proneness of rocks.

The violent nature of the specimen fracture in the soft testing machine satisfies most of the conditions for the occurrence of the rockburst. In a mining situation, the rock mass being stressed around an excavation behaves as a rock specimen and the strata surrounding this rock mass acts as a testing machine.

The amount of energy released by rock specimens during their failure under a soft testing machine was determined by the extent of the vibrations generated in the machine. The vibrations were measured by a seismograph DS-377 manufactured by Instantel Canada. The vector sum of the peak particle velocity, being a measure of the energy released at the time of specimen fracture has been termed as "Burst Energy Release Index" (BERI) in this paper. The experimental set up has been shown in Figure 5.

RESULTS AND DISCUSSION

The specimens from Sudbury rocks were subjected to various tests. Out of each pair of specimens, one was tested as it is, whereas the second was tested after chemical treatment. The study was conducted to investigate the change in the rock properties with the aim of assessing the feasibility of destressing by chemical means.

The change in the compressive strengths of Grano-diorite, quartz-diorite, diabase and rhyolite on chemical treatment has been presented in Table 1. The change in the values of compressive strength and burst energy release index for grano-diorite has been given in Table 2. Table 3 represents the change in the modulus of rigidity, strain energy per unit volume, burst energy release index and compressional wave velocity for grano-diorite and diabase.

In most of the cases, the reduction in the properties lies between 10-20%. This range of reduction is not sufficient to cause proper destressing. But the fact that the tests were done on intact specimens of various rocks warrants due consideration. Actually, the rock in-situ around the destressing holes would be fractured and it will be easier to cause significant change in the rock material properties. The comparison between the stress-strain curves of treated and untreated specimens has been displayed in Figure 6. The change in compressive strength and modulus of rigidity of rocks on chemical treatment has been clearly depicted in Figure 6.

The favourable change in the material properties can be achieved by injecting the chemical in the destressing holes at a pressure. The injection of surfactant results in the decrease of the elastic strain energy and increase in the permanent non-recoverable strain. Consequently, the proneness of the stressed rock mass to bursting decreases (16). The effectiveness of the technique to cause destressing depends upon:

- (i) the choice of the chemical
- (ii) the pressure at which the chemical is injected
- (iii) size and spacing between the destressing holes
- (iv) geological features of the rock mass
- (v) state of stress

Further in depth study is needed to achieve judicious trade off between controllable and uncontrollable variables to achieve effective destressing.

The favourable change in the properties of the intact rock specimens led to the conclusion that the injection of surfactants in a stressed environment would produce enhanced fracture propagation in the destressing holes similar to what is currently done by blasting, but in a safer and non-violent manner.

The destressing kerf can be created by drilling closely spaced parallel boreholes. The injection of chemical will crack and break the walls between these boreholes. The creation of this kerf would result in more intensive inelastic deformation of the surrounding mass. Then more holes surrounding the kerf can be drilled depending upon the requirements.

CONCLUSIONS AND FUTURE WORK

The favourable change in the properties of the rocks on chemical treatment suggested that the injection of appropriate chemicals in the destressing boreholes would produce the desired effect, similar to what is achieved by blasting but with less vibrations and in a safer manner. But more in depth study is required to arrive at a judicious trade off between controllable and uncontrollable factors.

During this study, unstressed specimens were treated with chemicals prior to testing. But in practical situations, the destressing is done in a highly stressed rock. It will be more advantageous to carry out these tests where the rock is already loaded. These tests will provide valuable information regarding the effect of surfactants on the highly stressed rock. This is an essential step before any field application of this technique is planned.

ACKNOWLEDGEMENTS

The author wishes to thank NSERC for their funding through the grant number A4945 and mining companies in the Sudbury area for their assistance during this study. Thanks to Bob Gustas for his assistance during the experimental work.

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Table 1: Change in the compressive strength of Sudbury rocks on chemical treatment.

Rock Type	Average Compressive Strength in MPa	
	Untreated Samples	Treated Samples
Grano-diorite	216.4	185.5
Quartz-diorite	233.0	206.0
Diabase	212.6	193.2
Rhyolite	195.0	No change

Table 2: Change in the compressive strength and Burst Energy Release Index for Grano-diorite specimens on chemical treatment.

SET # SAMPLE #	COMPRESSIVE STRENGTH (MPa)		BERI Burst Energy Release Index	
	Untreated	Treated	Untreated	Treated
I (1125, 1139)	217.40	169.58	11.84	7.44
II (1117, 1108)	247.95	173.93	11.12	10.04
III (1137, 1120)	217.40	193.25	-	9.31
IV (1123, 1142)	242.50	199.05	11.62	11.53
V (1140, 1128)	222.24	202.92	11.54	9.77
VI (1110, 1111)	173.90	236.74	-	9.85
VII (1116, 1113)	199.50	145.42	13.85	6.65
VIII (1122, 1104)	244.95	223.20	13.46	
IX (1134, 1112)	211.13	195.67	9.95	9.79
X T156a (1144, 1145)	219.8	172.96	8.92	8.73

Table 3: Change in the properties of Sudbury rocks on chemical treatment.

Rock Type	Condition of the specimen	Modulus of rigidity in GPa	Strain Energy per unit volume in MJ/M ³	Burst Energy Release Index	P-Wave velocity km/sec
Grano-diorite	Untreated	67.9	0.3448	3.92	5.33
	Treated	60.0	0.2867	3.19	4.85
Diabase	Untreated	85.5	0.2643	3.04	4.97
	Treated	72.3	0.2581	2.85	4.52

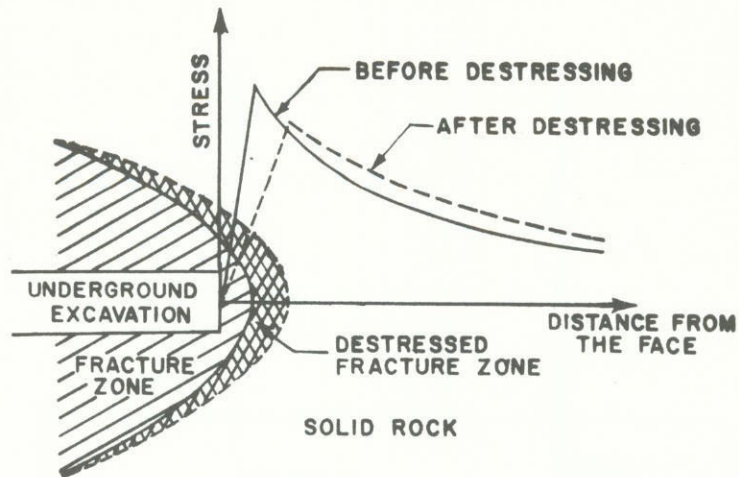


Fig. 1. The change in the stress concentrations around mine excavations.

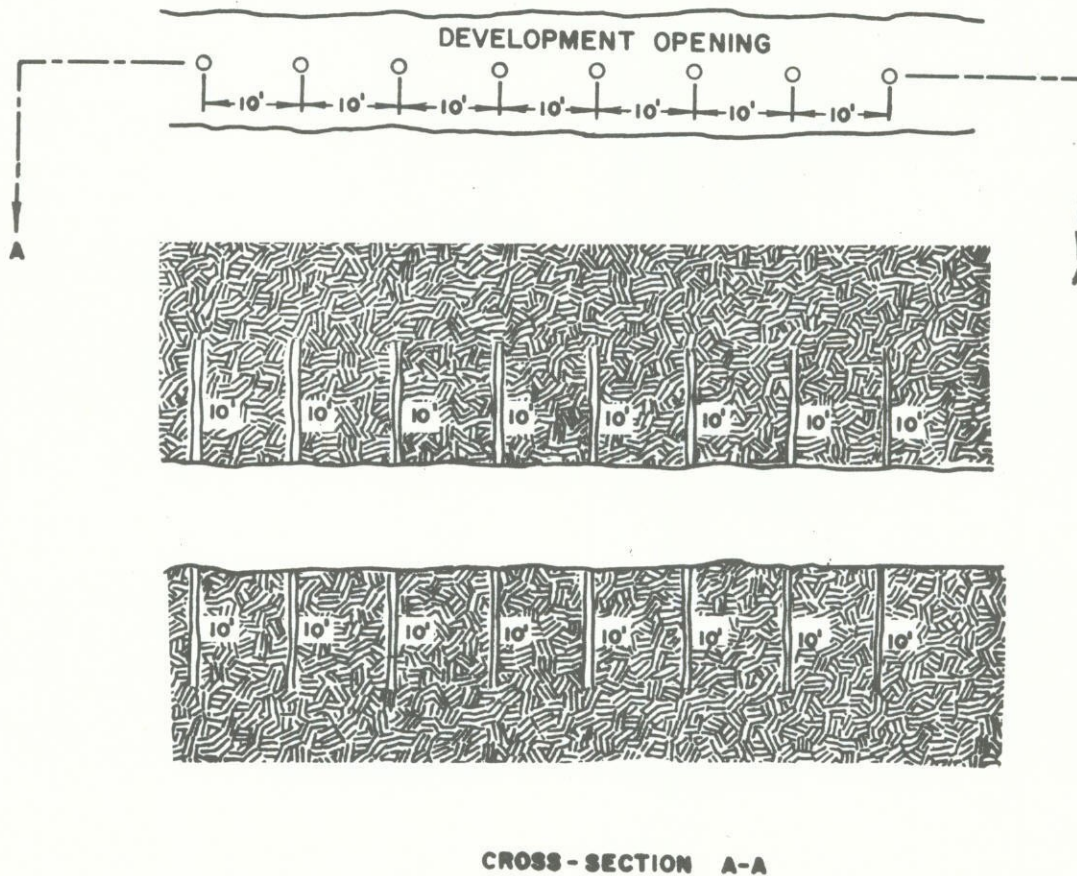
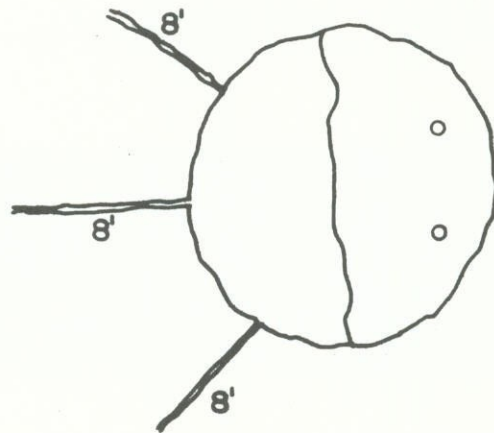
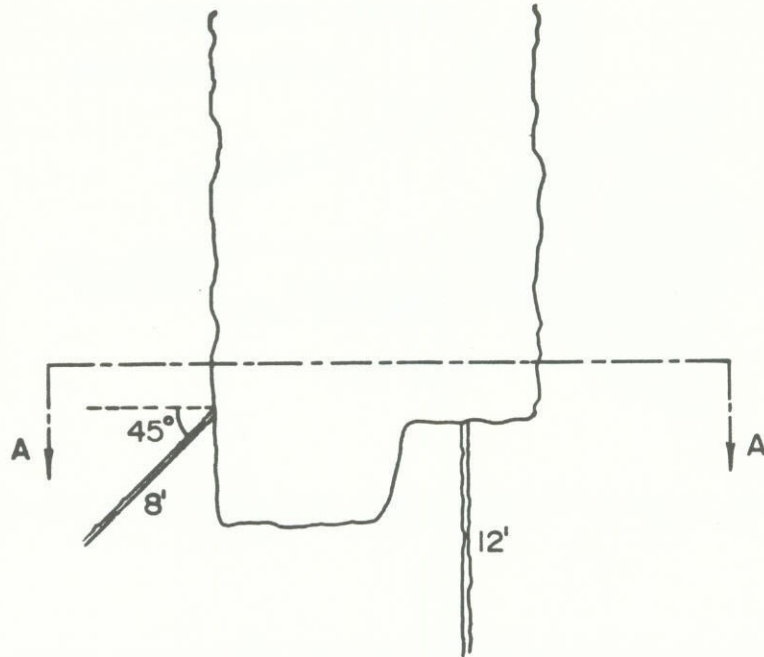


Fig. 2. Destressing practice in development openings.



CROSS-SECTION A-A

Fig. 3. Destressing in a shaft sinking operations.

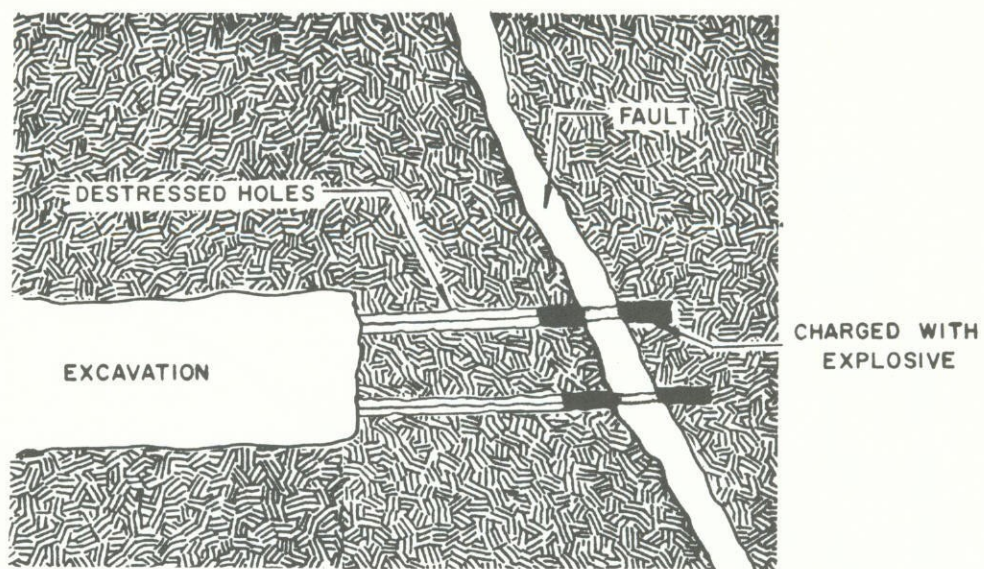


Fig. 4. Destress blasting near fault zone.

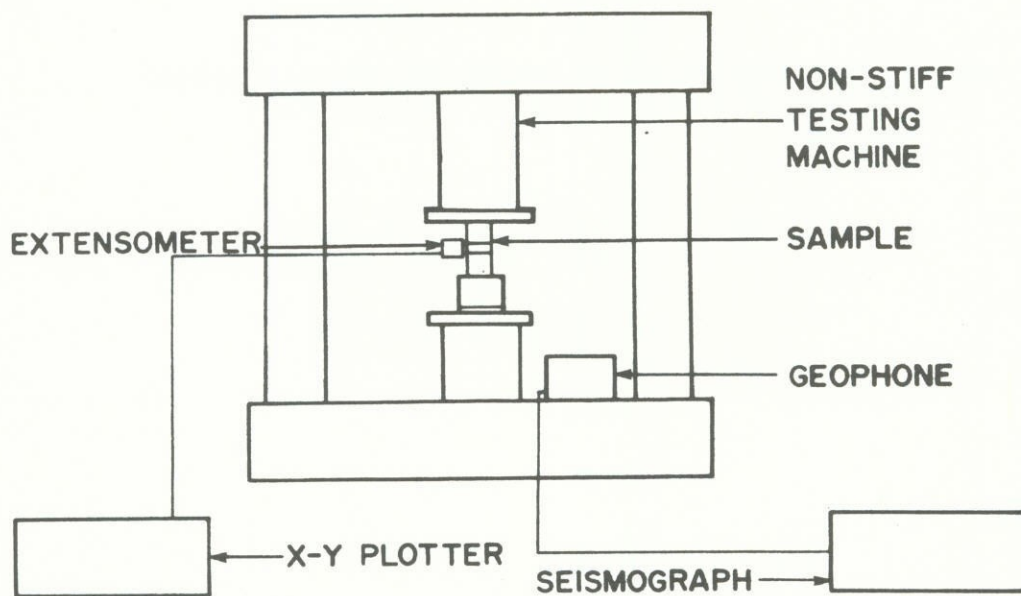


Fig. 5. Experimental set up for measuring the vibrations in a soft testing machine.

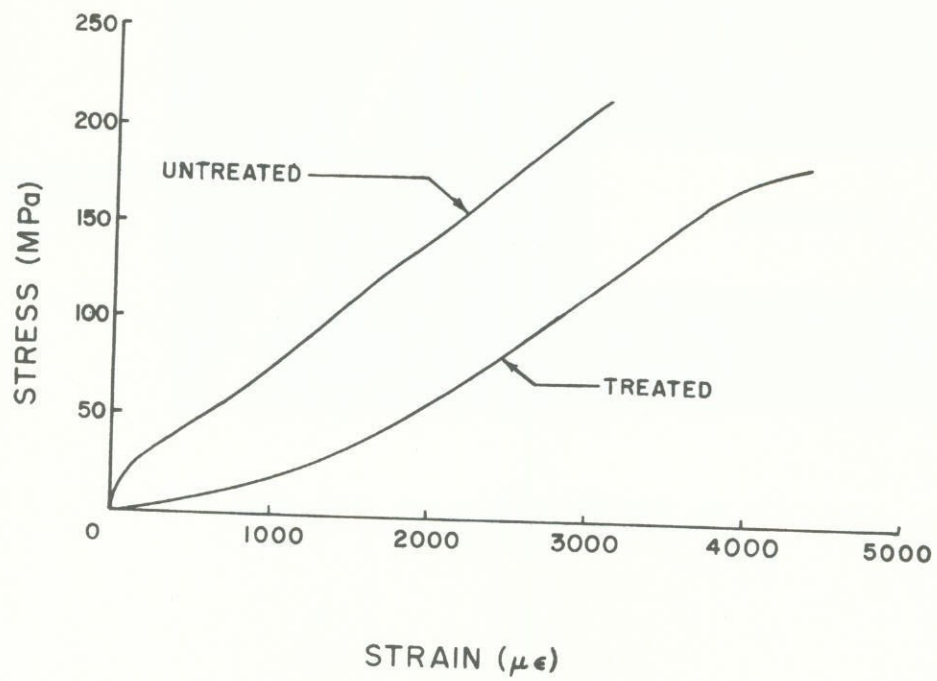


Fig. 6. Stress-strain curves for untreated and treated specimens.

ROCKBURST CONTROL MEASURES AT INCO'S CREIGHTON MINE

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ABSTRACT

Rockburst conditions were first encountered at Inco's Sudbury area operations in the mid 1930's when mining advanced to below the 610 m (2000 ft.) horizon. Now, mining is being carried out at depths down to 2130 m (7000 ft.) at Inco's Creighton Mine. While rockburst potentials have increased with depth, improvements in mine planning, methods, support systems and developments in destressing have alleviated the condition to where it is less of a problem now than it was in the early years. Many of the improvements were developed, of necessity, through observation and reasoning because these were the only tools available at the time. Now, we are making use of stress measurements, numerical modelling and microseismic source location technology to help further reduce rockburst hazards.

INTRODUCTION

Inco Limited currently has eight operating mines at various locations on or near the footwall contact of the nickel irruptive unit in the Sudbury District of Ontario, Canada, figure 1. Seven of the mines are at depths of less than 1200 m (4000 ft.). The eighth, Creighton has most of its production coming from depths ranging between 1645 m and 2130 m (5400 to 7000 ft.). A ramp access system is under development to extend mining to below the 2130 m (7000 ft.) horizon.

Creighton Mine began operations in 1901. Figure 2 is a general section of the mine showing the orebodies and some of the key development openings. Mining to 1935 was restricted to elevations above 790 m (2600 ft.). 5 Shaft, completed in 1936, opened up mining to the 1160 m (3800ft.) horizon, 6 Shaft to the 1525 m (5000 ft.) horizon in 1943, 8 Shaft to the 2010 m (6600 ft.) horizon in 1960 and 9 Shaft to the 2130 m (7000 ft.) horizon in

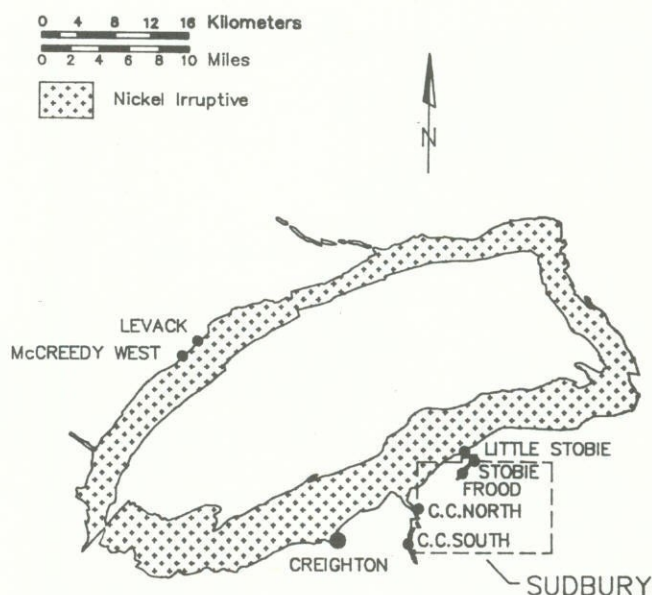


FIGURE 1. Plan of the Sudbury Basin showing the nickel irruptive and the locations of Inco's eight operating mines.

1969. No fill was used to mine the ore above the 460 m (1500 ft.) horizon. All mined openings below this horizon have been filled, although in some cases filling has been of a deferred nature. The first fill was rock crushed to minus 150 mm (6 inches). In the early fifties, hydraulically placed classified mill tailings were introduced. The addition of cement to the fill began in the early sixties.

The first rockburst at Creighton Mine occurred in 1934 at a depth of 700 m (2300 ft.). Since then, as mining has progressively advanced down dip, there

have been a total of 897 documented rockbursts, with the largest, a Richter 4.0 event, occurring July 6, 1984, well out in the hangingwall and above the 1160 m (3800 level) horizon.

Three transition horizons for rockburst problems have been identified at Creighton Mine. The first, at the 700 m (2300 ft.) horizon is the normal upper limit of depth for crown and remnant pillar rockbursts in both longitudinal and transverse mining operations. The second, the 1220 m (4000 ft.) horizon is the upper limiting horizon for development openings in virgin areas to experience rockbursts. The third, the 2010 m (6600 ft.) horizon is where rib pillars began to burst during transverse silling operations.

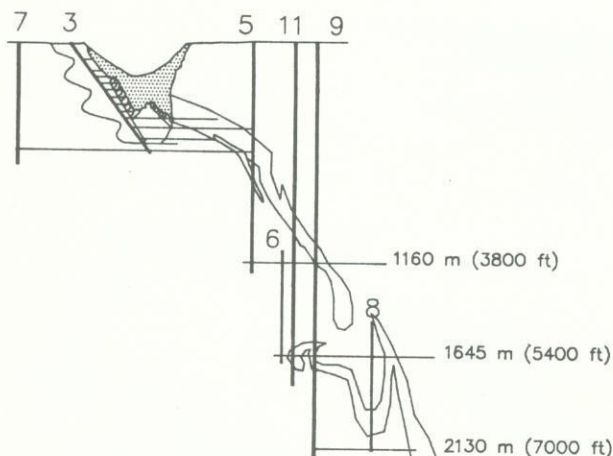


FIGURE 2. A general section through Creighton Mine

There are three factors over which we have no control when dealing with rockbursts, the geology, mining depth, and field stresses. This paper describes the rockburst control measures developed and practised at Creighton Mine to alleviate the rockburst problems associated with the above given factors.

GEOLOGY

The orebodies in the Sudbury District are all related to an elliptical shaped nickel irruptive unit which is believed to be the result of a meteorite impact. The long axis of the ellipse measures about 54 km (34 miles) and is oriented at a bearing of N65E. The short axis measures 22 km (14 miles), figure 1.

Creighton Mine is located in the South-West quadrant of the irruptive, along an embayment into the footwall rocks. The ore host rock is a discontinuous sublayer unit of the

irruptive, consisting of a mass of basic to ultrabasic inclusions of varying size and frequency of occurrence in a matrix of norite and sulphides. It is sandwiched between the main mass norite hangingwall and the footwall rocks which are mostly metavolcanics with intrusions of granites and gabbros.

The ore ranges from massive to disseminated, is structurally controlled and is separated into distinct orebodies. The average dip is 65 degrees. Disseminated orebodies range in width from 3 to 15 m (10 to 50 ft.). Strike lengths can be as much as 300 m (1000 ft.) and the down dip extent can be up to 760 m (2500 ft.). In places, the ore moves into the footwall rocks in the form of massive sulphide pods, associated with a steeply dipping quartz diorite dyke with a strike normal to the norite contact. The strike length of the pods can be up to 150 m (500 ft.) and they may extend up to 150 m (500 ft.) into the footwall rocks.

FIELD STRESSES

Field stresses have been measured at several horizons at Creighton Mine and the results are plotted in figure 3. The field stress pattern conforms closely to the normal for Northern Ontario "1" with a high principal horizontal stress, striking approximately parallel with the nickel irruptive contact. Stress measurements and experience also indicate the existence of anomalous zones where field stress values may be well above or below average projected values. The extent of these anomalous stress zones has not been established; however, stress related adverse ground conditions in lateral

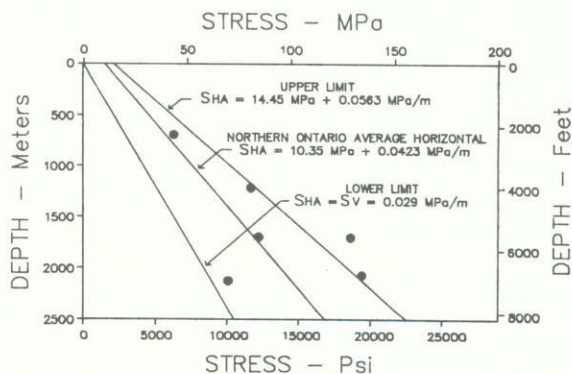


FIGURE 3. A graph showing the gradient for average horizontal stresses in Northern Ontario with upper and lower bound. It also shows principal stress magnitudes measured at Creighton Mine.

development headings have been known to occur over distances of up to 150 m (500 ft.). No structural or other features have as yet been identified as causing the anomalous stress zones.

The intermediate and tertiary stresses are about equal and strike normal to the general nickel irruptive contact. One dips at 25 degrees across the contact and the other at 65 degrees down the dip of the contact. The resultant average vertical stress component is equal to superincumbent loading when one takes into account the high average specific gravity of 2.95 for the ultrabasic host rocks as against the more normal figure of 2.65.

While 2133 m (7000 ft.) is not deep by South African standards, the effect of the high horizontal stress gradient is to create stress conditions at 2133 m equivalent to stresses at depths of 3800 m (12,500 ft.) in South Africa. The presence of the anomalous high stress zones make the situation even more striking. The stress measured at the 2070 m (6800 ft.) horizon of 133.6 MPa (19,370 psi) projects to a depth of 5000 m (16,400 ft.) in South Africa.

ROCK STRENGTH

The strength of the ore at Creighton Mine ranges between 125 and 150 MPa (18,000 to 22,000 psi). The hangingwall norite has an average strength of 155 MPa (22,500 psi) and the footwall rock strengths range between 180 and 210 MPa (26,000 to 30,500 psi). All strengths quoted are normalized to the NX core size of 54 mm (2-1/8 inches).

ROCKBURST AND DEVELOPMENT OPENINGS

There is a simple reason for the rockburst problem in development openings. Stress in the skins of the openings exceeds the rock strength. At a depth of 1220 m (4000 ft.) the normal principal stress is 62 MPa (9000 psi.). The stress multiplier factor for a simple circular shaped opening is about 2.3 for the stress ratios at Creighton Mine which would give a skin stress of about 143 MPa (20,700 psi) or something less than the strengths quoted for the NX cores. However, size effects reduce the average rock mass strength to something in the order of 100 MPa (14,500 psi), well below the magnitudes of the induced stresses.

The key to minimizing the rockburst hazard in development openings at depth is to recognize that stability requires the establishment and maintenance of a skin of failed material around all openings.

DRIFTS

In the forties, when most of the new development work moved to below the 1220 m (4000 ft.) horizon and was liable to be hit by rockbursts, timber or steel sets were introduced to support and maintain the zone of failed material about the openings. Square set booming practices were used to safely advance the sets which were maintained tight to the face so that at no time were miners exposed to unsupported ground. With square set booming practices, it was only necessary to scale the more immediate loose before booming out and installing the back blocking.

The first rockbolt was installed at Inco on January 11, 1952. Through the fifties, their use at Creighton was restricted to development headings above the 1220 m (4000 level) horizon where the standard procedure of scaling backs to sound intact rock could be followed. Below the 1220 m (4000 ft.) horizon, it was hazardous to attempt to scale to sound rock. Fresh loose would form explosively and could injure the person doing the scaling. In some areas, it was possible to support the backs on pin sets and to use rockbolts in the walls where scaling rules could be relaxed.

In the early sixties, three factors were introduced which made the driving of drift headings at depth safer and made it possible to eliminate timber or steel for back support. These were: the use of screen with mechanically anchored rockbolts, destressing, and the conversion from dynamite to ANFO for all blasting operations. The three items together worked to create and to maintain the skin of failed rock around drift headings.

Screen and Rockbolts

Screen made it possible to safely support ground with mechanically anchored rockbolts without having to first scale to a tight, solid, condition. Only sufficient loose is scaled to make it safe to install one bolt at a time. No one is permitted to work ahead of the screen line.

In mucking machine headings, only half the round can be mucked before screening and bolting operations take place. In mechanized headings with scooptrams, the operator sits far enough back to muck out the entire round without being exposed to unsupported ground.

Screen has a secondary benefit, in that it will bag and contain fly material from rockbursts. As a rule of thumb, the screen should be able to contain more than

half the displaced material from rockbursts. Anything less is a sign of too weak or stiff a screen or the rockbolts are too widely spaced.

Destressing

Destressing was first used in the early sixties "2" in the cross-cuts driving out from 8 Shaft below the 1645 m (5400 ft.) horizon. It started with a single 14 m (47 ft.) long, 57 mm (2-1/4 inch) diameter longhole drilled along the centerline of the drift. The hole was loaded to 3 m (10 ft.) from the collar and blasted. Four, 3 m (10 ft.) drift rounds were taken before the next destress hole was drilled and blasted. Destressing stopped the rockbursts with one exception. In that case, the procedure was not followed and an extra drift round was taken before the next destress hole was drilled and blasted. A rockburst occurred where the overlap of the destress holes was missing. This was taken as a clear sign of the effectiveness of destressing in that a rockburst occurred in the one case where destressing was not done properly. Subsequent incidents have added to the proof.

Two and three hole patterns were tried but offered no improvement over a single hole.

The overall effect of the destressing blasts was to double the numbers of joints visible in the walls and backs of openings in comparison to the numbers of visible joints in areas developed before destressing was introduced.

Eventually, the development cycle changed from one long round each two shifts to one shorter round each shift to increase advance rates and efficiencies. It became no longer feasible to stop a heading to set up a longhole machine to drill a destress hole. The practice changed to drilling two of the shoulder height, 32 mm (1-1/4 inch) drift drill holes with each round to a distance double the length of the holes in the round. These holes were loaded from the toe to the normal collar point and blasted with the round.

It was found that not all headings needed destressing. Most commonly, those that did require destressing were oriented at right angles to the direction of principal stress or were close in to the orebody. The twin destress holes made it difficult to drill the face and there was resistance to destressing by development crews unless the ground clearly demonstrated the need for destressing. One enterprising crew, in an area where destressing was needed, changed over to

drilling the destress holes up and out about 30 degrees from the top corners of the drift. The bottom metre (3 ft.) of these holes were loaded and blasted at the time the cut and helpers were detonating. The method proved to be very effective.

Today, most headings are for mechanized equipment and are driven with drill jumbos. They are larger than the tracked headings in which most of the destressing developments occurred. Cracked ground in the face is not the problem it was with hand held drills. Current destressing practices call for two drill holes about one-third the way up the face to double normal depth and two corner holes drilled up and out about 30 degrees, figure 4. The bottom 2.4 m (8 ft.) of the face holes and the bottom metre (3 ft.) of the corner holes are loaded. Both sets of destress holes are blasted with the round but are timed to detonate just ahead of the cut holes.

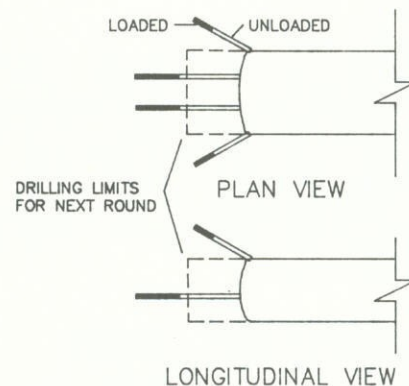


FIGURE 4. Drift destressing holes.

Destressing is more of an art than a science. The normal destressing pattern works in most situations. We have found, however, that there are places where we may need more face holes and we may also require holes drilled down and out from the lower corners in the same manner as those up and out from the upper corners. To date, we have always found it possible to develop a destressing pattern which will handle the worst inherent rockburst problems. All it takes is the persistent application of Terzaghi's Observational Approach (trial and error).

In a recent application, destress holes drilled out from the sides of the drift were substituted for the corner holes.

They were not effective. The experiment was not a total failure in that it, along with failure patterns observed in the development of a numerical modelling code with plasticity, provided some insight into the role of the corner holes. Corners create confined zones of highly stressed material. There is energy stored in these zones and they are potentially unstable. Most of the displaced material from rockbursts in development openings comes from the corners.

Figure 5 shows the predicted failure pattern around a drift opening in highly stressed ground using an elastic solution. The predicted failure zones are restricted to the sides, back and floor of the opening. There is no failure predicted for the corners. Stress plots would show both high major and minor stresses at the corners, with failure restricted by the high confining stresses. Figure 6 shows the predicted failed zone around the same heading where failure has been allowed to progress to equilibrium. In this case, the failure envelope is uniform around the opening. We believe corner distress holes help in the transition to the stable situation modelled by figure 6.

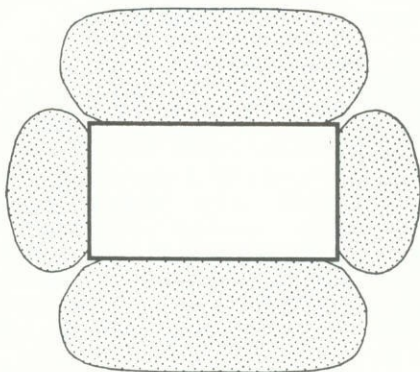


FIGURE 5. The predicted extent of failure about a drift using an elastic solution.

The rule that destressing will only be effective where the rock is stressed to within 80% of failure may have to be changed under the rock behavior modifier concept. The only difference I can see between modifying a section of ground prior to it becoming highly stressed and modifying it once it has become highly stressed is that the stored energy in the highly stressed ground may augment the energy from the blast in modifying the rock properties.

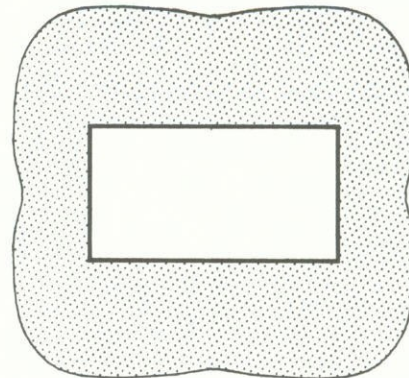


FIGURE 6. The predicted extent of failure about the same drift using a plastic solution.

The basic rule of destressing is to hit it where the stresses will be highest and to hit it hard. There is a tendency to be too gentle with distress blasting. The behavior of rock is not easily modified. A good rule of thumb is that if you are worried the distress blast may destroy the area, you have probably selected the appropriate level of destressing. A little bit of destressing is a waste of time, effort and money, and provides a false sense of security.

Ammonium Nitrate Explosives

The changeover from dynamite to ANFO was made for economic reasons. The fact that it greatly helps in the creation of a skin of failed material about an opening is a fortuitous circumstance. In the days of dynamite, where the costs of the explosives were charged out against development contracts, miners practiced cushion blasting without realizing they were doing so. Wooden spacers were generally used between dynamite sticks and tamping was limited to just above the primer at the hole toe and at the collar. The explosives were decoupled and it was common to see half the drill hole left intact.

ANFO changed that. In theory, the lower detonating velocity of ANFO should make it less damaging to opening walls than dynamite. In practice, it is far more damaging because there is 100% coupling with fully loaded holes. It does an excellent job crushing the borehole wall several hole radii out and in fracturing the rock in the vicinity of the borehole. ANFO has been most beneficial in helping to create the skin of failed

material needed for stable equilibrium conditions.

The approach of deliberately damaging the perimeter of openings where ground problems are being experienced is contrary to normal practices. There is a place to use controlled blasting to reduce damage to the rock around openings and it will even work at the transition depth where rockbursts are becoming a problem in development headings. Below some horizon, however, the skin around an opening must fail regardless of the care in blasting. Efforts to keep this skin intact may defer failure to a time when people are exposed to the consequences. In other words, controlled blasting will create hazardous conditions below some critical depth which in our case happens to be the 1220 m (4000 ft.) horizon.

RAISES

In 1946, an internal letter was written with a plea for a way to bore raises because of the hazards associated with driving conventional raises, timbered or otherwise, in rockburst prone ground. In the mid sixties, the letter was answered with the purchase of a Robbins 41R Raise Borer for Creighton Mine. There were many problems at the start, but through perseverance and the cooperation of the suppliers, we made raise boring work for hard rock mines.

SHAFTS

Number 6 Shaft was the first shaft sunk to below the 1220 m (4000 ft.) horizon. It started as a rectangular timbered shaft. Rockbursts, related to the presence of a steeply dipping trap dyke which followed the shaft down for the first 335 m (1100 ft.) of advance, made it necessary to convert over to a continuous concrete lining carried to within 1.5 m (5 ft.) of the bottom with each full faced round.

The next deep shaft was Creighton Number 8 which was collared at the 1525 m (5000 ft.) horizon and extended to below the 2010 m (6600 ft.) horizon. It was circular and concrete lined, with a finished inside diameter of 4.6 m (15 ft.). The walls were rockbolted and screened as the muck was removed and rounds were held to 1.6 m (5.25 ft.) to minimize the amount of open ground below the concrete which was kept close to the shaft bottom. The rounds were full faced and the bottom maintained in a dished shape.

At one point, where the shaft encountered mixed granites, short distress holes were drilled down below the shaft

bottom and blasted to try to overcome bottom bumping problems. The experiment stopped when conditions improved before the distressing experiment could be properly evaluated.

Further down, heaving and bumping in the shaft bottom again became a problem. Full face blasting was changed over to bench blasting with a marked improvement in bottom conditions which was attributed to benching.

The third shaft to be sunk below the 1220 m (4000 ft.) horizon was Creighton Number 9 which extends from surface down to below the 2130 m (7000 ft.) horizon. It is circular in cross section and fully lined with a continuous concrete lining a minimum of 0.3 m (1 ft.) thick. All sinking was done by benching.

Below 1220 m (4000 ft.), cushion blasting was tried in both the shaft and the 1280 m (4200 level) shaft station. Blasting in the shaft station was very well done and left a solid, intact back. But, the minute a hole was collared for a rockbolt, the back began to bang and snap. The experiment was not repeated at lower levels. In the shaft, the net effect of the cushion blasting was to increase the amount of overbreak as measured by the volumes of concrete poured. Cushion blasting was stopped and the amount of overbreak returned to normal.

There were a total of three rockbursts reported during sinking operations. The first two occurred just below the 1400 m (4600 ft.) horizon, one at a depth of 1411 m (4630 ft.) which caused injuries to five miners, with the worst injury a broken leg. The second occurred at a depth of 1435 m (4710 ft.) and two miners suffered contusions to their legs. Both rockbursts occurred near the end of screening and bolting operations and the displaced material came from the shaft wall. At the time the rockbursts occurred, shaft walls were screened and bolted at the end of the mucking cycle and the concrete lining was carried no closer than 3 m (10 ft.) from the shaft bottom. The shaft bottom had also begun to bump and heave.

As a result of the two rockbursts, support procedures were tightened up. Mucking operations stopped whenever the muckpile was lowered sufficiently for a new row of screen to be installed and the concrete lining was carried down as close to the bench as possible. Two of the bench holes with each bench were drilled to double the normal length and blasted as part of the bench blast to alleviate the bottom bumping condition.

The changes were effective in that there were no more injuries from rockbursts but at the expense of slowing up sinking operations. The one remaining rockburst damaged the station brow at the 1705 m (5600 ft.) horizon. The concrete was already in place when the rockburst was heard. No damage was observed until it came time to release the form for lowering in the shaft. Two of the pins which held the wedge section of the form were sheared and four others were bent. The concrete lining had failed in shear along a 3 m (10 ft.) long crack.

The last shaft to be sunk at Creighton Mine to below the 1220 m (4000 ft.) horizon was Number 11 Return Air Shaft. It extends from surface down to the 1770 m (5800 ft.) horizon and is circular with a continuous concrete lining which was kept within 7 m (23 ft.) of the shaft bottom. Spot bolting was used as a temporary support between the lining and the shaft bottom down to the 915 m (3000 ft.) horizon. Temporary support over the interval from 915 m to 1260 m (3000 to 4130 ft.) consisted of spot bolting and #9 wire welded screen where required. Below the 1260 m (4130 ft.) horizon, the crews began to experience problems with scaling the walls tight. The temporary support system was changed over to a continuous #4 welded wire mesh screen lining supported with mechanical anchored bolts and straps.

Bolt lengths of 1.2 and 1.8 m (4 and 6 ft.) were used depending on conditions.

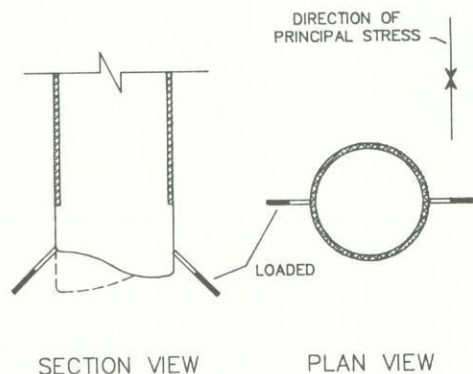


FIGURE 7. Destressing holes for Creighton No. 11 Shaft.

It had been observed in the rockbursts which affected the sinking of the number 9 shaft that there was a preferred orientation to the location of the damage.

Theory indicated the damage axis should be at right angles to the direction of the principal stress and subsequent stress measurements confirmed that this was the case.

It was decided to take advantage of this knowledge to locate destressing in the area of highest stress around the shaft perimeter. A single, 32 mm (1-1/4 inch) diameter, distress hole, 3.7 m (12 ft.) long, oriented at right angles to the direction of the principal stress and down at an angle of 45 degrees, was drilled with each bench. The bottom 1.8 m (6 ft.) was loaded and fired with the bench blast. It didn't work. Two holes were then tried for each bench blast, one to the NNW and the second to the SSE, figure 7. They worked. There was no need to carry the concrete lining down tight to the bench nor was there a need to interrupt mucking operations to install screen.

Bottom bumping occurred at times but was never severe enough to require distress holes in the bench.

ORE PASSES

Ore passes are a special problem in ground control. Unless they are supported before being put into operation, they will enlarge. The mechanism for enlargement at depth is the scaling action of the muck as it moves through the pass system. As stated earlier, stability requires that there be a skin of failed material around openings at depth where the stresses around the perimeter of an opening exceed the rock mass strength. In passes, as the skin of failed rock is scaled away, it is replaced by more failed material. The cycle continues until the pass is either abandoned, supported, or maintained by strictly enforced pulling limits. At times, the enlarging action will be a rockburst which may do damage to nearby openings. Where there are two parallel passes, the enlargement of one or both passes may rob the pillar separating the passes to the point where it will fail and the failure is liable to be in the form of a major rockburst. The ore pass size may also become sufficiently large to induce additional stress on a nearby opening which may also begin to experience rockburst activity. A system of parallel passes near Number 5 Shaft enlarged below the 915 m (3000 ft.) horizon and they had to be abandoned to protect the shaft. They were also the source of a number of rockbursts which damaged the shaft both before and after the passes were abandoned.

Ore passes normally enlarge to a long elliptical shape, with the long axis of the ellipse at right angles to the

direction of principal stress. It is not uncommon to have this axis measure 20 m (65 ft.) or more in length, with a short axis of less than 5 m (16 ft.). The size of the material in the pass does not seem to have a bearing on ore pass enlargement. We have had to install support in bins below crushers which enlarged once the steel and concrete liners wore away.

We have found that resin grouted fibreglass dowels provide the best support for passes. Rebars do not work as well. They vibrate when struck by passing muck and the grout bond breaks down. In an experiment in slusher trenches, where alternate drawpoint brows were fibreglass and rebar dowelled, brow loss in the fibreglassed drawpoints was at between one-half and one-third the rate experienced in the rebar dowelled drawpoints.

There is one very important rule for passes. Do not place them close to shafts. Use conveyors or some other form of haulage to bring ore to the shaft.

ROCKBURSTS IN STOPING OPERATIONS

Rockbursts during stoping operations are primarily pillar failure phenomenon. At depth, it is either uneconomical or impossible to subdivide an orebody into a series of stopes and pillars where the pillars will not fail before the stopes are fully mined. The nature of the failure (violent or yielding) is dependant on the width to height ratio of the pillar at the time of failure. In narrow orebodies, it is almost impossible to develop slender yielding pillars and rockbursts are common. Strict sequencing rules must be followed to restrict the quantity of ground at the threshold of failure. It is better to have many small rockbursts than one big one. In wide orebodies, it is possible to have most of the pillars yield rather than fail with rockbursts.

There are four types of pillars: ribs, crowns, retreat, and rock remnant pillars. Each type has either unique problems with solutions, or conditions which should be alleviated.

RIB PILLARS

Most of the orebodies at Creighton Mine are wide enough to be mined with transverse stopes separated by slender rib pillars, ranging in width from 4.2 m (14 ft.) to a maximum of 6.0 m (20 ft.). Stope widths are restricted to less than 9.0 m (30 ft.). Above the 1950 m (6400 ft.) horizon, rib pillars yielded during the course of mining the second to fourth

cuts when pillar width to height ratios were less than 0.5/1. Yielded pillars have sufficient residual strength to support the ground immediately over the stopes. The balance of the load is transferred to the abutments when pillars yield. The associated drop in local stress levels improves conditions in the stopes.

At the 2010 m (6600 ft.) horizon, rockbursts began to occur in rib pillars during silling operations when pillar width to height ratios were around 1/1. The depth had been reached where pillar stresses were closely matched to pillar strengths on the sill cut.

A system of destressing the pillars was developed during and for the silling of the stopes on the two mining levels below the 2010 m (6600 ft.) horizon. Figure 8 shows the locations of the pillar destress holes drilled and blasted in each pillar as the stope faces advance. The bottoms of the holes are charged and fired at an early stage during blasting of the face round which is taken to the full stope width.

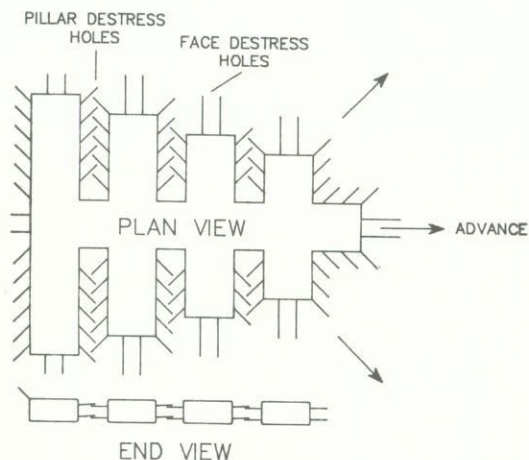


FIGURE 8. Rib pillar destressing during stope silling operations.

Pillar destressing did not stop the rockbursts, but it did reduce their intensities and caused most rockbursts to occur at or very shortly after blasting when no one was around. There is a condition here that can only be alleviated. A finite amount of convergence must take place between the back and floor for the excess pillar stress to be transferred to the abutments. It is this fixed amount of pillar shortening which forces wedges of material to be displaced out from the pillar walls,

with the quantity almost independent of whether the pillar goes off with a bang or just grunts.

Pillar walls are screened down to within 1 m (3 ft.) of the floor. Split Sets were found to be far more effective than mechanically anchored rockbolts in containing displaced pillar material.

One problem encountered with each new silling operation was convincing the miners that destressing the pillars would not unsettle the stope backs. In the early stages of silling, there would be no problems with the pillars and the miners could see no reason for destressing. Once the critical area had been silled, as evidenced by the first rockburst, the miners quickly became believers in pillar destressing and lost their fear of damaging the stope back.

Destress holes are also drilled and blasted in the advancing faces of the stopes in the same manner as is done in development headings. At the final perimeter of the silled unit, pillar destress holes are replaced by holes angled up into the perimeter corner.

Silling operations follow these rules to move out the zone of high abutment stress in a smooth and uniform manner:

1. Silling is to start adjacent to an existing mined area and advance away in a radiating pattern. Where no prior mining unit exists, silling is to radiate out from the starting point.
2. Silling will advance to common mining limits to avoid sharp breaks in the silling perimeter. The face of any single stope should not be more than 3 m (10 ft.) ahead or behind the adjacent stope faces.
3. Interconnecting slots should not be located along geological contacts or other structural weakness features.
4. Uneconomic zones within stoping blocks must be mined to avoid the creation of large rockburst prone pillars.
5. Blasts in critical areas should be taken at the end of the week to permit stress adjustments over the weekend.

CROWN PILLARS

Crown pillars are a major source of rockbursts in deep mining operations. Most of the energy release from the mining of a block of ore will be associated with

crown pillar failures. In narrow vein deposits, failure commonly takes place while the pillar width to height ratio is somewhere between 4/1 and 6/1 and failure is almost always by bursting unless heavy destressing is used effectively at the start of the critical crown pillar failure sequence.

Crown pillars have a difficult geometry in that they start with a fixed pillar height, the orebody thickness, and a wide width, the level interval. As mining advances upward, pillar stresses increase in an exponential manner while the pillar strength reduces more slowly in an almost straight line manner. A common characteristic of crown pillar rockbursts is the vertical motion of displaced material, though the direction of convergence is across the pillar.

Most of the rules of sequencing which we follow, were first published by R. G. K. Morrison in 1942 "3", and were directed towards limiting the volume of ground in the critical failure state. They were also directed primarily at the planar type of orebody. Figure 9 shows a typical inverted "v" sequence for transverse mining which limits the number of stopes at the critical failure elevation at any one time.

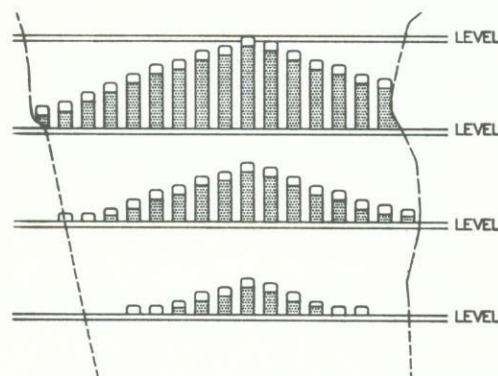


FIGURE 9. A typical inverted "v" sequence.

We have found that where the ore widths exceed a minimum dimension, which in our case is about 25 m (80 ft.), that the third dimension comes into play in crown pillar failures. The lead area of an inverted "v" sequence still experiences high stresses and rockbursts, but the dominant stress direction is along strike, rather than normal to the plane of the

orebody. Once the lead area is softened and the strike direction stresses cut off by several stopes mining through to the level above, the rockburst problem disappears. The remaining stopes can mine up through the crown pillar without initiating rockbursts. The ground may become heavy if tensile conditions develop.

With mechanized cut and fill mining, it became necessary to have four or more stopes at a common elevation as shown in figure 10. There were no problems in the lead area units where the footwall to hangingwall spans were in excess of 50 m (165 ft.). Our best guess, as indicated by a slight change in stress effects and the changeover of the stope backs from a dry to a damp condition is that the crown pillars yielded while the stopes were 27 m (90 ft.) below the level. There have been rockbursts in crown pillars with mechanized units, but only where the orebody thickness was less than 30 m (100 ft.) and the stopes had advanced to within 20 m (66 ft.) of the level.

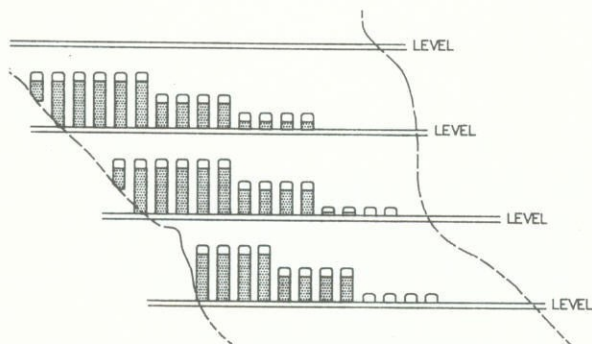


FIGURE 10. Modified sequence for Mechanized Cut and Fill Mining

The experience with crown pillars is similar to the experience with rib pillars. In wide ore areas, where crown pillars yielded while there was still some 90 feet of ground between the stope backs and the mining on the level above, the pillar width to height ratio was close to 0.5/1. Where the crown pillars failed with rockbursts, the pillar width to height ratios were about equal to or greater than 1/1 at the time of failure.

The reduced incidents of crown pillar rockbursts with increased ore widths, and the observation that the direction of

displacements in crown pillar rockbursts was primarily vertical provided an insight into the nature of crown pillar failures. To fail, the volume of material in the crown pillar must expand. It cannot expand from footwall to hangingwall because this is a direction of closure. It also has difficulty expanding along strike because this too may be a direction of closure. The areas free for expansion are up into the mined area above and down through the stoping unit below. The larger the lateral dimensions of the crown pillar, the easier it is for the crown pillar to dilate vertically up into the mined area above and down through the yielding pillars in the stoping unit.

For narrow vein situations, the crown pillar is a confined core pillar which is very resistant to lateral expansion and will take a high level of stress before failing. When failure occurs, it tends to be violent and with strong vertical motions as the confinement fails.

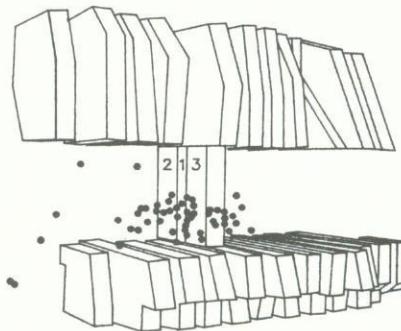


FIGURE 11. Perspective view of the slot area below the 2010 m (6600 ft.) horizon with 3 slices mined and showing the locations of the more significant seismic events associated with mining the third slice.

The conversion from cut and fill to VRM mining for the area between 2010 and 2070 (6600 and 6800) levels changed the crown pillar configuration back to a case where the early crown pillar failure mechanism would be in the form of one or more rockbursts. An early three dimension boundary element numerical model which could accommodate failure was used to model a footwall to hangingwall transverse slot. The model calculated the slot would release 90% of the energy contained by the crown. A second slot was modelled along

strike with mining to start from the primary slot. It contributed very little more towards energy release and was left out of the mining plan.

Most of the primary slot has been mined and the seismicity associated with slot mining was monitored with a 250-MP microseismic source location system which monitors the major mining areas. Figure 11 is a perspective view of the slot area and shows the first three panels of mining as well as the locations of the seismic events associated with mining of the third (hangingwall) slice. Most of the events occurred within two hours of the slot blasts and only those with energies equivalent to or greater than energies measured from nearby blasts of 2.3 kg (5 lbs.) of ANFO have been plotted. Most of the activity occurred as mining of the third slot approached the lower limit of the area where no seismicity was located. We had anticipated the presence of an envelope of failed material around the mining above the 2010 m (6600 ft.) horizon and the drill sill for the slot was taken just under the mining floor to take advantage of this possibility. The locations of the seismic events tend to confirm both the presence and the extent of the failure envelope.

REMNANT ROCK PILLARS

The problem with remnant rock pillars is in determining if the rock remnant is large enough to not fail. If it fails, the odds favour a major rockburst. The consequences of failure require a conservative approach and if there is any doubt as to the long term stability of a remnant pillar, a slot must be taken across the rocky area as part of the normal mining sequence. Slot mining is not without its hazards. Rock is usually more brittle than ore and the narrow width of a slot does not provide relief from high horizontal stresses. However, the hazards of not slotting out unstable rock remnants exceed the risks associated with slotting.

RETREATING PILLARS

There are times in the more planar orebody areas, where production goals can only be achieved by starting mining at several locations along the strike of the orebody. Mining must retreat towards mining. The problems and conditions influencing the behavior of retreating pillars are similar to those for crown pillars, a fixed height and a reducing width. The diagnosis is that at some stage, the pillar width will be reduced to the point where failure will occur and the odds favour rockbursts.

One solution is to mine the final pillar in a single blast. This requires that the pillar width be great enough to allow for the development of an expansion slot amounting to between 15 and 25 percent of the pillar volume without triggering pillar failure, and without increasing stress levels to the point where drilling becomes a problem. The balance of the pillar is then slashed into the expansion slot. If blast timing is fast enough, the blast will be over before convergence can do significant damage to the blastholes. We have some indication from blast monitoring studies, that pillar blast durations of less than 750 ms will achieve this purpose.

The benefits of blasting a pillar through the failure stage is that no one is around when the energy stored in the area by the pillar is released. A secondary benefit is that with less than 25 percent expansion room, the pillar area is stabilized by the tight filling action of the blasted ore. The experience of others is that pulling problems will not occur provided the expansion room exceeds 12 percent.

The problem with the final blast approach is one of determining how much pillar must be left so that development of the expansion slot does not cause the split pillar to fail. We are looking to numerical modelling for answers to this problem.

ROCKBURSTS AROUND AN OREBODY

A 16 geophone, 250-MP microseismic source location system was installed at Creighton Mine in the fall of 1980 to monitor mining below the 2010 m (6600 ft.) horizon. It was subsequently expanded to a 64 geophone system, with 48 active channels by the end of 1981 to study the seismic activity caused by mining below the 1645 m (5400 ft.) horizon.

Before the installation of the source location system, most unlocated rockbursts were assumed to have occurred out in the hangingwall, since damage was rarely identified in footwall openings. On occasion, there would be some evidence of damage found in footwall openings; however the amount of damage in many cases did not match the magnitude of the event as felt on surface. The microseismic source location system changed all that. More than 95% of all seismic events which occur outside the orebodies are located in the footwall rocks, with a large number of them associated with the quartz diorite dyke which is common to all of the orebodies. The locations may be 100's of metres back in the footwall.

The large Richter 4.0 event which occurred July 6, 1984, was an exception to the rule and occurred in the hangingwall and above the main mining area between the 1280 m (4200 ft.) and 1585 m (5200 ft.) horizons. It was believed to have been triggered by subsidence advancing upward and approaching a shear out in the hangingwall. Other rockbursts were triggered in the Number 6 Shaft pillar which was the last ore remaining to be mined in the area and this fact was clear from the pattern and extent of the damage in the mining area.

There is little that can be done to prevent rockbursts from occurring in the rocks around an area that has been mined and filled. The best that one can do is to install a microseismic source location system to identify active seismic structural features for the purpose of not locating key openings such as shafts and ore passes in potential rockburst areas.

SUMMARY

The key to mining at depth is to recognize that failure of the rock around openings and in pillars is both inevitable and desirable. Failure does not always have to be in the violent rockburst form. Rockbursts occur under conditions of high stress and high confinement. Efforts to fight failure increase confinement and stress. Except under marginal stress conditions the effort made to fight failure will succeed only in making the inevitable failure more violent and hazardous. Weakness and softness are the answers. Make pillars weak, with little lateral confinement so that they will fail under low levels of stress. If that will not work, look to making the pillar material soft by ground modification blasts. If neither approach is feasible, control the timing of the rockburst to coincide with or to follow a weekend blast.

It is easy to identify areas where failure will not be accompanied by rockbursts and it is easy to identify areas where failure can take no other form. There is, however, too large a grey area between the extremes where both the nature and timing of failure is uncertain. We have set out to develop and verify a three dimensional numerical model with plasticity to reduce the size of the grey area and to help us look into better ways of promoting failure. The microseismic system has not and is not expected to develop into a rockburst warning system. It is showing promise as a tool to monitor the growth of failure around mine openings for the purpose of checking numerical model predictions.

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PREVENTION, CONTROL AND MANAGEMENT OF COAL AND GAS OUTBURSTS

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ABSTRACT

Coal and gas outburst is one of the serious natural disasters in the coal mines of China. Since 1950, more than 10,000 coal or gas outbursts have occurred in more than 200 underground coal mines of different types.

The outburst prone coal mines are widely spread out. It occurs in different types of coal seams and is very serious in some areas. Most outbursts occurred at the time of roadway driving in coal seam, serious outbursts occurred mainly when entering the coal seam during the cross-cut driving; of the outbursts, coal and gas outbursts take up major proportion, while pressing out and pouring out also account for a large proportion; outbursts occur under different operating conditions, but mostly occur during shotfiring.

This paper introduces the prevention and control measures as practiced in China and illustrates the three types of outburst classification. Both technology and management are equally important in outburst prevention and control. Measurement of the volume of drilling cuttings and the initial gas outflow rate from boreholes have proven to be effective. A special team and organization must be set up to deal with the problems.

INTRODUCTION

General Status and Characteristics of Outburst

Coal and gas outburst is a complicated dynamic phenomenon and also a serious natural disaster in underground coal mines. This dynamic phenomenon has long been reported in various parts of the world, but not in our country. It is only after the founding of the People's Republic of China did the outburst occur and gradually become serious.

More than 10,000 outbursts have occurred in over 200 underground coal mines of different types in China since 1950. The number accounts for more than one third of the outbursts in the world. The most intensive outburst occurred in

SANHUEIYI COAL MINE of TIANFU COAL MINING ADMINISTRATION, location in CHONGQING area. There were 12780 tons of coal discharged and 1,400,000 m³ of gas emitted in that outburst. Now China has become one of the countries with the most serious outbursts in its underground mines.

The coal and gas outbursts occurred in our country have the following general features:

1. The distribution of outburst prone underground coal mines is very widespread, and outburst is more serious in some areas.

Outbursts have occurred in some underground coal mines in 21 provinces and autonomous regions in China. Among them, the outbursts were more serious in Sichuan, Hunan, Liaoning, Henan, Guizhou and Jiangxi provinces. The numbers of outbursts in those mines are not only large but the intensity is also high. The numbers of outburst prone underground coal mines and the total number of outbursts in Hunan Province account for one third of the total number in our country.

2. Outbursts distribution in different types of coal seams.

The coal seam occurrence is very complicated in the outburst prone coal mines in China. They occurred in single and groups of coal seams; steeply inclined, inclined and gently inclined coal seams, thin, medium-thick and thick coal seams. The outburst-prone coal mines with a single outburst prone seam or more than two minable seams account for about one third and one fourth of the total number of the coal mines, respectively. What is more, all of the minable coal seams in many of the outburst prone mines with multiple seams are of outburst-prone coal seams and with no release-seam to mine. Therefore, more than half of the outburst-prone coal mines in the whole country need to adopt outburst prevention and control measures in its own seam.

3. The initial outburst depth in the underground coal mines are different. The initial outburst occurs in shallower

depths in the coal fields in the southern parts of the country.

There is a big difference in the initial coal and gas outburst depth in different outburst prone underground coal mines due to the effects of geological conditions and other factors. The initial outbursts occurred in vertical depth of 500-600 m in some mines, whereas they were 50-60 m deep in other mines. A violent outburst occurred in one of the local mines in Hunan Province at a vertical depth of less than 100 m from the surface which discharged more than one thousand tons of coal.

4. Small and medium outbursts account for great majority while serious ones also occur many times.

According to the statistics and analysis at the end of 1981, small and medium outbursts with intensity of discharging less than 100 tons of coal account for 87.8% of the total number of outbursts. Though the large intensity outbursts which discharged more than 500 tons of coal, accounting for only 2%, their numbers of occurrence were as many as 100 times. The above mentioned statistics show that when selecting and applying prevention and control measures, the general and common measures should be directed toward the small and medium outbursts, but other effective measures for the high intensity outbursts should not be overlooked either.

5. Most of the outbursts occurred during driving in the coal seam, large intensive outbursts mainly occurred in cross-cut driving while entering the coal seam.

According to the same statistics and analysis mentioned above, among the total number of outbursts, those occurred in coal roadway, raise and dip account for 47.3%, 24.9% and 3.8% respectively, which summed up to 76%, accounting for three-fourth of that in coal roadway driving. Though the outbursts that occurred while entering the coal seam in crosscut driving only account for 5.85%, large intensity outbursts, especially those with more than 1000 tons coal discharged always occurred there. So special attention should be paid to the preventive measures of outburst in coal roadway driving and in entering the coal seam during crosscut driving.

6. Coal and gas outbursts account for great majority, while pressing out and pouring out also account for a large proportion.

Among the outbursts, there are different types. Outburst accounts for about one half, pressing out and pouring out each accounts for about one fourth, rock and gas outbursts account for a small proportion.

7. Outbursts occur under different operating conditions, but mostly occur during shotfiring.

Outbursts occurred in different operating conditions, some even occurred before operation started. Most outbursts occurred during shotfiring which account for 55.67% among those occurred under different operating conditions. It should be noted that, a large number of outbursts occurred during hand operations, accounting for 11.2%, among which one large intensity outburst occurred with 2500 tons of coal discharged.

Prevention and Control of Coal and Gas Outburst

For more than 30 years since the founding of new China, we have carried out long term research, both experimental and practical, in order to solve the problems of coal and gas outburst. The research has generated good results and established a set of measures for outburst prevention and control.

To mine a release seam adjacent to the burst-prone seam is the most effective and economic measure to prevent coal and gas outburst. It is also the major measure adopted in our country. Whenever it is possible, the outburst prone mines generally adopted the method of mining the release seam. At present, the selected release seam mining is carried out in coal seams with medium interburden, in small (7-8 m) and large (80 m) interburden (including the steeply inclined seams). All of the mining operations incorporate gas drainage. Through the practices of dozens of mines, we have come to the conclusion that the coal and gas outburst should basically be controlled whenever the method of mining the release seam is carefully adopted. What is more, we have succeeded in mining the extreme thin release seam by adopting the method of drilling to release the gas in some of the mines, extending the range of applications of the technique of the release seam mining.

Gas drainage before the mining of coal seams is also another major measure which have been tested and adopted in many mines. There are two ways for gas drainage: one is to drill holes through coal seams and the other is to drill a hole along the seams. Some mines have employed the process of gas drainage for about one year while others are even longer. All of those mines have achieved good results of coal and gas outburst prevention. Continuous practices and testings should be carried out to solve the problems of gas drainage in coal seams with low permeability and difficult drainage.

Injecting water to coal seam is another method which only tested and adopted in some mines with certain preventive effects.

There are many local preventive measures, such as multiple rows of drill holes, drilling and injection method (i.e. hydraulic drill hole injection), hydraulic injection (scouring), metal frame structure, pilot drilling (both large and small diameter holes), loose blasting (both deep and shallow holes), etc. Though shock blasting is not a preventive measure for outburst, it is still

widely used as a safety measure in outburst prone underground coal mines. Among the above mentioned measures, some are used in cross-cut when entering the coal seam, some are used for roadway driving in coal seam, some are used in working face, some are suitable for the coal seams with the risk of moderate outbursts, still some are suitable for the coal seams with the risk of serious outbursts, others can be adopted in key mines equipped with good technology and equipment, and still others can be adopted by the local small coal mines which lack equipment and technology.

There are two measures that we would like to introduce with particular emphasis:

1. Drilling and injection method. It is a new measure that we have tried and succeeded in the coal mines in our country. The procedures are as follows: Take the coal or rock pillars as a safety screen, drill holes in the seams where the risk of spontaneous outburst exists, and inject water with certain pressure. Through cutting and shocking by drill bit and hydraulic jet, the coal mass will be partially broken and the internal stress of coal and rock as well as unstable balance of gas will be disturbed and stimulated to release the outburst potential energy and cause the development of the gas pores. The gas pores will continuously be created with the advance of the drill hole. The coal, water and gas discharged from the drill hole will be transported to a place 400-500 m away from the working face for separation by a set of equipment consisting of sleeve, tee, jet pump, and transporting pipe. At the same time, the drilling set is equipped with a checking bit which can prevent the reversing flow of coal dust and gas in the pipe. Together with the restriction of the diameter of the hole and the protection of the coal pillar, the strength in the gas pore is confined. This not only insured the safety during drilling and injection, but also fully released the pressure and drained gas in a large area around the hole. In this way, outburst is prevented during mining and driving operations. The application of this method is very widespread. It can not only be adopted when entering the coal seam in cross-cut driving, but also be adopted in the working faces of coal roadway driving and coal mining. So there is a possibility of becoming a major method of prevention and control for the coal seams with risk of serious outburst.

2. Deep hole loose blasting. This method has been tested and applied in the coal mines in our country for quite a long time. But most still don't agree with each other on its function and effects for quite a long period of time. Some think it can prevent outbursts, but other think it can induce outbursts. After several years of special research and experiments, and through improvement and perfections of the technology, we have confirmed that this method can effectively prevent outbursts. When adopting this method, we usually drill 3-5 holes in the roadway with 42 mm diameter and 8-12 m long. Within the hole 5 m is served as pilot distance, which is not charged. Shortly after blasting, a strip (zone) of

pressure relief and gas drainage is formed 7 to 8 meters wide along the dip, and along the strike of coal seam of the length for the blasting section, which will have the potential energy of outburst released, and will ensure the safety in roadway driving. This method has a lot of advantages, such as simple in operation, needs less tools, convenient in transportation, economical and easy to popularize, etc. It is suitable for coal seams with the risk of widely distributed and large number of medium and small intensity outbursts. There are dozens of underground coal mines which are applying this measure, and the number is still increasing.

There is no standard for classification of outburst prevention measures in our country yet. But according to the actual practices, there are basically three types of classifications.

1. Classification according to the range of functions:
 - A. Regional measure -- including extraction of the release seam, gas drainage before mining of coal seam and water injection into coal mass.
 - B. Local -- all the measures except the above mentioned ones.
2. Classification according to the places where the measures are adopted:
 - A. In entering the coal seam during crosscut driving. The measures include drilling multiple rows of holes, hydraulic injection, drilling and injection, metal frame structure and shock blasting, etc.
 - B. Coal seam roadway driving. The measures include large and small diameter pilot holes, drilling and injection as well as deep hole loose blasting, etc.
 - C. Working face. The measures include pilot drill holes, drilling and injection as well as loose blasting, etc.
3. Classification according to the functions of the measures which are adopted in different seams.
 - A. Measures for adjacent seams -- mining the release seam.
 - B. Measures for outburst prone seam -- all measure that are directly applied in the seams with risk of outbursts.

In recent years, we have strengthened research for outburst prediction and forecast before mining emphasizing the geological factors, stress, gas and coal structures, etc. we have carried out underground research and laboratory tests, and regional prediction (forecast) and daily forecast. Preliminary results have been good and some methods have been applied in practical operations. In this way, it is only necessary to apply preventive measures in regions with outburst risk, not in regions without such risk. Therefore, it

greatly reduce the amount of work and time as well as costs.

At the same time, we have adopted effective measures in some outburst prone underground coal mines, which is checking the preventive measures by means of drill hole cuttings and the initial gas outflow rate from boreholes. Whenever it is found that the measures are ineffective, supplemental measures are adopted until the risk of outburst is eliminated.

The comprehensive application of the above mentioned measures not only have improved the capability of outbursts prevention and control of our country, but also enhanced the technology in this field to a new level.

Management of Coal and Gas Outburst

The long term practice has shown that strengthening the management of outburst prone coal mines is an important key link of coal and gas outburst prevention and control. Our practices in this area are as follows:

1. Setting up special organizations and working teams.

It is clearly stipulated in the Safety Regulations of Underground Coal Mines issued by the State that all the Mining Administrations and coal mines extracting outburst prone coal seams shall set up special organizations responsible for mastering (control) the trends and laws of the outburst, filling in cards of outburst, collecting information, summarizing experience and lessons, and formulating measures of outburst prevention.

What is more, all the mines have set up special working teams responsible for the operation and management of outburst prevention. The number of people involved can be from dozens to a hundred depending upon the distinct conditions of each mine.

To master the trends and prevention as well as control conditions of the coal and gas outburst of the whole country, we have set up a "National Archives of Coal and Gas Outburst" in Chongqing Research Institute of the Central Coal Mine Research Institute in accordance with the policy of the Ministry of Coal Industry. What is more, we have assigned special people to be in charge of the archives and carried out investigations for the basic conditions of outburst prone coal mines of all kinds. We have collected and sorted out all the information about coal and gas outbursts that have occurred over the years, summarized and compiled the related information and experiences. In addition to the above mentioned activities, we have participated in the preparation of some standards and methods, as well as the investigations of outbursts accidents.

2. Formulation and perfection of all kinds of regulations and systems.

The State issued the safety regulations for underground coal mines in 1951, and has since revised five times. In the recently issued 1986 revision of Safety Regulations of Underground Coal Mines, coal and gas outburst is listed in a separate chapter, and there are 33 clauses included. All the coal mining administrations and coal mines with outburst prone coal seams have formulated supplemental provisions, detailed rules and regulations, and are wholeheartedly implementing them.

3. Installing and equipping with different facilities and equipment.

Since most outburst prevention measures involve drilling, the outburst prone underground coal mines have been generally equipped with necessary drilling machines. Gas monitoring systems have been set up in more serious and key outburst prone coal mines, in order to effectively monitor the trends of gas outbursts. At the same time, gas drainage system, and hydraulic measures as well as other corresponding facilities and equipment have been set up and equipped with in different mines according to the measures adopted in each of them. The installation of the facilities and equipment have enhanced the capability of fighting against the outburst disasters of the mines.

4. Training of personnel of different types.

In order to meet the requirements of operations in the outburst prone mines, all the staff and workers will receive special training to master the basic knowledge of coal and gas outburst, become familiar with and master all kinds of outburst prevention and control measures, master the presage of outburst, become familiar with the operation of safety instruments, related rescue apparatus, as well as the escape route, etc.

At present, the Ministry of Coal Industry has its own Coal Mining Management College, related institutes and schools, as well as different safety educational centers that are in charge of the training for the staff engaged in management and the technicians by stages and in batches. All the workers will be trained by the mine itself. Through training, the quality of personnel of different types is enhanced.

5. Summarizing and popularizing the effective experiences.

We have accumulated abundant good experiences in practices of fighting against coal and gas outbursts in different coal mining administrations. If those experiences are summarized and popularized, it will in time push the work of outburst prevention and control forward in a whole. There are several ways of doing this: First, national and regional special meetings, experience exchange meetings, accident analysis meetings and academic seminars involved in coal and gas outburst are held nonperiodically. Secondly, articles are published in different magazines.

6. Formulating necessary standards and methods.

The dynamic phenomenon of coal and gas outburst is very complicated. What is more, there is quite a lot of underground coal mines where coal and gas outbursts have occurred, so it is necessary to have a unified understanding and management for the outburst phenomena in the underground coal mines. That is why we have primarily formulated the following two guides: "Standard for Outburst Classification" and "Identification Method for Outburst Prone Underground Coal Mines".

There are two methods for outburst classifications:

The first one is to classify according to the features of outburst dynamic phenomena. There are four types of outburst according to the features of the outburst phenomena appearing in the underground coal mines in our country:

1. Coal and gas outburst
2. Sudden coal pouring out
3. Sudden coal pressing out
4. Rock and gas outburst

The second one is to classify according to the intensity of outbursts. There are five types according to the amount of coal and rock discharged in each outburst:

1. Small outburst --- the amount of coal (rock) discharged is less than 50 tons.
2. Medium outburst --- the amount of coal (rock) discharged is from 50 tons to 100 tons.
3. Sub-large outburst --- the amount of coal (rock) discharged is from 100 tons to 500 tons.
4. Large outburst --- the amount of coal (rock) discharged is from 500 tons to 1000 tons.
5. Extremely large outburst --- the amount of coal (rock) discharged is more than 1000 tons.

Designation method for outburst prone coal mines:

Any mine showing signs of outburst phenomena during mining operations will be identified for the features of dynamic phenomena before it is designated as an outburst one. The designation will be carried out by a research unit designated by the Ministry of Coal Industry. Any mine that is to be designated so needs to obtain approval from the Coal Mining Administration to which it belongs. They then will formally apply to the research unit and provide all the technical information needed. The identification procedures are mainly for the outburst phenomena that occurs for the first time in the coal mines, or the outburst phenomena which has not been identified, but have appeared for many times. The features and types of dynamic phenomena should be defined so as to serve as a technical basis for the

determination of the nature of the outburst prone coal mine. Whenever it is defined to belong to one of the dynamic phenomena of "Classification of the Outburst Phenomena," it will be designated as an outburst prone mine. If the characteristics of the dynamic phenomena is not obvious, the identification method of comprehensive analysis can be adopted, i.e. comprehensive analysis and judgement are made based on the following parameters: geological factors, coal structures, gas desorption features, primary flow of gas emission and expansion, the solidity coefficient (f) of the coal, etc. What is more, redesignation can be made for the mines that were originally considered as an outburst ones, but with no outbursts occurring for a long time (usually about 5 years). Suggestions can be made to withdraw the original designation. The designation for and the withdrawal from an outburst coal mine will be approved by the Provincial (Autonomous regional) coal mining administration, and report to the Ministry of Coal Industry for the record.

What is more, according to the actual needs of the production management of the mines, we are now considering to adopt the method of underground coal mine and coal seam classification in accordance with the degree of risk of the outburst occurring in the coal seams, so that there will be some differences for the equipment installed and technical measures adopted, to realize the management according to different levels (degrees) of outburst risks and to rationally and effectively prevent and control the outbursts.

CONCLUSIONS

According to the long time practice of fighting against the coal and gas outburst in our country, we have come to the conclusion that to comprehensively and effectively prevent and control the outburst, it is quite necessary to do a good job in technology and management, for both of them are equally important.

Practices in many outburst mines have shown that outbursts have been better controlled in the mines that have set up and perfected the organizations and special teams, with good quality personnel, equipped with necessary facilities, formulated technical and safety measures for outburst prevention and control in accordance with the actual conditions of its own mine, with systematic measures for outburst prevention, continuously perfecting and enhancing the technology, and with all clear and definite parameters, formulated correspondence regulations and detailed provisions and rules with strict management. Otherwise, outburst will occur continuously.

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APPLICATIONS IN DESIGN FOR CLOSE PROXIMITY MULTI-SEAM MINING

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INTRODUCTION

Virtually all mineable Appalachian coal seams exist in a multi-seam environment. This makes it inevitable that most seams will eventually experience interaction induced ground control and mining problems. Indeed, it has been estimated that every working face will experience interaction effects from previous or current workings at least once during its lifetime (1), and that these problems are considerably exaggerated when seams are in close proximity -- a distance of less than 110 feet. The continued increase in multi-seam problems has resulted in considerable research activities over many years (2,3,4,1). Theoretical analysis, numerical and physical model studies, statistical and empirical analysis of numerous interaction related field data have been applied to multi-seam mining situations and have identified many useful trends and design criteria (4,5,6,7).

To fully utilize research results and transfer multi-seam mining technology to field engineers, a design model -- USEAM was constructed (8). This model, designed for close seam under-mining situations, was computerized to facilitate utilization. In the current research, a new computer program "MSEAM" was developed to produce a model for both under- and over-mining close seam situations. It is hoped that both of these software packages will aid field engineers in using important interactive design criteria to gain a better understanding of interaction mechanisms. Through advance planning with these programs, detrimental interaction may be avoided and minimized, and occasionally a design can be selected which will allow interaction effects to be utilized to the benefit of mining operations.

MULTIPLE SEAM DESIGN MODEL

Interaction Mechanisms

Analysis of field data has demonstrated that interaction problems occur due to one of four major ground control mechanisms: pillar load transfer, arching effect, upper seam subsidence and massive innerburden shearing (Table I). In the formulation of the model, all four interaction mechanisms were used to produce a totally comprehensive design package. To accomplish this, the interaction mechanisms were adapted and quantified to represent field conditions.

Pillar Load Transfer: The initial stress calculation under an upper seam remnant pillar or a gob line can be carried out based on analytical solutions for a classical 2-dimensional problem in the theory of elasticity -- distributed load on a semi-infinite half plane. However, the stresses calculated have to be modified to take into account innerburden layering (Figure 1).

Arching Effect: In the analysis of arching effect, one of the most important parameters is the maximum pressure arch width ($L'a$). Based on empirical analysis (10), the lower bound of this width ($L'a$) can be related to cover depth (H) by the equation:

$$L'a = 0.15H + 60 \quad [1]$$

Similarly, the upper limit of the maximum pressure arch ($L''a$) can be estimated by the equation:

$$L''a = 0.09H + 270 \quad [2]$$

If one assumes that the variation of data (Figure 2) is caused by the physical condition of rock strata, characterized by the percentage of sandstone or hard rock (IBSP), another empirical relationship can be derived:

Table I. Interaction Mechanisms.

Mechanisms	Applying Condition	Characteristics of Interaction Problems	Major Controlling Parameters
Pillar Load Transfer	Under-mining	Pillar failure; floor heave; roof falls in the area below upper seam remnant structure	Innerburden & its characteristics; Overburden; Old working layout
Arching Effect	Under-/Over-mining	Serious problems associated with abnormal load on pillars in lower or upper seam; distressed zones	Same as above plus caving conditions
Upper Seam Subsidence	Over-mining	Roof falls and associated problem in tensile zone created by lower seam workings	Lower seam height & old working layout
Innerburden Shearing	Over-/Under-mining	Caving or cracking of upper seam floor; Large scale roof falls in lower seam workings	Innerburden thickness and characteristics

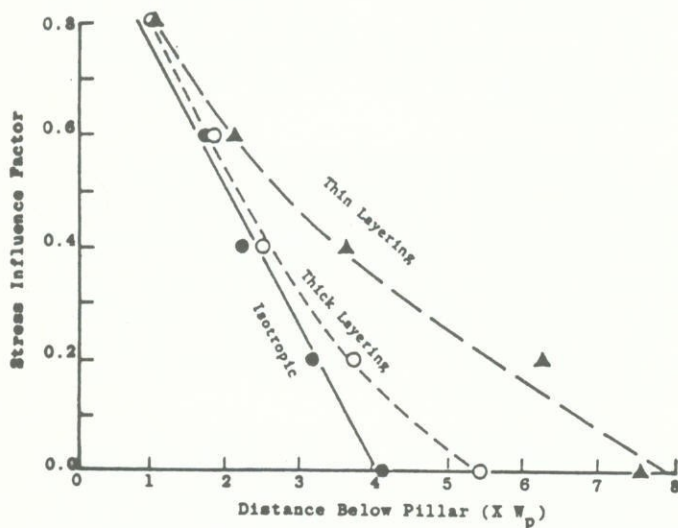


Figure 1. The Effect of Layering on Stress Influence Factor (9)

$$L_a = L'a + \text{IBSP}(L''a - L'a)/100 \quad [3]$$

Upper Seam Subsidence: This mechanism postulates that extraction of the lower seam causes the innerburden strata to cave, subside, and therefore results in damage to upper seam workings (Figure 3a). To use this mechanism in multi-seam mining design, the subsidence theory, originally developed for the surface over mined area, must be modified in order to predict the tensile zone in the upper seam level (Figure 3b).

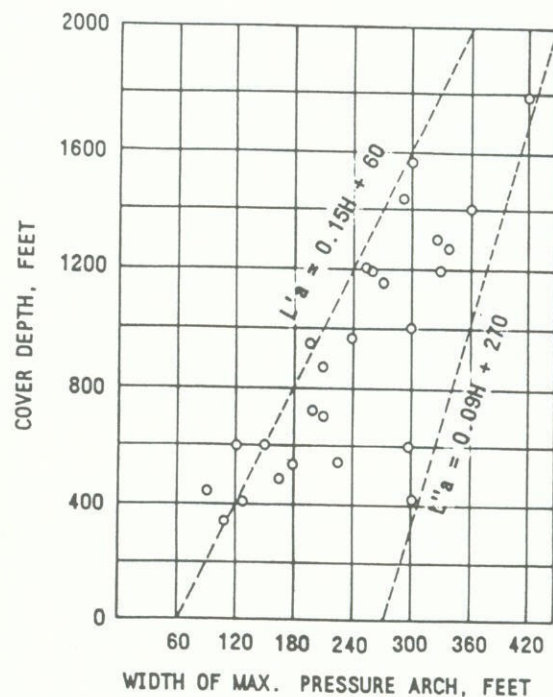
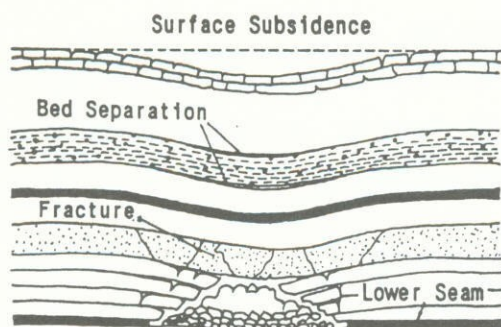
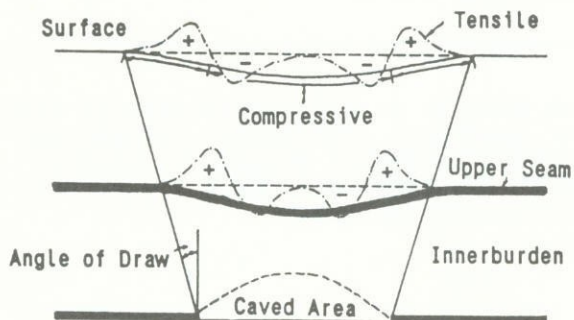


Figure 2. Relationship between Observed Widths of the Max. Pressure Arch and the Cover Depth (10)

Massive Innerburden Shearing: Shear stresses in the innerburden between two seams vary considerably depending upon the relative location and size of the workings of both seams. As shown by photoelastic study (Figure 4), in the case of mining over a remnant pillar, the maximum shear stresses (indicated by fringe orders) in the floor of the upper seam mine occur



a. Influence of Previous Mining to Overlying Strata and Upper Seam



b. Prediction of Tensile Zone at Upper Seam Level by Subsidence Theory

Figure 3. Using Subsidence Theory to Predict Possible Upper Seam Damage During Over-Mining

when the current working face is just about to leave the lower seam remnant pillar edge.

Factors Controlling Interaction

The major factors controlling interaction are shown in Table II (3,11). The fixed factors are defined as those which cannot be altered by mining design, while design factors are defined as those which can be thus altered. Both fixed and design factors are interrelated and must be considered simultaneously in order to assure formulation of a realistic mine design.

Design Method in Multi-Seam Mining

Independent interaction mechanisms can be used to explain some observed interactive phenomena between adjacent workings. However, the complexities of mining and geological conditions often dictate the use of more than one mechanism in explain-

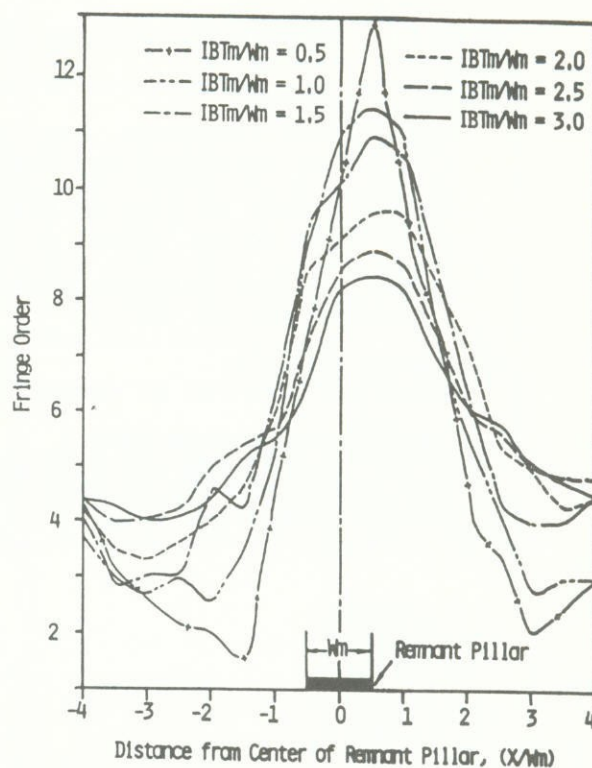


Figure 4. Comparison of Fringe Orders in a Specified Point in the Upper Seam Floor at Different Innerburden Thickness (IBT_m)

ing a specific problem. Therefore, in actual field applications, all possible interaction mechanisms must be considered in order to produce a most reliable design. In addition, relevant geological factors, such as abnormally weak zones or the presence of water, must be considered.

To fully utilize available research results and take into account all major variables contributing to interaction, an integrated design approach was used in this study. All factors and possible interaction mechanisms were considered and examined. Results from statistical, empirical and numerical analysis were incorporated or combined for a more accurate evaluation of potential interaction problems. Geological factors were designated by weighing indices following classification by an empirical method. The realization of this integrated design method and its application were greatly facilitated by the use of computer programs.

PROGRAM "MSEAM"

Program Construction

The MSEAM program contains two major options: the first for an initial analysis and the second for a detailed analysis where necessary (Figure 5).

Table II. Factors Controlling Interaction Effects.

A. Fixed Factors:	
-	Innerburden thickness, stratification, and physical characteristics;
-	Overburden thickness and physical characteristics;
-	Mechanical and physical properties of coal on both seams;
-	Seam (mining) heights of both seams;
-	Extraction ratio, layout of old workings;
-	Roof, pillar and floor condition in old workings;
-	Other abnormal geologic conditions (abnormal geologic zone, in-situ stress, etc.).
B. Design Factors:	
-	Relative location of current workings with respect to old ones;
-	Current extraction ratio;
-	Pillar dimensions and location;
-	Time delay between operations in two adjacent seams;
-	Mining direction.

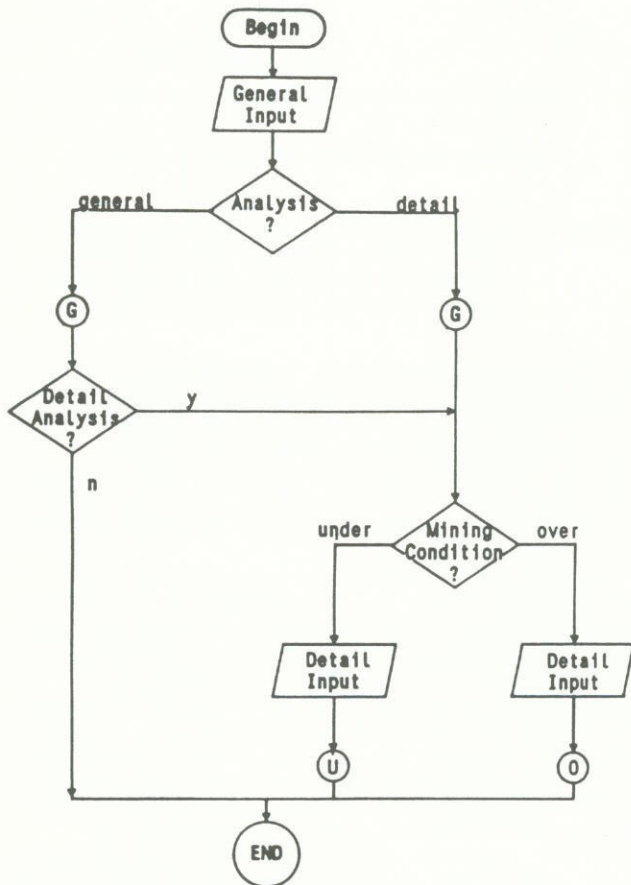


Figure 5. Macro Flow Chart of MSEAM Program

General Analysis: An initial prediction of possible interaction problems is made based on a limited amount of input data. The criteria and guidelines used for the prediction are developed from statistical and empirical analyses, and a general flowchart is shown in Figure 6. Through this first step analysis, the user deter-

mines whether or not interaction is likely under specified conditions. As another feature, regression models are used to predict the scale of damage due to the possible interaction.

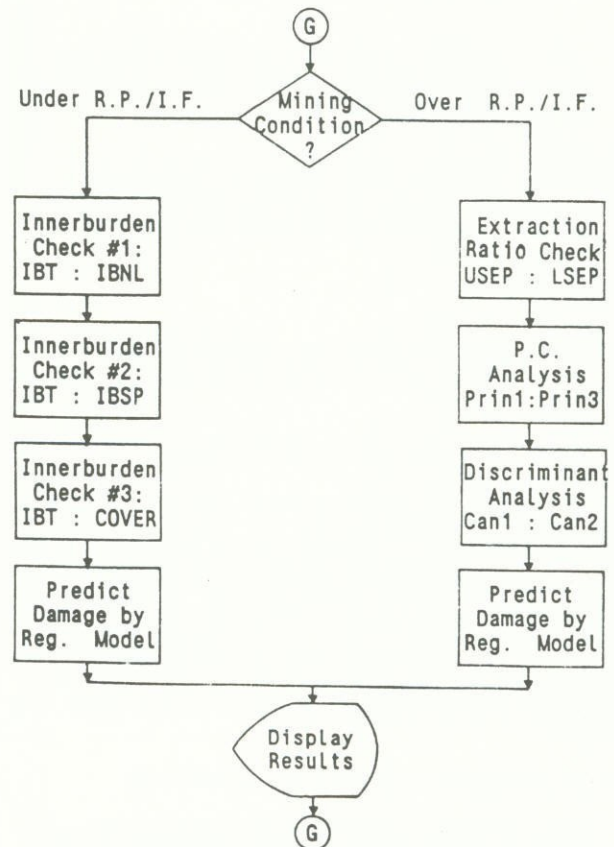


Figure 6. Macro Flow Chart for General Analysis of MSEAM program

Detailed Analysis: The detailed analysis can be further divided into two situations: under-mining or over-mining. Stress analyses based on different interaction mechanisms are carried out in order to estimate the magnitude and location of maximum stresses. Pillar dimensions are then computed using pillar design equations. Geologic factors that are not included in stress analysis are designated by indices according to an empirical classification scheme. This provides another independent estimation of possible interaction based on geological conditions.

For under-mining (Figures 7), the stress calculation under upper seam remnant pillar or a gob line is based on an analytic solution from the theory of elasticity and is then modified to take into account innerburden layering. Loading characteristics on the remnant pillar or a gob line in the upper seam are estimated from upper mine working layouts and caving conditions. Based on calculated stresses, the location of maximum stress in the lower seam as well as suggested pillar dimension are provided. For over-mining situations (Figure 8), different methods are used to conduct stress analyses since different interaction mechanisms are involved. For both situations, a separate prediction according to geological ratings is given. This has the potential for helping to identify cases where interaction is primarily induced by abnormal geologic settings.

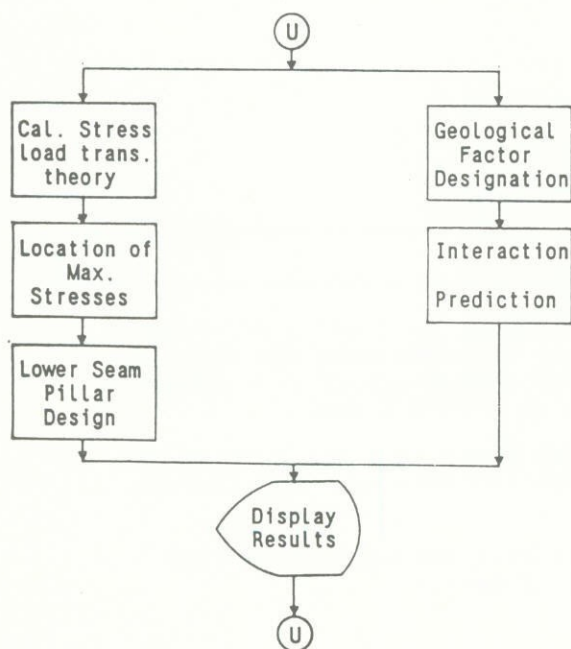


Figure 7. Macro Flow Chart of MSEAM Program for Under-Mining Conditions

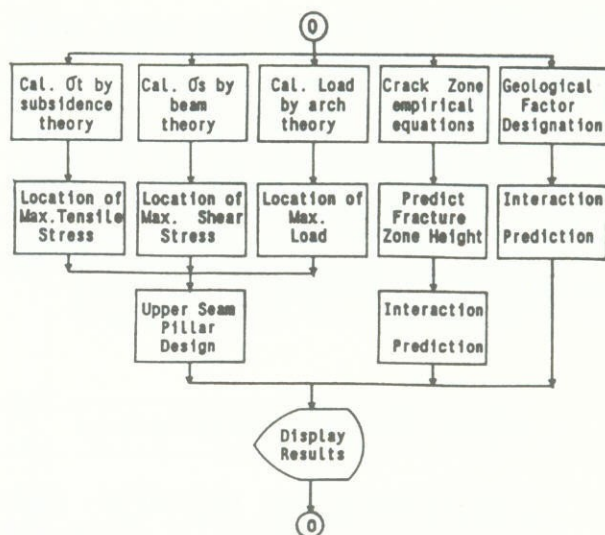


Figure 8. Macro Flow Chart of MSEAM Program for Over-Mining Conditions

Features of the Program

The MSEAM program is written in advanced BASIC language and was developed on an IBM personal computer. It can be used to predict possible interaction problems for the following four cases: mining above or below a remnant pillar or gob line left in upper seam; mining above or below a remnant pillar or gob line left in lower seam. It can conduct stress analyses under these conditions and suggest designs for room-and-pillar mines.

The program was developed with the end-user in mind and is fully interactive. Some user friendly features include: on-line help menus which define and give the range of all input variables; suggested input values which assist first time users by supplying a tentative value for some special variables; menu-driven input and output which allows users to easily change a specific input without re-entering others; extensive use of graphics to illustrate results and on-line help information. Another very useful feature is the ease in performing a parametric sensitivity analysis which is very useful for selecting alternative design.

APPLICATION OF THE PROGRAM

The usage of "MSEAM" program can be best described through example. The following two case studies clearly demonstrate the application of the program as a design tool in actual field conditions.

Case No. 1 (Under-mining)

This was a case in which the interac-

tion problems occurred in the lower Freeport coal-bed with severe floor heave and eventual pillar crushing and failure (12). The overburden above upper mine averaged 400 feet and the innerburden, composed of primarily interbedded shale, was 65 feet thick. Using these parameters and others as input, the program predicted that interaction problems were likely because the 65-foot innerburden failed to meet the requirements established by empirical rules (Figure 9a). By supplying more

input data (some of them assumed), a detailed analysis (Figure 9b, c) showed that a pillar of 93 feet in width might be required to support the concentrated load from upper mine. The actual pillar size was less than 50 feet. Considering other geological factors, the interaction experienced could also have been attributed to small delay between operation on both seams and incompetent innerburden (Figure 9d).

GENERAL ANALYSIS (undermining)		Output Menu	U 1/4
--------------------------------	--	-------------	-------

- Mine [Indiana Co., PA (Chekan 1986)]
- Stability prediction for LOWER [Lower Freeport] seam:
- Interaction problem [is] likely based on the evaluation of Innerburden Thickness and Number of Layers in it. The required innerburden for LOWER seam stability is [116] ft., the actual innerburden is [65] ft.
- Interaction problem [is] likely based on the evaluation of Innerburden Thickness and Percentage of Sandstone or Hard Rock in it. The required innerburden is [95] ft.; the actual innerburden is [65] ft.
- Interaction problem [is] likely based on the evaluation of Innerburden Thickness and Overburden Thickness above upper seam. The required innerburden is [95] ft.; the actual innerburden is [65] ft.
- Based on the result of a regression model, the degree of damage for the interaction is [4] (scale 0-5).*
- * See help menu for detail

PgUp/PgDn = new menu Home = main menu F1 = help

a. Output Menu for General Analysis

STRESS ANALYSIS (undermining)		Output Menu	U 2/4
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- Location of max. vertical stress in LOWER seam level is [46] ft. from the center line of remnant pillar (or gob line) in UPPER seam and the max. vertical stress is [742] psi. If a rectangular opening of [9]-feet wide is created, the stress can reach as high as [2819] psi.
- Location of max. principle stress in LOWER seam level is [46] ft. from the center line of remnant pillar (or gob line) in UPPER seam and the max. principle stress is [742] psi.
- At location you specified (x=[65], z=[20]; ft.) the vertical stress is [692] psi and principle stress is [779] psi.

b. Output Menu for Detailed Analysis (stress computation)

Figure 9. Results for Field Case #1 Shown on Output Menus of MSEAM Program

STRESS ANALYSIS (undermining)	Output Menu	U 3/4
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- Based on the average vertical stress at LOWER seam level [596] psi, the estimated coal strength [2100] psi, and selected safety factor of [1.0]; the protective pillar should be centered at the location of max. vertical stress with pillar width of:

[37] ft. (using Holland-Gaddy's pillar strength formular)
 [93] ft. (using Bieniawski's pillar strength formular)

c. Output Menu for Detailed Analysis (pillar design)

GEOLOGIC/MINING FACTOR ANALYSIS (undermining)	Output Menu	U 4/4
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- Based on the rating of geological and mining factors, the interaction between two seam [is] likely. The total rating number is [18] and can be attributed to the following factors:

	Rating
- Geological Weak Zone:	[2]
- Presence of Water:	[2]
- Innerburden Layering:	[3]
- Strong Rock in Innerburden:	[4]
- Mining Direction:	[2]
- Time Delay:	[5]

d. Output Menu for Detailed Analysis (geological factor analysis)

Figure 9. (Continued)

Case No. 2 (Over-mining)

In this case (12), current mining in the Sewickley seam was observed to be approximately 95 feet above a gob line of the lower Pittsburgh seam which was mined previously (Figure 10). The overburden above the upper seam was 500 feet in average and consisted predominantly of inter-bedded shale with some competent sandstone units. The mining height of the lower seam was 6 feet with full extraction in the panel. From the general analysis, interaction problems were predicted with a damage scale of 4 based on a regression model (Figure 11). This prediction compares well with field observations. In the upper mine, several roof falls were encountered, which can be explained by the upper seam subsidence mechanism.

CONCLUSIONS

Based on the research conducted to date the following conclusions have been drawn:

1. The complexities of mining and geological conditions often dictate the use of more than one mechanism in analyzing and quantifying specific interaction problems.

2. In dealing with interaction problems in multiple seam mining, an integrated design approach, which incorporates all possible interaction mechanisms and takes into account major geological factors, has been proven to be very successful.

3. Field validations of MSEAM program demonstrate its potential for fast, reproducible technology transfer in future multi-seam mine design.

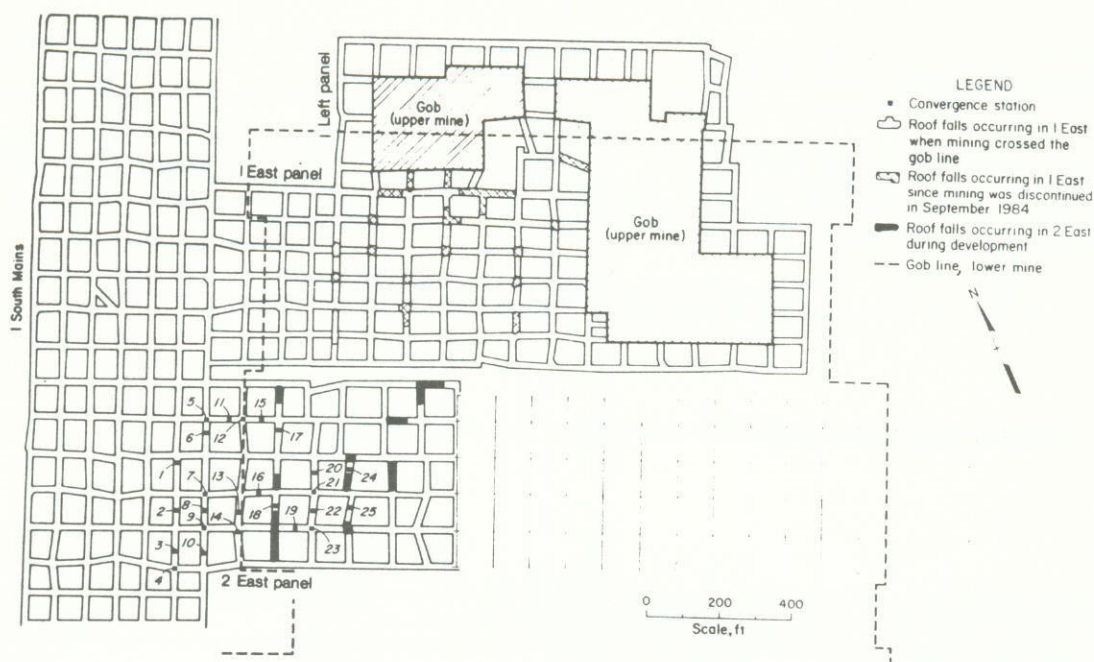


Figure 10. Location of Case Study #2, Showing Roof-falls in Upper Seam (12)

GENERAL ANALYSIS (overmining)		Output Menu
▪ Mine [Greene Co. PA (Chekan, 1986)] ▪ Stability prediction for UPPER [Sewickley Coalbed] seam: ▪ Interaction problem [is] likely based on the evaluation of Extraction Percentage on both seams. Based on the extraction of lower seam the upper seam extraction ratio should be [36] %; the actual ratio is [100] %. ▪ Interaction problem [is] likely based on the results of Pricipal Component Analysis. ▪ Interaction problem [is] likely based on the results of Cananical Discriminant Analysis. ▪ Based on the result of a regression model, the degree of damage for the interaction is [4] (scale 0-5).* * See help menu for detail		D 1/6
PgUp/PgDn = new menu Home = main menu F1 = help		

Figure 11. Results for Case Study #2 Shown on Output Menu of MSEAM Program

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GEOLOGIC CONDITIONS AFFECTING MINEABILITY IN THE JANE MINE
KEYSTONE COAL MINING COMPLEX
ARMSTRONG COUNTY, PENNSYLVANIA

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Rochester and Pittsburgh Coal Co.
Indiana, PA

ABSTRACT

Geologic conditions in eight northern operating sections of Jane Mine, Keystone Coal Mining Complex, Armstrong County, Pennsylvania are defined by utilizing drillhole data and underground geologic mapping. Three of the eight sections are experiencing difficult mining conditions in the Shelocta "D" coal seam (formerly the Lower Freeport). The five remaining productive sections will exhaust their reserves within one year. Coordinating mine planning with future production levels is aided by indicating areas that appear favorable and lack the geologic conditions that cause difficult mining. Roof rock lithologies, lithologic variations in and below the floor, jointing, overburden thickness, coal splits, paleochannels, and clay veins are the principal geologic factors believed to influence mining conditions in Jane Mine. Careful definition of each cause allows for a reasonable assessment of future mining conditions.

FULL TEXT WILL NOT BE AVAILABLE

ANALYSIS OF CUTTER ROOF USING THE BOUNDARY-ELEMENT METHOD

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ABSTRACT

This paper describes recent innovative analyses conducted by the Bureau of Mines to demonstrate the application of the boundary-element method in evaluating the effectiveness of caving chambers or sacrificial entries to relieve the stress conditions initiating cutter roof failure in underground coal mines. The analysis assumes that cutter roof failure results when the shear strength of the roof rock at an entry corner is exceeded by the applied shear stress. The applied shear stress is due to large differences between the horizontal and vertical in situ stresses known to exist in many coal mining districts experiencing cutter roof failure. The results of boundary-element analysis suggest that driving an arched lead entry in advance of adjacent rectangular entries and allowing it to cave significantly reduces the difference between the principal stresses at the corner of the rectangular entry, thereby lowering the maximum shear stress. Slippage of the immediate roof along horizontal bedding planes into the caved entry is shown to further reduce the stress conditions in adjacent entries. Practical applications of this stress control technique to longwall and room-and-pillar mining are discussed.

INTRODUCTION

Statistics published yearly by the U.S. Department of Labor's Mine Safety and Health Administration (1) indicate that between 1978 and 1985 roof falls in underground coal mines comprised 85% of fatalities involving ground control problems; and that these fatalities accounted for approximately 40% of all fatalities in underground coal mines. These figures suggest that, in spite of all the recent advances in roof support technology, more research is required.

The U.S. Department of the Interior's Bureau of Mines is actively involved in research to develop theoretical methods and design techniques to assist the mining industry in-

- solving basic ground control problems common to the mining industry; and
- designing optimum mine layouts to ensure safe, productive, and cost-efficient operations.

This research includes computer program and numerical modeling procedure development, application of numerical methods to structurally related mine design and rock mechanics problems, and field verification of the theoretical results.

One type of roof fall occurring in many U.S. coal mines is commonly referred to as "cutter roof." Cutter roof is a failure that initiates at one and/or both upper corners of an entry of rectangular cross section and propagates nearly vertically, resulting in massive roof collapse with little or no warning. What makes this type of failure particularly difficult to control is that, even though the mine roof is reinforced with conventional roof bolts, with time, massive roof collapse still results, usually to a height equal to the bolt length, or higher, as illustrated in figure 1 and photo 1.

Cutter roof problems are generally considered to be most prevalent in eastern coal mines, where the immediate roof is often comprised of thin shale laminae exposed to high regional tectonic stress fields. Coal mine operators mining coal in southwestern Pennsylvania, central to southern West Virginia, and the Illinois Basin frequently experience cutter roof problems in their main entries, room-and-pillar development entries, and longwall development entries.

However, evidence of cutter roof type behavior has also been noted in western coal mine regions. For example, results of stress relief overcoring conducted by

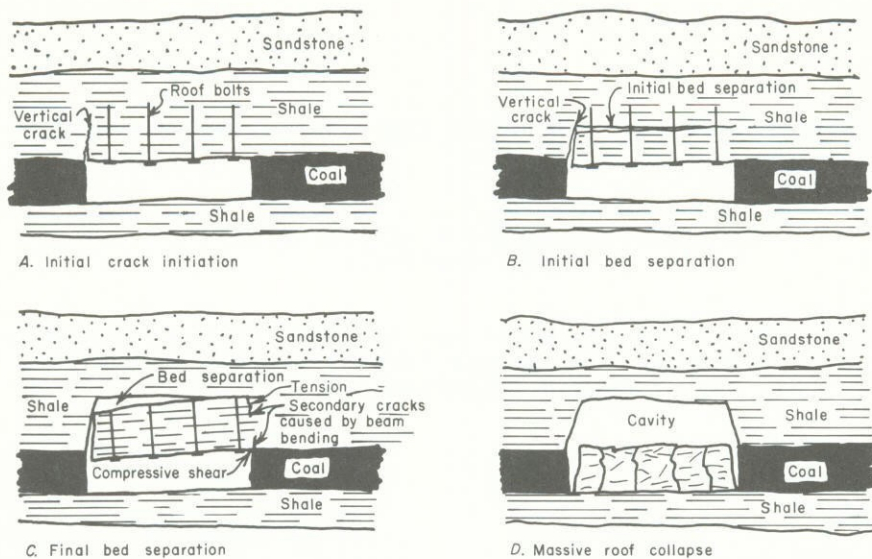


Figure 1. - Progressive sequence of events leading to overall roof collapse resulting from cutter roof failure

the Bureau at the Deserado Mine, Rangely, CO, indicate the presence of extremely high biaxial regional stresses. The values obtained using the Bureau's overcoring method compare very closely with values obtained by using hydraulic fracturing methods during an independent geologic investigation of the Rangely oilfield (2).

The objective of this paper is to illustrate how the boundary-element method can be used as a stress analysis tool in evaluating the effectiveness of stress control alternatives, such as caving

chambers or sacrificial entries, to reduce, eliminate, and/or control the initiation of cutter roof failure.

BACKGROUND

There continues to exist controversy regarding the exact causes of cutter roof failure. Past publications relating to understanding the causes of cutter roof behavior and methods for control are voluminous. Explanations have been based on geologic considerations (local and regional), stress aspects, and in-mine observations and experience. It is beyond the scope of this paper to list every reference dealing with cutter roof analysis. However, the following few are worthy of mention.

Hill and Bauer (3) attributed the increased occurrence of cutter roof failure in a central Pennsylvania coal mine to a high frequency of clastic dikes. Iannacchione, et al. (4), also observed through detailed mapping the influence of the direction of mining and occurrence of clastic dikes on cutter roof failure.

Peng (5) suggested that cutter roof is caused by the shear stress at the entry corners being larger than the shear strength. This high shear stress at the entry corners results from a large overburden weight and/or high horizontal stress at the pillar rib. However, high vertical and horizontal stresses are not the only reasons for cutter roof development. Other factors, such as local geology, large topographic relief, and relative stiffnesses between the floor,

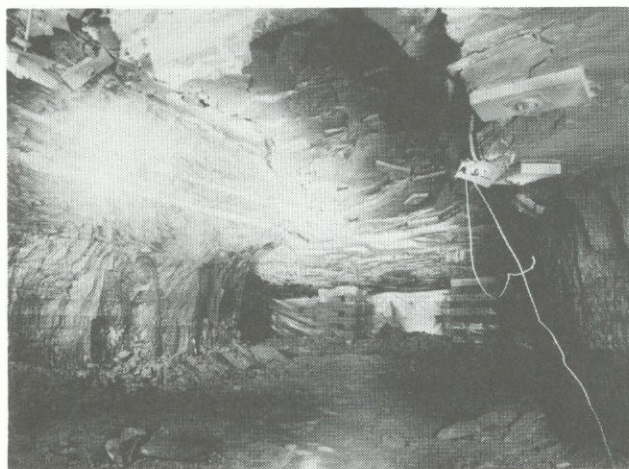


Photo 1. - Remaining cavity following overall roof collapse (Courtesy Bureau of Mines, Pittsburgh Research Center)

coal pillars, and immediate roof, will also contribute to the formation of cutter roof.

Aggson (6) showed, using two-dimensional finite-element analysis, that for a given set of stress conditions, the applied shear stresses at an upper corner of a typical entry in the Beckley coal seam could exceed the shear strength of the immediate roof material. Aggson also first illustrated, with two-dimensional finite-element analysis, the preferred direction and magnitude of the shear failure above the mine opening. Agapito, et al. (7), studied the effect of a high in situ horizontal stress field on mine roof stability and employed two-dimensional finite-element and boundary-element methods to investigate four alternative mining configurations. Su and Peng (8) studied three-dimensional effects of various stress fields on development of cutter roof and concluded that the intermediate principal stress, often overlooked in two-dimensional analysis, plays an important role in cutter roof formation.

Kripakov (9) summarized several past explanations relating the causes of cutter roof development to overburden stress, tectonic stress, lateral movement of the strata along bedding planes, and variations in the strength of individual rock layers comprising the immediate roof and gas pressure. Kripakov (9) also investigated the effect of various stress control measures with two-dimensional finite-element analysis.

Hanna, et al. (10), reports that a room-and-pillar mine in the Illinois Basin, experiencing severe intersection stability problems on development in an area of high horizontal stress, reduced the severity of roof falls by reorienting mine entries, modifying the bolting pattern, and altering the mining sequence. However, the frequency of roof falls was not significantly reduced.

Thomas (11) proposed, based solely on underground observations, that cutter roof generally initiates in geologic conditions where the immediate roof is relatively strong and is comprised of thinly laminated shale loaded by a weaker strata comprising the mine roof.

A recent Bureau publication by Hill (12) provides a comprehensive overview, with an extensive list of references, on past and present research relative to the causes and methods for control of cutter roof failure in underground coal mines.

The fact remains, however, when all is said and done, that the problem still

exists. Why? Probably because a truly systematic approach to reduce the severity of the problem has not been tried. All mines experiencing cutter roof are different to varying degrees. Potential solutions are often implemented in a "shotgun" or trial-and-error manner. Design changes, roof support methods, entry reorientation, mining sequence, etc., all have an impact on roof stability, but very often are implemented without a thorough understanding of the cause and effect of each change. What may work in one mine, for a particular set of site-specific conditions, may not necessarily work in another mine. Too often, the pressures imposed to meet production quotas override the time required for a mine's engineering staff to make a rational design decision based on a well-thought-out plan.

METHOD OF ANALYSIS

The technical approach proposed in this paper is based on the premise that cutter roof failure, on a mine-wide basis, results when the applied shear stress at an entry corner exceeds the shear strength of the immediate roof rock material, and that the value of the maximum shear stress, assuming plane strain conditions, is equal to one-half the difference of the principal stress values at every point in the vertical plane at any time near or surrounding a mine opening. These principal stresses can be due to pre-existing in situ stress conditions, mining-induced stresses created during the mining cycle, or some combination of both.

It follows that since "cutters" are inherently prone in mining districts where high biaxial stress fields exist, if the difference in principal stresses at an entry corner resulting from this high biaxial stress field can be decreased, then it is possible that the initiation of cutter roof failure can be reduced, eliminated, and/or controlled.

The two-dimensional boundary-element method is used to evaluate, through parametric analyses, the effectiveness of a caving chamber in terms of its ability to reduce the difference in principal stress components at an entry corner for a given set of geometric and in situ stress conditions.

Boundary-Element Method

The boundary-element method has become a well established numerical technique for determining the solution of boundary value problems in solid mechanics. Many practical rock mechanics problems related to mine design can be reduced to numerical

representations of boundary value problems. The boundary-element method requires that only the boundary be divided (discretized) into elements, as shown in figure 2B. This method significantly reduces the program input required compared with that for a finite-element model of a similar problem, where the entire rock mass must be discretized, as shown in figure 2A. A variation of the boundary-element method is called the displacement-discontinuity approach. This variation can be used to simulate a fault or slip plane in two dimensions, as shown in figure 2C.

The boundary-element method is based on the use of certain "singular solutions" as applied in elasticity problems. These singular solutions satisfy the governing partial differential equations that represent the system of interest; in mining applications, the system of interest is an opening in an elastic continuum (figure 2B). Therefore, it is not necessary to discretize the region surrounding the opening into a network of finite elements (figure 2A). It is only necessary to specify these singular solutions at each element along the boundary of the opening in such a manner as to meet the required stress or displacement boundary conditions. This leads to a system of equations that can be solved for the strength of each singularity along an element comprising the boundary of the opening. This type of approach is referred to as an indirect boundary-element method. First, the singularities satisfying the specified boundary conditions are solved for. Then, stresses and displacements are computed at each element along the boundary of the opening and at specified locations some distance from the opening in terms of the singular solutions.

The advantage of using the boundary-element method over the more popular finite-element and finite-difference methods is that the region surrounding the

opening (figure 2B) does not need to be discretized into a network of elements. This leads to a much smaller system of equations to solve. In addition, considerably less program input for a given problem is required. Infinite or semi-infinite problems are easily modeled using boundary elements, whereas with finite elements, an arbitrary external boundary must be specified and this boundary extended far enough away so that the applied conditions on it do not appreciably affect the solution near the region of interest surrounding an opening. Finally, the boundary-element method is much more versatile in modeling nonlinear slip along faults or bedding planes in two dimensions. This approach, called the displacement-discontinuity method, assumes that the crack or fault is divided into a series of N boundary elements, each of which is assumed to have a constant displacement-discontinuity. By knowing the analytical solution for a single, constant elemental displacement-discontinuity, the numerical solution to the problem is determined by summing the effects of all N elements.

The primary disadvantage of the boundary-element method is its limited application in modeling complex constitutive material behavior in the general rock mass. Most singular solutions on which the method is based only apply for linear elasticity, though some formulations exist for certain types of nonlinear material behavior. Even so, combining various boundary-element formulations together gives the method tremendous versatility in analyzing rock mechanics-related mine design problems. The reader is referred to Crouch, et al. (13), for a more in-depth explanation of the formulations and mathematics behind the different boundary elements available.

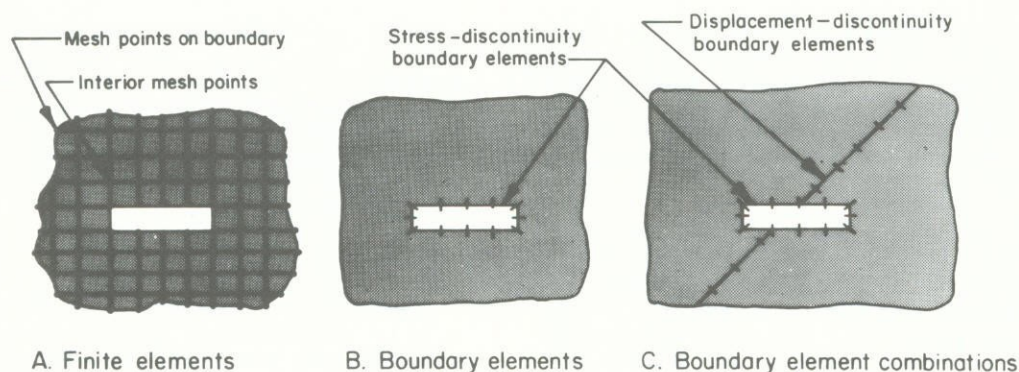


Figure 2. - Finite-element, boundary-element idealizations

Statement of Problem

It has been observed in many underground coal mines prone to cutter roof failure, that when an entry caves to some considerable height, adjacent entries often remain "stable". These observations have been made in mine sections where high horizontal stresses either have been measured or are suspected to exist. Many mine operators and research personnel believe that one reason for the stability of these adjacent entries is due to the shielding effect provided by the caved entry. In effect, a horizontal pressure arch is created that diverts the high horizontal stress away from the entry corners. In addition, it has been observed that when an arched entry is driven with a roadheading machine 100 ft ahead of adjacent rectangular entries, the bedded deposits comprising the immediate roof layers displace or slip laterally along the bedding planes into the void created by the arched opening mined with the roadheader. Both effects, a significant change in geometry and lateral movement of the immediate roof, are hypothesized to relieve the magnitude of the horizontal stress field locally near adjacent rectangular entries.

The specific problem addressed in this paper is to demonstrate how the boundary-element method can be utilized as an analysis tool to evaluate numerically the effects of a hypothetical arched, caved entry and the effects of slippage of the immediate roof into the caved entry on reducing the high shear stresses generated by an excessive horizontal stress field at the upper corners of an adjacent entry. This analysis is an offshoot of an actual case study in which an underground coal mine in West Virginia was adopting the use of a lead, arched middle entry in a three-entry system. Although the testing continued for only a few months and was halted by mine closure, nevertheless, improved roof conditions in the adjacent rectangular entries were observed. The use of an arched entry is not common practice in U.S. coal mines and would have to be based on the cost a mining company is willing to incur, since it would require specialized machinery and most likely a second face crew. Likewise, an arched entry is not necessary to induce roof caving. A mining company suffering from severe cutter roof may opt to allow caving in an entry that has not been arched, since one would expect a similar stress reduction in adjacent entries in the form of shielding and lateral slip along bedding planes.

Description of the Model

A two-dimensional boundary-element analysis under plane strain conditions was conducted. Figure 3 illustrates the two discretized boundary-element meshes used for the analysis. A three-entry system was modeled, taking symmetry into account. Figure 3A represents a typical standard three-entry configuration. Figure 3B represents two outside standard entries with a caved middle entry and slip planes along the immediate roof.

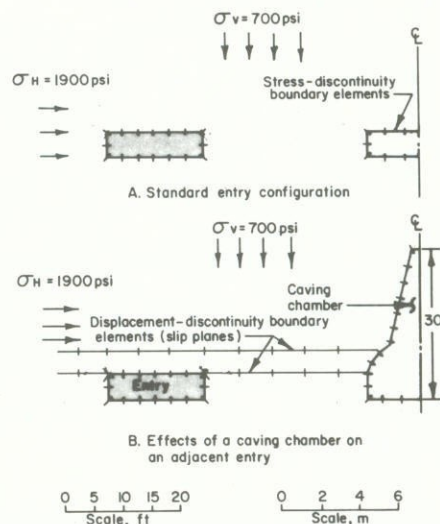


Figure 3. - Boundary-element model

Two different types of boundary elements were used. The fictitious stress or stress-discontinuity elements are utilized to model entries, or openings, in the rock mass. Displacement-discontinuity elements are used to simulate slippage along the horizontal joints, or bedding planes, representing the immediate roof. To simulate nonlinear slip behavior, the joint elements are loaded in an incremental manner, unlike in elastic behavior where the final loads are applied in one step, giving a unique solution. This is accomplished by incrementally reducing the normal and shear tractions along the boundary of the entries from their initial in situ values to zero. This procedure is described in more detail by Crouch, et al. (13). This type of loading can be thought of as a realistic simulation of the gradual mining of a long entry in an initially stressed rock mass along with the observed lateral displacement along bedding planes, as previously discussed. The joints, or slip planes, are assumed to behave according to a Mohr-Coulomb failure criterion. The normal and shear stress of a given joint element must lie on, or within, the failure envelope. If the applied stress state lies within the

envelope, elastic behavior is assumed. However, if the stress condition lies on the failure envelope, that particular element will be allowed to undergo a certain amount of inelastic deformation, or permanent slip, in the lateral direction. Once this occurs, the cohesion for that element is reduced to zero. A joint element also has the capability to crack open and subsequently reclose.

For this analysis, the pillar dimensions shown in figure 3 are assumed to be 30 ft, the entry width is equal to 17 ft, the seam height is equal to 5 1/2 ft, and the height of the caved entry is 30 ft. The length of each boundary element was equal to approximately 2 feet; therefore, a total of 150 elements were used for the problem shown in figure 3B. The model was loaded from infinity with horizontal and vertical in situ stresses equal to 1,900 psi and 700 psi, respectively. The rock mass was assumed to be a homogeneous, isotropic, linear-elastic solid and was assigned an elastic modulus of 1.0×10^6 psi and a Poisson's ratio of 0.25. The joints or bedding planes were initially given a cohesion value of 0.0, and a value of 10° for the angle of internal friction. The normal and shear stiffnesses of the joints were both assumed to be equal to 1.0×10^4 psi/inch.

RESULTS OF BOUNDARY-ELEMENT ANALYSIS

Figures 4A-D show a standard rectangular three-entry system subject to various initial stress conditions. The entries were taken to be 17 ft wide by 5 1/2 ft high, and separated by a 30-ft pillar. The purpose of this analysis was to show the effect of the initial state of stress on the orientation of the plane of maximum shear at points 2 ft above the outer entry corners. There are actually two planes of maximum shear oriented at orthogonal directions to each other. For the case where the horizontal stress is less than the vertical stress (figure 4A), one plane of maximum shear is oriented back over the pillar. For this case, a block of material formed by shear failure followed by bedding plane separation would largely be supported by the pillar, and thus would still present a stable roof condition. The other plane of maximum shear is oriented at a flat angle over the opening, and the weight of any loose material would be supported by conventional roof bolts. However, once the in situ horizontal stress equals or exceeds the vertical stress (figures 4B-D), one of the planes of maximum shear is inclined vertically or inward above the entry. Cutters developing in this direction may result in a massive roof failure when the failure propagates beyond the length of

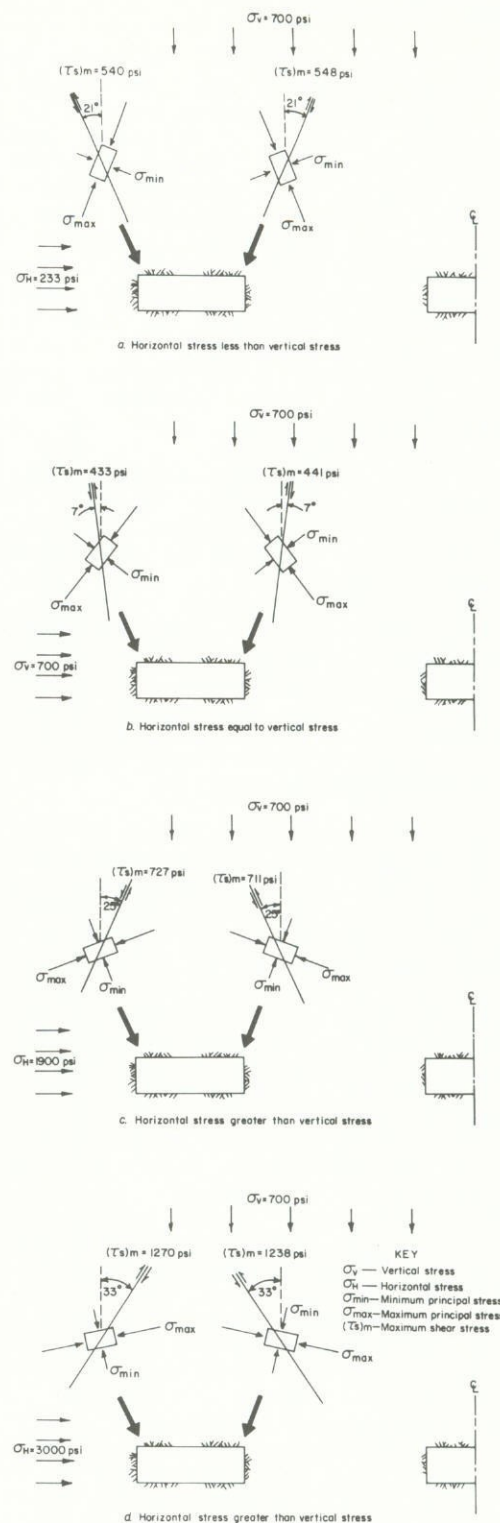


Figure 4. - Effect of in situ horizontal stress on the orientation of maximum shear stress at the entry corners

the roof bolts. Installation of cribs or steel arches may support or stabilize the roof at the expense of increasing costs and restricting the entry to vehicle travel and ventilation.

Figures 5A-D show the results of several computer runs in an attempt to relieve the stress conditions in the outer two entries, and thus control or eliminate the occurrence of cutter roof failure. The initial stress state shown in figure 4C of 1,900 psi horizontal and 700 psi vertical, which is common in many shallow eastern coal mines, was used for this analysis. Figure 5B shows the effect on the outer entry of driving a 13-ft-high arched middle entry and then allowing it to cave naturally to a height of 30 ft (figure 5C). Note that the arched entry configuration reduces the maximum shear stress at the inner corner of the adjacent entry by only 7%, while the effect on the outer corner is negligible. The introduction of a caving chamber has a more noticeable effect in shielding the adjacent entry, reducing the maximum shear stress at the inner and outer corners by 10% and 17%, respectively. It is assumed that this caved height is totally relieved of any confinement from the caved material, since initially an arched entry extended to a height of 13 ft. This assumption would most likely not be true if an arched entry was not driven, and the roof was allowed to cave from the seam height to the same height of 30 ft. Figure 5D shows how slippage along two horizontal joints or bedding planes in the immediate roof will reduce the maximum shear stress by 56% on the inner corner and 30% on the outer corner, compared with those given in figure 5A. This can be attributed to the release of elastic energy stored in the roof rock above the entries. These results assumed the slip planes had a friction angle of 10 degrees and were free of cohesion.

Figures 6B and 6C show the effect of increasing the pillar width to 40 ft and 50 ft, respectively. As expected, this increase in pillar width results in the caving chamber having less of an impact on shielding the high-stress conditions in the two outer entries.

Figures 7A-C show the results of gradually increasing the cohesion and angle of internal friction between the slip planes. Increasing these parameters results in a larger Mohr-Coulomb failure envelope, thus requiring a larger shear stress along the joints to initiate slip. If the cohesion and friction angle become high enough, as shown in figure 7C, the joints or bedding planes are essentially locked up from moving, and the stress relief at the entry corners can be

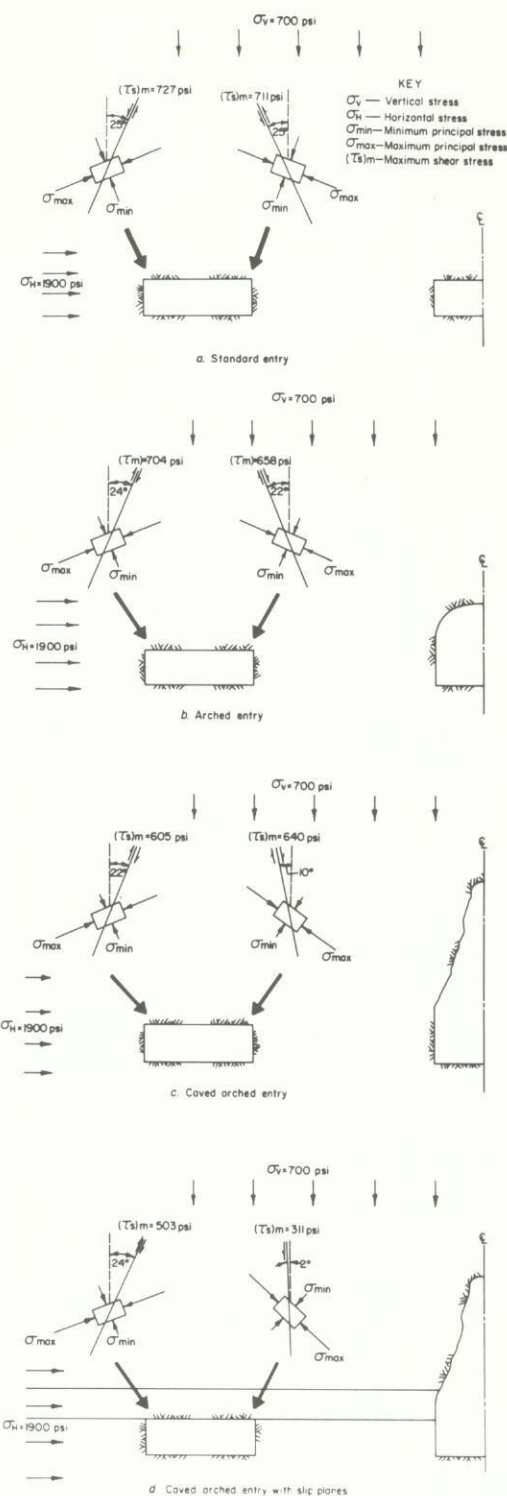


Figure 5. - Maximum shear stresses and orientations at entry corners

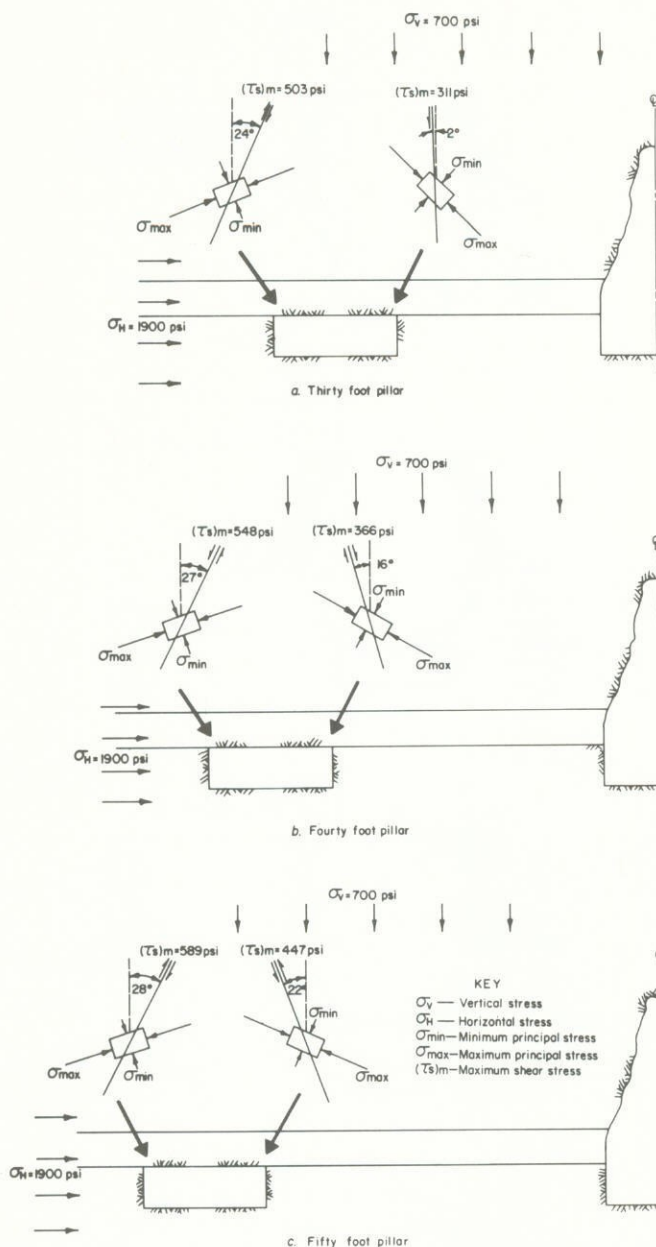


Figure 6. - Maximum shear stresses and orientations at corners due to varying pillar widths

attributed solely to the caving chamber.

Figures 8A-B show the influence of caving height on the entry corner stresses. Comparing figure 8A, having a 50-ft caving height, with figure 5C, having a 30-ft caving height, note that the maximum shear stress at the inner corner of the adjacent entry increases with the higher caving chamber, while the maximum shear stress on the outer corner decreases. Thus, a higher caving height does not appear to be of any benefit, since it results in a condition on the inner entry corner that is actually worse than what is

shown in the initial three-entry system of figure 5A. A 50-ft caving height would most likely not occur naturally or on a regular basis, and the assumption that this entire height be relieved of confinement would no longer be valid. This was only meant to be a hypothetical case to show that there is a certain height at which the caving chamber will cease to be beneficial. The optimum caving height

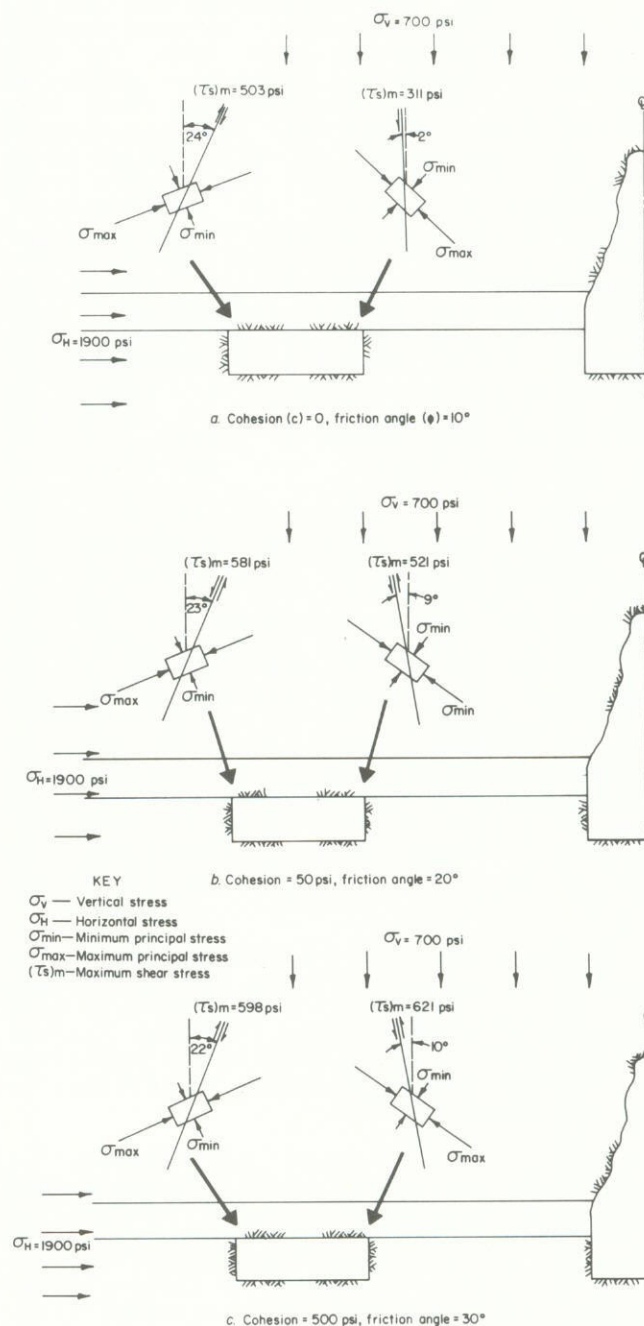


Figure 7. - Maximum shear stresses and orientations at entry corners due to varying joint cohesion and friction angle

depends on a number of variables, such as respective entry and pillar widths, and would have to be determined for each specific mining environment. The reality is that it may or may not be physically possible to control this height.

IMPLICATIONS OF ANALYSIS TOWARD PRACTICAL MINING RECOMMENDATIONS

Current room-and-pillar mining operations experiencing severe cutter roof problems may consider converting to longwall mining, thus limiting the amount of exposed roof area of mine entries subjected to this hazardous condition. In many areas in the East, where coal seams are mined under relatively shallow cover, mining-induced overall ground movement and surface subsidence are a critical issue. Uncontrolled subsidence can potentially damage underground aquifers supporting rich farm lands and cause surface damage in densely populated areas. Mine operators constrained to room-and-pillar mining under shallow cover subjected to high horizontal stress conditions may consider experimenting with the proposed concepts presented in this paper. One approach is to develop a panel with a multiple-entry system using narrow yielding pillars, advancing the middle entry first and allowing it to cave a substantial distance up into the main roof before mining the adjacent entries. The theoretical analysis presented earlier could be extended to a multiple-entry system to analyze whether or not a natural or an induced cave can potentially lower the shear stresses in the upper corners of the adjacent outside entries, and thus lower the probability of cutter roof occurring.

Longwall mining systems experiencing cutter roof failure resulting from high horizontal stresses can utilize three-entry gateroads on their panel development. The middle entry is driven in an arched configuration with a roadheader machine in advance of the adjacent two entries. Yielding steel sets with appropriate lagging are installed between the steel sets and the roof rock, such that the roof is allowed to cave. The roof is allowed to cave gradually onto the yielding arch support, thus relieving the stresses and protecting the outside entries for use as a headgate and tailgate for the longwall panels. The caved entry supported by the steel sets and lagging can still be used to provide a sufficient cross-sectional area for ventilation, transportation of personnel, equipment, and supplies, and secondary escapeways from both the headgate and tailgate sides of the longwall face.

A word of caution is necessary when considering the use of this type of analysis approach. The method assumes that the immediate roof is free of any local weak zones and acts as a continuum. In most actual mining conditions, the immediate roof is affected to varying degrees by local geologic anomalies, such

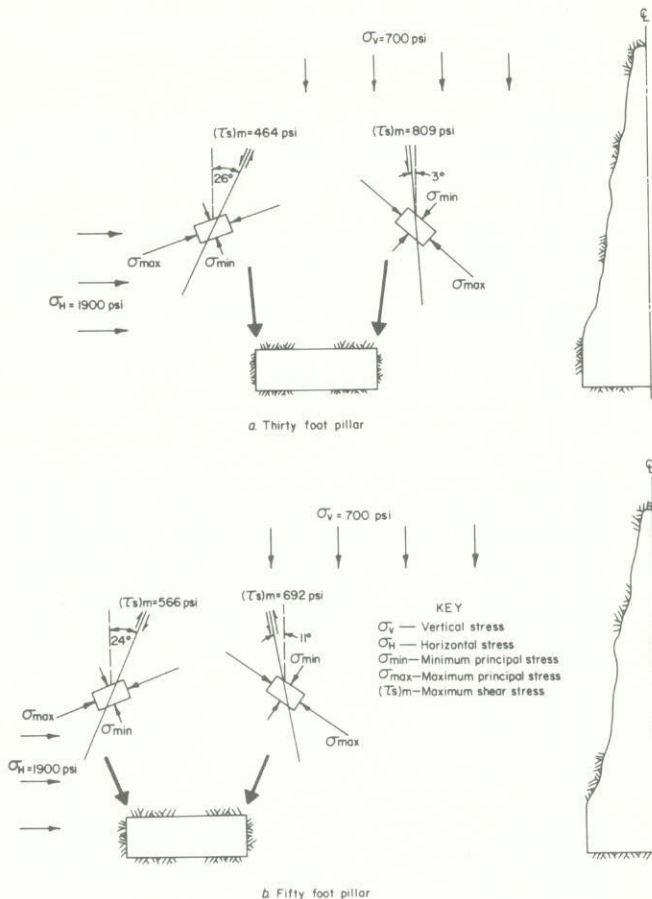


Figure 8. - Maximum shear stresses and orientations at entry corners due to a 50-foot caving chamber

The results presented above should be viewed with some degree of care. The boundary-element model used was only a simplified representation of an actual coal mine environment. Obviously, the assumption of an isotropic, linear elastic material is not truly correct. Nearly all coal mines consist of layered materials of different stiffnesses and exhibit some degree of non-linear material behavior in the form of pillar yielding. However, these features were not considered at this time to be as important as slippage along bedding planes above the entries. It is the intent of future research to investigate these types of material behavior and incorporate them into a better computer model.

as clastic dikes, kettlebottoms, slickensides, a rolling and/or dipping coal seam, and channel sandstones. In mines where these geologic features predominate, inherent weaknesses exist and, other than providing supplementary secondary support on an as-required basis, the practical reality is that it may not be possible to totally eliminate local roof failure. However, in mine sections where the immediate roof geology is comprised of relatively uniform strata (bedded deposits comprised of interbedded shales and sandstones) subjected to high horizontal stresses, the use of induced caving chambers or some other stress control technique to protect adjacent entries may have merit.

SUMMARY AND CONCLUSIONS

Cutter roof is probably due to a combination of many contributing factors acting together. These factors include the geology of the immediate roof and main roof, surface topography, dip of the seam, and the stress field surrounding an opening. The stresses surrounding the opening are continually changing. These stresses are a function of preexisting tectonics and additional stresses induced during the mining cycle. From an analysis point of view, it is very difficult to sort out the exact contribution of each effect because, on a minewide basis, conditions change.

The boundary-element method was used to analyze the formation of cutter roof solely from a stress analysis point of view. The analysis was based on the premise that if the applied shear stress at an entry corner exceeded the shear strength of the immediate roof rock, a failure would result. In a section of a mine where known potentially deleterious conditions exist, if the value of shear stress at an entry corner can be reduced by some external mining-induced mechanism, the initiation of cutter roof failure may be controlled. Boundary-element analyses indicate that in mine sections where the immediate roof is comprised of fairly uniform interbedded competent strata subjected to high horizontal stresses, use of caving chambers with slippage of the immediate roof can significantly reduce the stress conditions in adjacent entries. In areas of a mine where the immediate roof is consistently weak and contains numerous geologic anomalies, this type of stress control technique may not work well at all, and other support techniques, such as angle bolts, roof trusses, cribbing, and steel arches, may be necessary. The analysis presented in this paper shows that, even with a simplified model, the boundary-element method is a powerful tool

for parametric studies of different geometric configurations and mining conditions for improved mine safety.

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A NOVEL GROUND CONTROL PROGRAM AT PLATEAU MINING COMPANY

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ABSTRACT

A novel ground control program has been developed at Plateau Mining Company through five years of geotechnical investigations. The study was initiated to determine the causes for roof stability problems and to develop alternative support and monitoring systems for increased safety and productivity.

From systematic roof fall observations, extensive geologic mapping, roof drilling and monitoring ground movements along gate entries of four longwall panels, a sandstone channel system was detected and shown to impact roof stability. In particular, underlying mudstones up to seven feet thick experienced instability due to differential compaction and load transfer from the gob.

Roof bolter operators were trained to report roof lithology for mapping channel positions into the roof and for selective use of longer bolts. This boosted operator morale, and improved ground conditions. Monitoring rates of roof movement is used in these areas for detection of instability and for application of secondary support. This has significantly increased safety and productivity.

INTRODUCTION

The Star Point No. 2 Mine, operated by Plateau Mining Company (PMC), a subsidiary of Cyprus Coal, is located approximately 23 miles southwest of Price, Utah in the Wasatch Plateau Coalfield, as illustrated in Figure 1. The three mineable seams present on the property are, in descending order, the Wattis, Third and Hiawatha Seams, as illustrated in Figure 2.

Currently, three continuous miner units and one longwall are producing coal from the two uppermost seams. The longwall is operating in the Wattis Seam directly above the longwall recoverable reserves in the Third Seam and continuous miner recoverable

reserves in the Hiawatha Seam. Due to the fact that these reserves are superimposed and ground control problems were occurring in gate roads in the Wattis Seam, the management of PMC recognized the need for a geotechnical program to remedy gate road problems and to provide the optimum design of the longwall panel layout. The resultant program was conducted over a five year period by PMC, with assistance from J. F. T. Agapito & Associates, Inc. in designing field programs, evaluating results and designing alternative mine layouts, and with cooperation from the U. S. Bureau of Mines. This geotechnical program consisted of geologic investigation, instrumentation, monitoring and computer analysis. This paper presents a portion of this evaluation which resulted in a novel ground control program at PMC.

Statement of Problem

The objectives of this program were to determine the causes of roof instability problems which occurred in the Wattis Seam during the development and retreat of the first longwall panel and to design an alternative ground control program for correction of the instabilities.

The program began with careful and detailed documentation of every roof fall in an attempt to understand the nature of roof falls occurring in the gate roads. The information gathered pertaining to the falls included location, geometry, lithology, in-place roof support prior to the event, and any notable geologic anomalies that may have influenced the fall. With the growing body of data in place, initial correlations were made between fall locations, geologic features (i.e., surface lineaments, predominant joint orientations, roof lithologies, etc.) and roof support.

In conjunction with the roof fall documentation, an instrumentation/monitoring program was initiated. The purpose of this intensive portion of the program was to enhance the understanding of the geologic

features, support components and mining factors.

Geologic Setting

The mineable coal seams occur near the base of the Blackhawk Formation, and were formed as swamps lying between distributary river systems along the shore of an Upper Cretaceous Seaway. Peat accumulations were sufficient to form coal seams reaching 12 ft in thickness and averaging 8 feet.

As the shoreline retreated, the swamps were covered by a thin (approximately five feet) layer of lagoonal muds rich in plant debris. A system of post-swamp distributary streams meandered across the area scouring the lagoonal muds, and locally, the peat from the underlying swamp, replacing them with clean sand. This process formed long ribbons of lenticular (in cross section) sand bodies residing in a matrix of mudstone immediately overlying the Wattis Seam. Contacts between the lagoonal mudstone and the sandstone-filled distributary channels are very sharp. Prominent slickensides developed at the contacts due to differential compaction of dissimilar material. Abundant, randomly oriented, compactional slickensides also developed throughout the lagoonal mudstone, primarily between coalfield plant remains and the enclosing mudstone. The slickensides, together with a high clay content, make the mudstone a weak roof rock. The Wattis Seam roof contains more compactioned slickensides and a higher clay content than does roof rock in the Third and Hiawatha Seams.

Interburden between the Wattis and underlying Third Seam varies from 55 to 35 ft, and is composed of channel sandstone (in the thicker parts) and overbank mudstone and siltstone (in the thinner parts). Interburden between the Third and Hiawatha Seam is similar in composition to that between the Wattis and Third Seam, but varies in thickness from 70 to 50 feet.

The overburden is composed of slope forming lower coastal plain mudstones and siltstone overlain by a massive cliff forming fluvial sandstone sequence and bentonitic shales and thin freshwater limestone units. In the longwall panel area, the average overburden thickness is 1200 ft, but ranges from 1100 to 1500 feet. The overburden is deeply incised by drainages to form a rugged, mountainous terrain.

Major structural features in the longwall panel area include a N80°W fault-fracture system which has been preferentially infilled with phlogopite-rich intrusive material, forming dikes up to eight feet thick. Joints from the most prominent to the least prominent are N60°E,

N-S, N60°W and N80°W. Surface lineaments have been identified from multi-platform aerial photos; however, their relationship to structural or depositional features remains unclear.

ELEMENTS OF GROUND CONTROL PROGRAM

A novel ground control program was developed through evaluation of roof instabilities, classification of roof conditions, an assessment of primary support performance, the design of alternative primary support systems, and the detection/reinforcement of potentially unstable roof.

Evaluation of Roof Instabilities

Early in the investigation, a correlation between local geology and ground stability was suspected. Initially, the surface lineaments, particularly lineament intersections, were believed to be the locus of ground instability. However, as the program continued, it became apparent that any correlation between surface lineaments and underground roof falls was largely coincidental (Figure 3). This conclusion led to a search for other geologic features that may be influencing roof falls.

By integrating lithologic data from mapping of the immediate roof, investigating roof falls, observing caves behind the shields, and recording roof cuttings, an extensive system of post-swamp channels was detected several feet above the roof, cutting the gate roads at locations shown in Figure 3. A strong correlation is established between sandstone channel distance into the roof, areas of increased joint density, and roof falls. Correlations between roof fall locations and channel margins are evident in Figure 3.

Sandstone channels impacted roof stability both during the development and retreat due to the differential compaction of underlying mudstones and by load transfer from the gob to the gate pillars. This created shear movements at the sandstone/mudstone contact, reduced the effectiveness of the roof bolts, and caused roof falls. This is illustrated in Figure 4, which shows the entry closure at several locations along the first longwall gate road. As the longwall face approached a channel position, load transfer to the gate pillars increased, resulting in an increase in the entry closure and the occurrence of roof falls behind the face.

There is also a good correlation between increased roof joint frequency and the distance of sandstone channels above the roof, as illustrated in Figure 5. Since

most roof falls occur where channels are five to seven feet above the coal seam, these correlations are being used to locate sandstone channels above the roof in areas where detailed roof lithology data is not available.

As the body of detailed roof data grew, identifying the character or type of channel at a specific location became possible. Current data show that there is a continuum of types which can be loosely categorized into five typical ones, as illustrated in Figure 6. The data also indicate a very strong correlation between low angle channel margins and roof falls, and between dimples in the base of the channels and roof falls.

Classification of Roof Conditions

From knowledge gained during the geologic investigation and instrumentation/monitoring of gate roads, the next logical step appeared to be a roof classification system of gate roads on advance. At present, the primary targets of this classification are the detection of sandstone channels overlying five to seven feet of mudstone, characterization of the nature of the sandstone channel (Figure 6), and joint mapping. This information permits prediction of channel locations in adjacent entry advances.

To predict the position of the sandstone channels in the roof during the development, the roof bolter operators were trained to report roof lithology at each mining cut to the ground control group. The ground control group comprises a foreman responsible for roof control and the engineering personnel responsible for the sandstone channel/geologic projections. The roof lithology data is processed continuously to locate channels in the roof and to identify zones with less than 5 to 7 ft of underlying mudstones. Figure 7 shows areas where 0 to 5 ft of mudstone are identified. Roof zones with up to seven feet of mudstone were shown to experience stability problems, possibly because of the interaction between sandstone channel contacts and the bolt anchorage horizon.

The classification should eventually expand to include a wetness indicator and joint densities on advance. Areas of roof then could be assigned a stability value(s) considering the above mentioned parameters and others found to influence ground control.

Evaluation of Primary Support

In conjunction with the geologic investigation, an evaluation of primary support systems began. The initial step was to determine if roof falls occurred less frequently in areas supported with resin

bolts as opposed to mechanical bolts. From the roof fall data available, it was determined that four and five foot resin bolts were no more effective than four and five foot mechanical bolts for the support of up to seven feet of mudstones underlying the sand channels.

At this point, a large-scale mechanical bolt evaluation study was undertaken to demonstrate the overall effectiveness of mechanical bolts (1). The study was conducted in cooperation with the U. S. Bureau of Mines, and involved instrumentation of over 800 ft of entry in a long-wall gate road.

The study indicated a very low tension-torque ratio of approximately 22 and a low bolt installation tension of approximately 4000 lbs (2). This low tension was attributed to improper bolt installation practice and hardware, and in some cases, an inadequate bolt anchorage strength. Accordingly, a two-part solution was implemented. First, roof bolters were trained to install bolts properly by: reducing thrust (thus reducing friction) during bolt installation, decreasing bolt bending to less than five degrees (3), using a hardened washer at the bolt head to reduce friction, and recognizing inadequate or poor anchorage horizons from the drill speed and cuttings.

Secondly, different systems were tried to enhance areas of poor anchorage strength for the effective use of mechanical bolts. These systems included six inch long shells and quick setting resin cartridges. Neither system appeared to improve anchorage substantially in a weak mudstone horizon. Therefore, these areas were recognized as requiring either longer bolts to reach an area of sufficient anchorage strength or fully grouted resin bolts.

Due to training and hardware changes, the tension of bolts at installation doubled to nearly 8400 pounds. Roof sag was then compared between areas with low bolt installation tension to areas of higher bolt installation tension. On an average, sag was reduced by 43 percent in areas of similar lithologies, as illustrated in Figure 8. This resulted in more effective use of mechanical bolts to inhibit roof sag and increase the stability of a two foot thick caprock.

Since neither five foot long resin nor mechanical bolts were effective in providing long-term support for the underlying mudstones, other alternative primary support systems were evaluated for these specific areas.

Alternative Primary Support Systems

Two alternative support systems are being evaluated for reinforcement of

IMPLEMENTATION OF THE PROGRAM

unstable roof underlying sandstone channels. One is the use of eight foot long resin/mechanical or combination bolts to prevent bolt anchorage from coinciding with the five foot mudstone/sandstone contact horizon. The second is the use of truss bolts to reinforce the roof adjacent to the pillars and to provide lateral confinement for the unstable roof; the trusses have shown promise in reinforcing weak roof, but evaluations currently are limited to retreat mining, as reported by Maleki et al. (4).

Current evaluations have shown improvements in roof control for the underlying mudstones where long bolts were used. Both eight foot long mechanical and combination bolts have proven to be effective in stabilizing the roof in the bleeder entries where large pillars are used. Long, fully grouted resin bolts, however, are preferred for longwall gate roads utilizing yield pillars because of the better compatibility of their load/deformation characteristics with that of the roof.

In summary, either five foot long mechanical or resin bolts are used for thick mudstone roof unless a weak layer exists at the bolt anchorage horizon where resin bolts are used. The long bolts are used for ground control in the vicinity of the channels where the roof is composed of five to seven feet of mudstones.

Detection of Roof Instabilities and Use of Secondary Support

Extensive monitoring adjacent to the sandstone channels has shown that the long bolts are effective in prolonging the life of the entries. Monitoring the roof movements also has proven to be beneficial in assessing the stability and for allocation of supplementary support. An excellent correlation between the rate of roof movement and the occurrence of roof falls has been established at the mine, as illustrated in Figure 9. From 115 measurements, 15 had rates greater than 0.025 in./day; at all these 15 locations, roof falls occurred between two to eight days after detection of this critical rate. This has made it possible to detect development of roof instability. Roof lithology consisting of 0 to 7 ft of mudstones overlying sandstone channels has contributed to the high rates of roof movement and the occurrence of roof falls. This is illustrated in Figure 10, showing the position of roof falls which occurred behind the longwall face in conjunction with roof lithology and rate of movement. From this figure, it can be seen that a critical rate of roof movement of 0.025 in./day is a reliable indicator of the development of stability problems at PMC.

The implementation of a ground control program has become possible at PMC with the objectives of increased safety and increased productivity with reduced costs. This program requires input and cooperation throughout PMC's organization to be successful. The input begins with the roof bolters using the training they have received to detect and report all roof lithologies to their immediate foreman. The foreman recommends appropriate primary support and oversees that the support is installed properly.

The foreman then reports roof lithology data to the geologist, who uses the data to refine and predict channel locations. As a follow-up, all operating sections are mapped. This provides an opportunity for the interchange of ideas on ground conditions between the geologists and face crews.

When potentially unstable ground is suspected due to the presence of sandstone channels, structural features, or direct observations, sagmeters are installed. Regular monitoring of these meters is required to detect when and if the critical rate of sag develops. Detection of the critical rate of sag is immediately reported by engineering to the mine foreman for determination of the need for secondary support.

The objectives of detecting unstable ground before a problem occurs and effectively using the primary support installed are obtainable only with involvement from all facets of the PMC organization in ground control. Meeting these objectives will result in increased safety and productivity, which equates to a lower cost product necessary to remain competitive in today's market.

CONCLUSIONS

The geotechnical program has brought geologic influences and ground control to a better understanding at PMC. The points clearly brought out through the program are:

- o Mudstone roof of five to seven feet in thickness overlain by sandstone channels causes the majority of roof control problems at PMC.
- o Mechanical bolts are adequate for the majority of roof conditions encountered at PMC given that proper installation procedures are used.
- o Neither mechanical nor resin bolts of four to five feet length will provide long-term support for five

to seven feet of mudstone under channel sandstone. Eight foot bolts as a minimum are required.

- o There is a critical rate of sag for the Wattis roof that indicates a roof fall will occur unless remedial action is taken.

Through this program, bolter operators became involved in improving ground conditions, boosting their morale. Furthermore, safety was significantly increased by the ability to detect unstable roof during development. Productivity was increased by using special support at potentially unstable positions during development, minimizing the need for subsequent reinforcement/cleanup.

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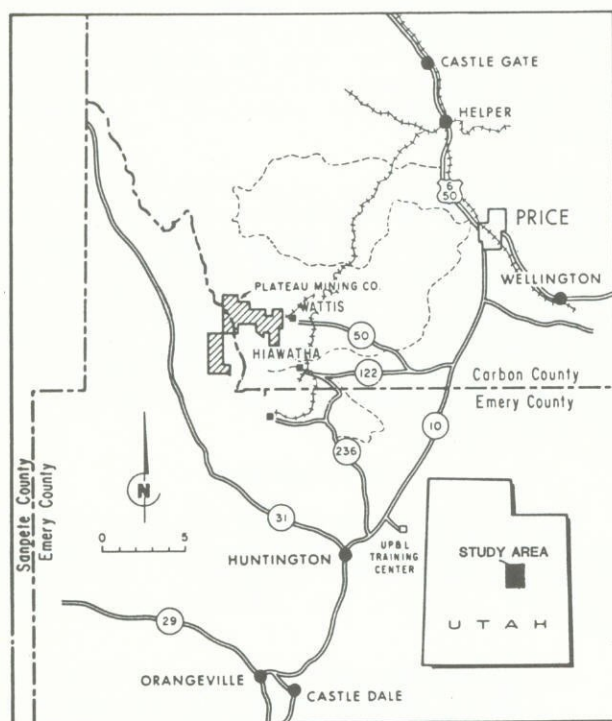


Figure 1 Mine Location Map

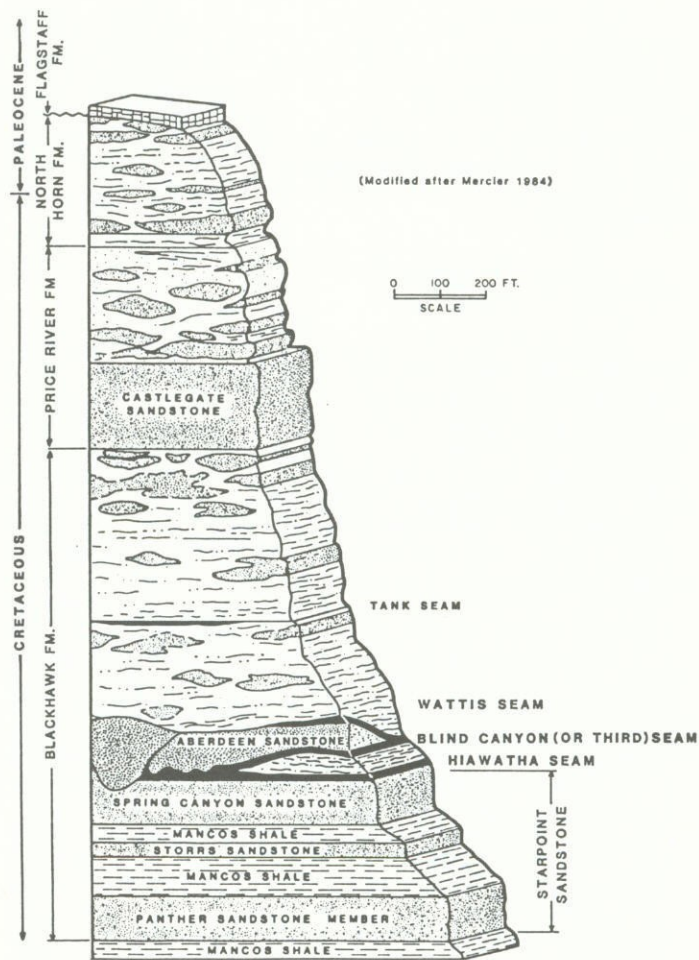


Figure 2 Generalized Stratigraphic Column

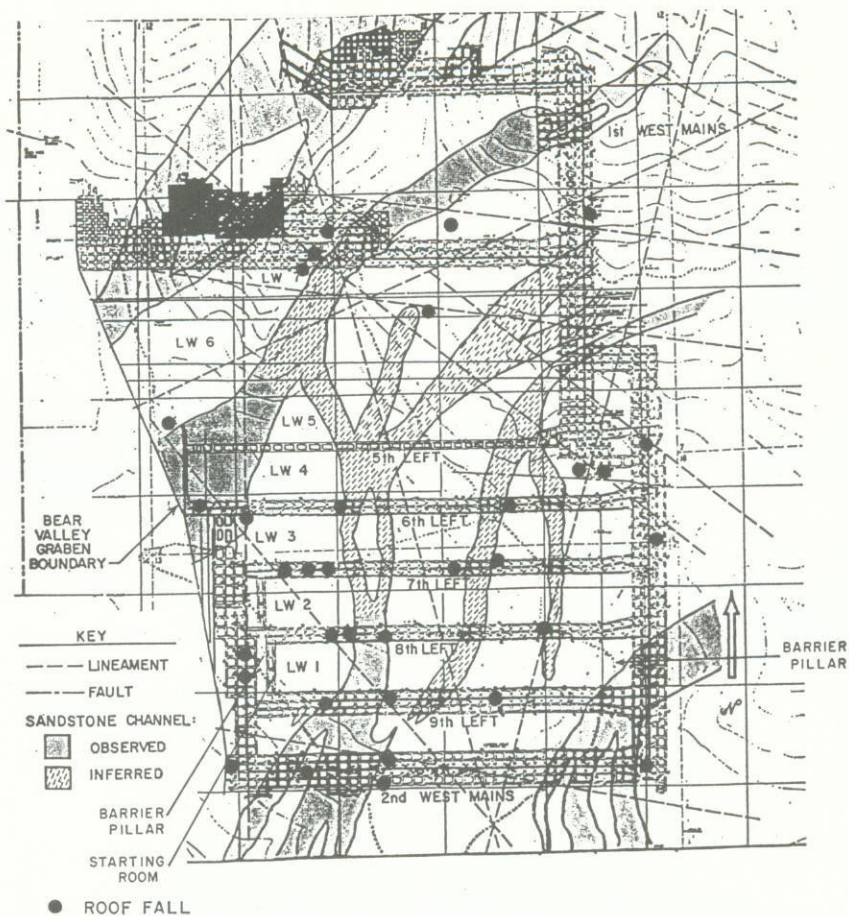


Figure 3 Sandstone Channel, Lineament and Roof Fall Location Map

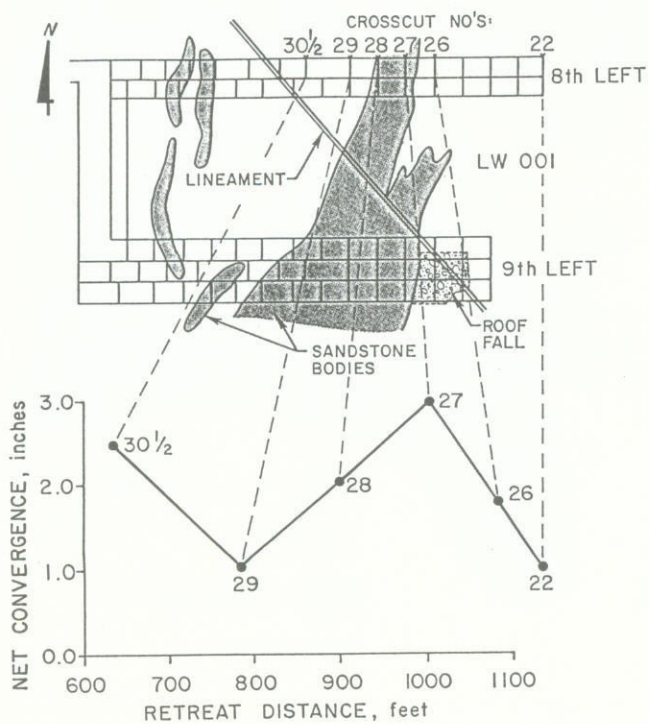


Figure 4 Cyclic Loading as Affected by Sandstone Channels

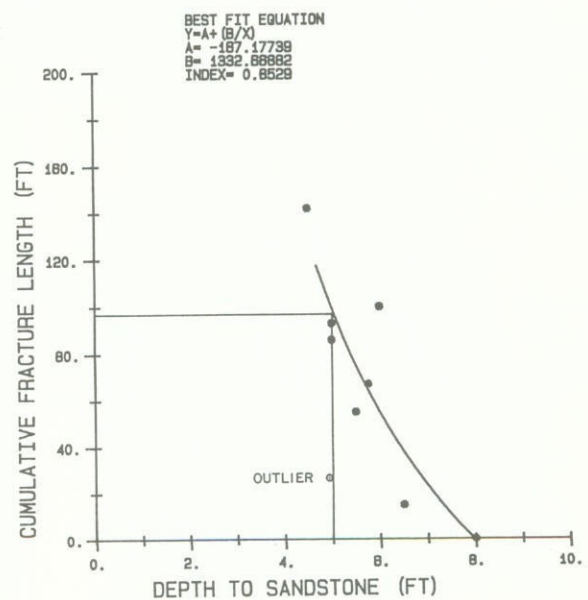
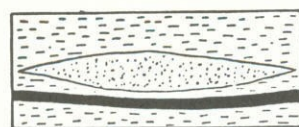
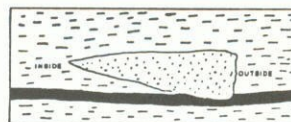


Figure 5 Best Fit Curve for Total Fracture Length and Mudstone Depth to Sandstone Member



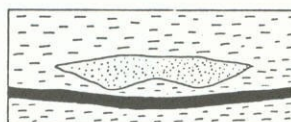
TAPERED (Low Angle)
STRAIGHT REACH



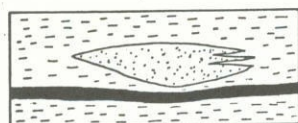
CUTBANK-POINTBAR
(Outside-Inside)
BEND



ABRUPT (High Angle)
STRAIGHT REACH



ISLAND OR DIMPLE
STRAIGHT REACH OR BEND



INTERMEDIATE
(Moderate Angle)
WITH FEATHER EDGE
STRAIGHT REACH

Figure 6 Typical Sandstone Channel Characterization

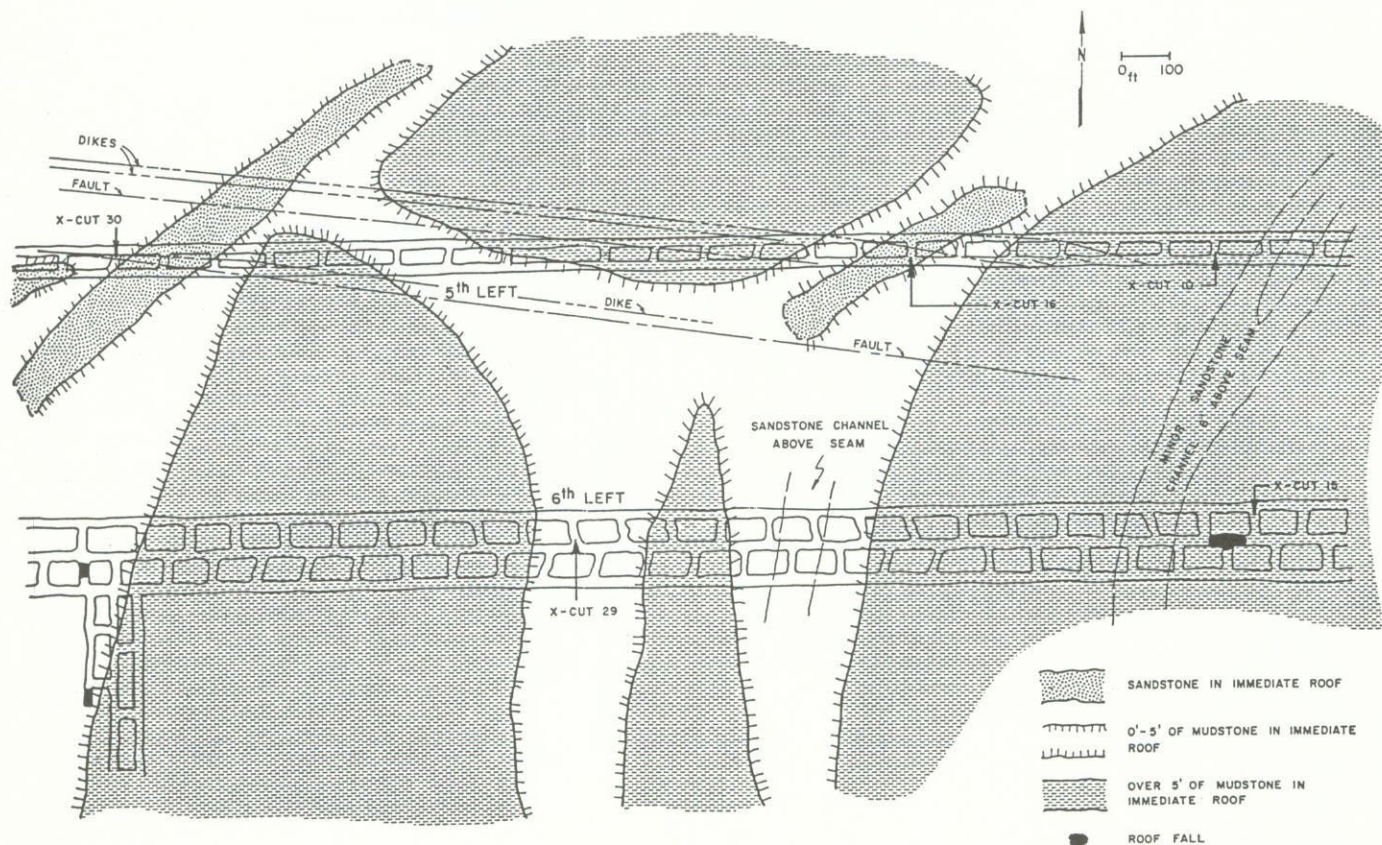


Figure 7 Roof Lithology Map for the Fifth and Sixth Left Entries

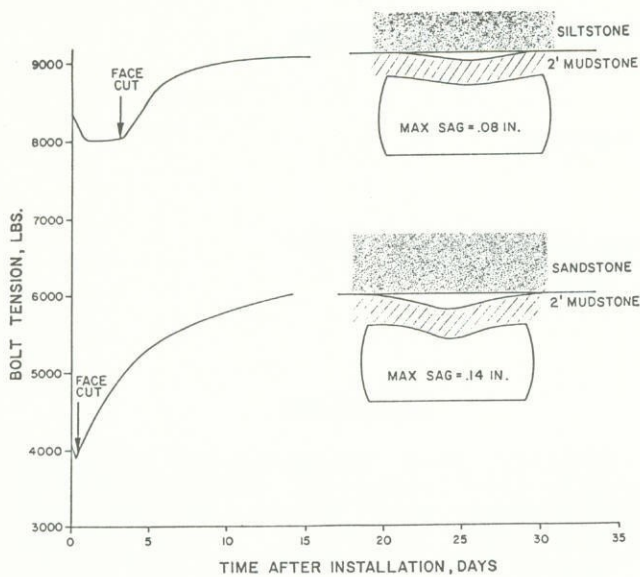


Figure 8 Short-Term Load History on Roof Bolts and Schematic Presentation of Roof Sag

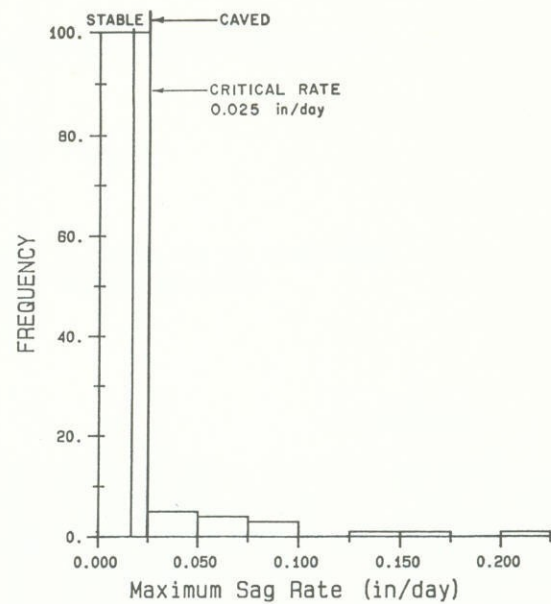


Figure 9 Histogram Frequency Diagram of Rate of Roof Movement

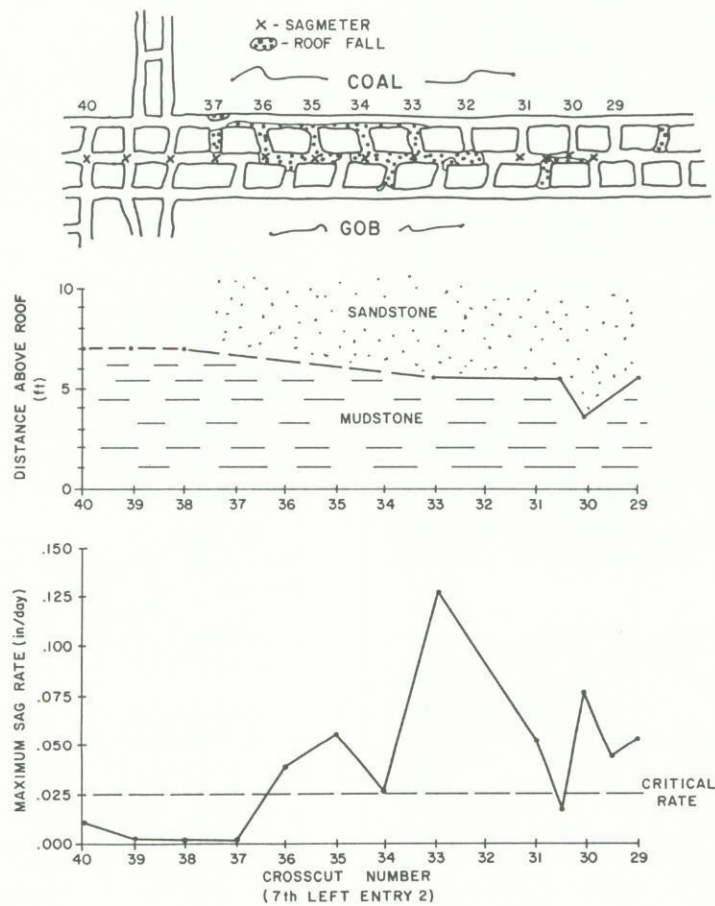


Figure 10 The Correlation Between Rates of Roof Movements, Roof Lithology, and Roof Fall Occurrence

EVALUATION OF ANCHORAGE INTEGRITY FOR GROUTED BOLTS IN WEAK STRATA

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INTRODUCTION

Over the past several years, the Bureau of Mines has conducted studies on the performance of fully grouted resin bolts in underground mines. The effectiveness of resin-grouted bolts in maintaining a competent roof, even in weak rock, has been well documented in the literature. Recent Bureau studies have evaluated the effects of shortened grout columns on bolt behavior in a variety of rock types (1, 2, 3). The application of partial grout columns to primary roof support systems has been introduced as a viable alternative to fully grouted bolt systems in some mines. The point-anchored resin bolt and combination bolt represent such alternatives. While both systems usually employ some mechanical interlocking system, their primary difference lies in the length of resin column used. Point-anchor systems rely on an anchoring device that is a short length of resin column of 2 feet or less. Combination bolts, on the other hand, often require 3 feet or more of resin column. Resin migration into voids and fractures intersecting the borehole wall is a major problem when installing fully grouted bolts in weak roof rock and can result in a reduced column length that provides reduced anchorage. The relationship between column length and anchorage integrity deserves closer study.

The Denver Research Center of the Bureau of Mines has examined the load-carrying capabilities of resin-grouted bolts in weak strata. Pull tests were performed on about 900 resin-grouted, 4-foot-long bolts with column lengths ranging from 6 to 48 inches. The practical performances of 8 different column lengths were evaluated for 10 mines under in situ conditions using data obtained from standard pull tests. In addition, criteria are discussed, including engineering properties of the host rock for designing safer and more effective grouted bolt installations.

BACKGROUND

Ideally, when designing the necessary support, the length and placement of the grout column will depend on the rock strength and geology. Four widely recognized theories of roof support are suspension, lamination, keying, and column reinforcement.

Suspension is traditionally defined as the technique of supporting a weak immediate roof by bolts anchored in an overlying bed of competent rock. Point-anchor resin bolting can be used in such a manner where the resin column is positioned in the competent rock zone, thereby supporting the overall less competent rock masses between the anchored zone and the bearing plate.

Lamination is the process of bonding individual layers of stratified rock into a stiffer and stronger composite beam than the same individual layers unbonded. The characteristics of high anchorage and lateral stiffness allow full-column, resin-grouted bolts to restrict lateral displacements along bedding planes and bed separations in much the same way as bonded layers of stratified rock.

Keying is similar to lamination but involves restricting movement along slip planes of randomly jointed rock masses. Again, high anchorage and lateral stiffness requirements are met through the use of fully grouted resin bolts.

Column reinforcement generally occurs when "stiff" full-columned, grouted bolts are installed in a less-stiff rock mass. Fully grouted bolts, because of their high lateral stiffness, restrict buckling in the horizontal direction and, in a sense, stiffen the strata. Also, an increase in the vertical stiffness results due to a decrease in steel elongation.

Frequently, a combination of the above support modes can be applicable to any given mine roof. The most widely studied

mode is suspension. Research has shown that varying the length of the grout column will influence the mode of bolt behavior (4, 5, 6). Resin columns greater than 24 inches long create stratified, reinforced beams, while those that are less do not create such beams, but instead behave as untensioned anchors. The untensioned, point-anchor system can supply acceptable support in situations where some initial roof sag can be tolerated. Otherwise, greater than a 24-inch column will be required to stiffen the strata into a stronger beam, further reducing downward movement. A tensional component added to the support, such as is found in tensioned, point-anchor or combination bolts, actively inhibits bed separation even further.

Resin-bolt studies have shown similar column length requirements for resin-grouted bolts tested in different lithologies (7, 3). This suggests that comparable physical characteristics of different rock types, such as strength, bed thickness, and the presence of inherent defects, are major contributing factors in determining how a resin bolt responds to short-term, axial loading. Data from other research show that the load transfer around a resin-grouted bolt is dependent on rock strength (8,2).

The popular technique for testing roof bolts has been and continues to be the standard pull test (9). The pull-test procedure was designed to measure the short-term ability of the bolt to resist an axially applied downward load. Mining personnel have long equated the successful testing of a bolt with proper installation; if the bolt load approached or exceeded the yield strength of the rebar, then the parameters for a successful installation were met. Such parameters include proper annulus size, spin time, and bolt thrust. Bureau and other research have shown that often less than a full column of resin will pull test successfully. In this report short columns refer to 24 inches and less of installed length of resin column; long columns refer to the longer lengths up to fully grouted (48 inches).

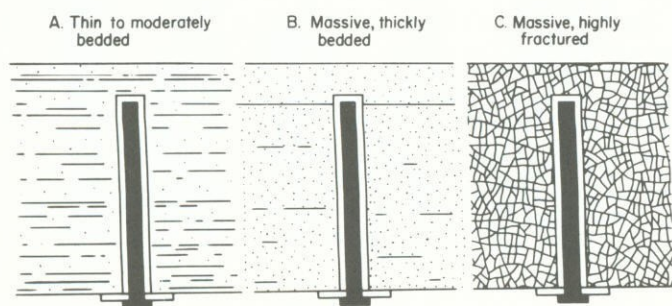


Figure 1. Grouted bolts in different roof types.

The Bureau, using pull-test data, examined the column-length effects attributed to long and short columns in weak roof rock of three types: thinly to moderately bedded; massive, thickly bedded; and massive, highly fractured (Figure 1). These general roof categories were established to describe the strata surrounding the 4-foot bolts.

TEST SITE SETUP

Test sites in 10 mines having the 3 different roof types were established to study column-length effects in potash, salt, gypsum, trona, and borate. At each site, 90 to 100 resin-grouted bolts with various column lengths were installed and pull-tested to assess load-carrying capacity. The bolts consisted primarily of 4-foot-long, 0.75-inch-diameter, type 40 grade steel rebar grouted in place with different length columns of polyester resin. Some larger diameter rebar (0.88-inch diameter) were used at a borate mine. Generally, the bolts were installed in a grid pattern on 3-foot centers at each test site.

Prepackaged, fast-setting resin was used in accordance with resin manufacturers' instructions for proper installation. Partial grout columns were installed by lessening the length of resin cartridge necessary for full column by inserting the package into the back of the borehole ahead of the rebar, which is rotated until the catalyst and adhesive components of the resin uniformly mix. Tape, wrapped around the bolt, was used to restrict the partial grout column towards the back of the borehole during installation. Figure 2 shows how the bolts are installed and prepared for testing in greater detail. To check for an adequate column after installation, boltheads were cut on randomly selected

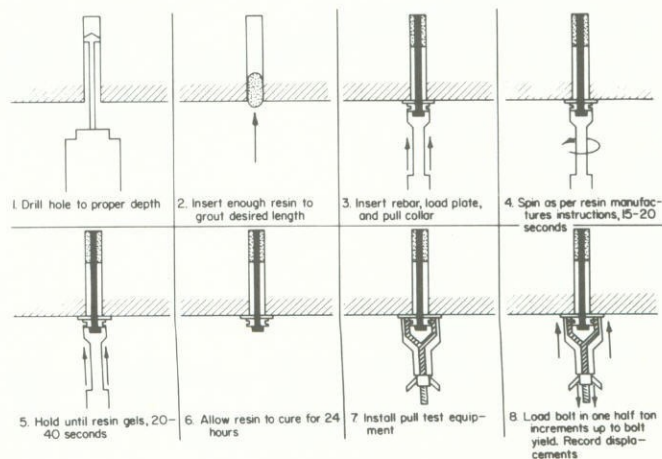


Figure 2. Bolt installation and testing procedure.

bolts, and a wire was inserted to gauge the depth of open hole near the collar. Resin bolts having column lengths ranging from 6 to 48 inches were evaluated for each test site. Six-inch-square bearing plates and pull collars were placed on all bolts. Pull testing was conducted in a conventional manner; bolts were loaded in 1,000-lb increments with the accompanying displacements recorded at each increment up to failure. Figure 3 shows a test in progress with plotted data. The pull-test equipment consists of a hydraulic piston assembly, an extensometer, and a dial gauge to measure bolthead displacement. Figure 4 shows the pull-test equipment. The accuracy of the instrumentation is ± 0.001 inch and ± 100 lb.

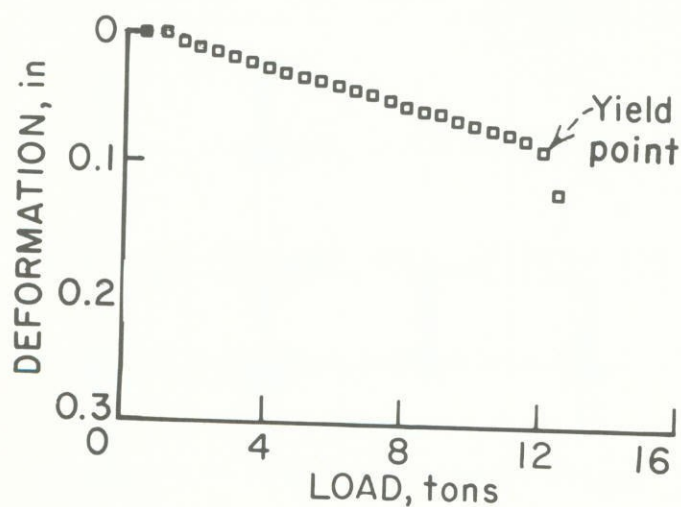
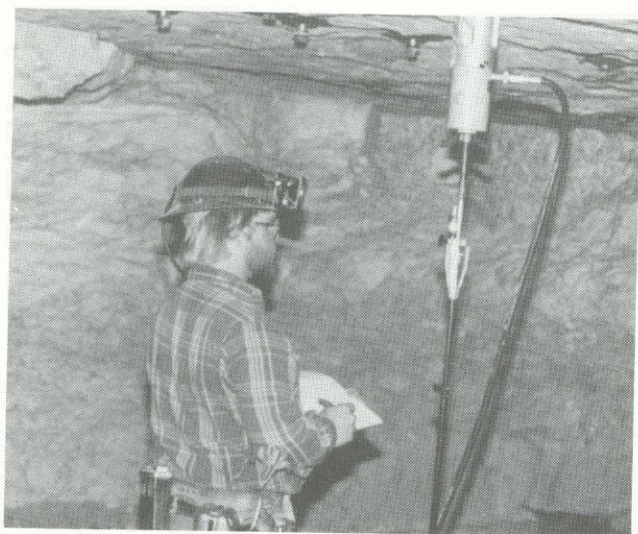


Figure 3. Pull test in progress with plotted data.

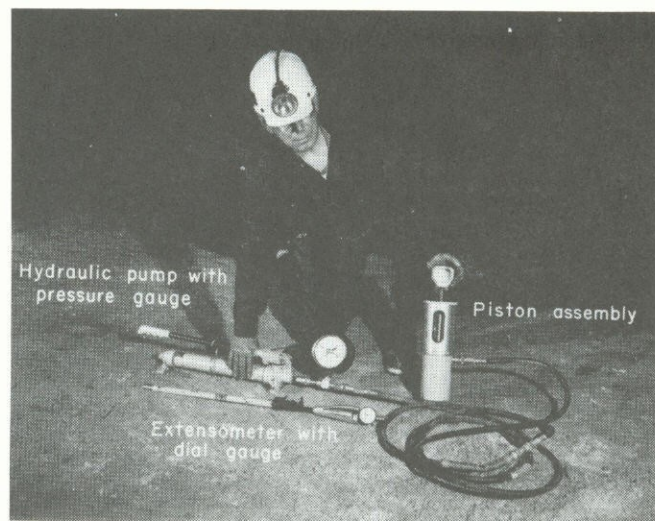


Figure 4. Pull test equipment.

TEST STANDARD FOR EVALUATION OF PULL-TEST DATA

A systematic procedure for rating the success or failure of bolt installations has been developed by the Denver Research Center (1, 7, 3). This technique examines the load-displacement behavior of individual bolts and considers a "good" bolt to be one that accepts loads greater than the calculated yield of the steel with less than 0.2 inch of bolthead displacement. Specific column lengths are rated based on the percentage of "good" bolts within that column length category. The "Effective Column Length" (ECL) is selected as the shortest column length for each test site having 90% or more "good" bolts. Test results using this technique in another study have been included in Figure 5 (3).

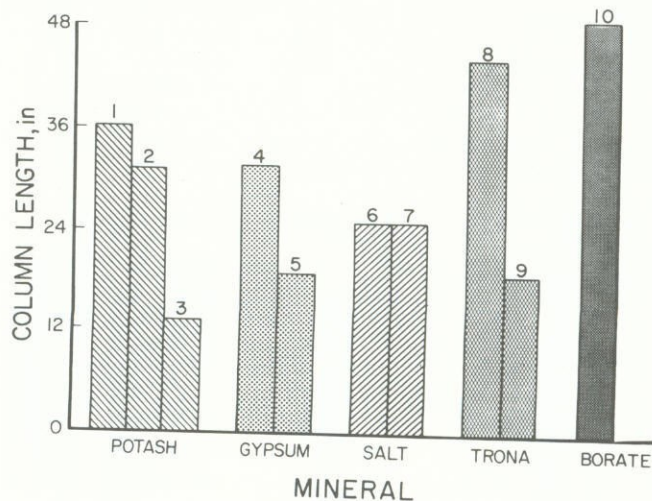


Figure 5. Effective column lengths of 10 test sites.

The purpose for the above procedure is to establish criteria for minimum acceptable column lengths based on axial-loading characteristics, a measure of bolt performance in the suspension mode. The addition of extra column would represent an enhanced safety factor to account for less-than-desired installation practices. Also, additional column would tend to increase the role of the other support modes (lamination, keying, and column reinforcement) for which no simple field test is currently available. Ultimately, the above considerations are input into the decision-making process governing the use of resin-grouted bolts, once the general lithology and structure of the roof are known.

ANCHORAGE TEST RESULTS

Nearly 900 4-foot-long bolts with column lengths ranging from 6 to 48 inches were installed in roofs grouped into three major types. The three major types were thinly to moderately bedded; massive, thickly bedded; and massive, highly fractured. Overall laboratory tests of the roof rock reveal an average unconfined compressive strength of $3,850 \pm 1,173$ psi, shear strength of $1,035 \pm 247$ psi, and Young's modulus of $1.20 \pm 0.45 \times 10^6$ psi. These engineering properties represent weak roof rock. The massive, highly fractured rock is assumed to have weaker strength than the other two rock types.

Pull tests in 10 mines have shown that substantial anchorages can be achieved with short resin anchors installed in weak roof rock. Figures 6 to 8 show profiles of average bolt loading behavior for five typical mines representing the three roof types. The reader is cautioned that the profiles may not agree with the corresponding ECL's since both represent different approaches.

Bolt and pull unit stretch corrections were accounted for in the calculations of bolt displacements. Steel yield effects are dominant at the higher column lengths, as evidenced by the steep load-displacement curves. At the higher column lengths, the bolts failed at approximately 11 tons, corresponding to a rated load of 50,000 psi. This compares with the minimum calculated steel yield of 8.8 tons for 0.75-inch-diameter, type 40 rebar. It is common practice for rebar to exceed the rated steel yield by approximately 20%.

The thinly to moderately bedded mine roof type was identified in trona and gypsum mines. The roof rock at both mines contained thin layers of shale and marlstone which formed the bedding; however, greater than 85% of each borehole was drilled in rock similar to the ore being mined. Pull-test results from both

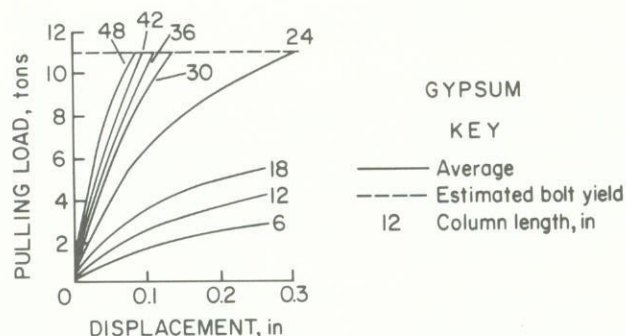
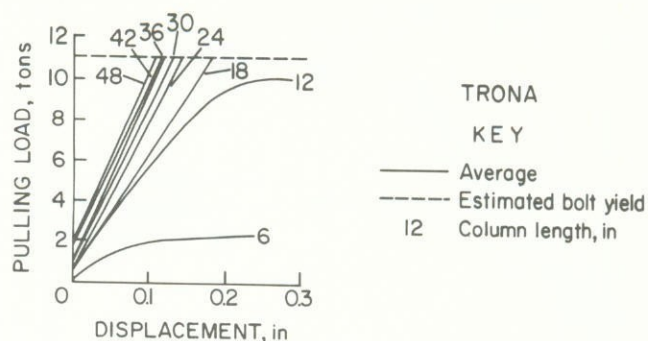


Figure 6. Thinly to moderately bedded roof type.

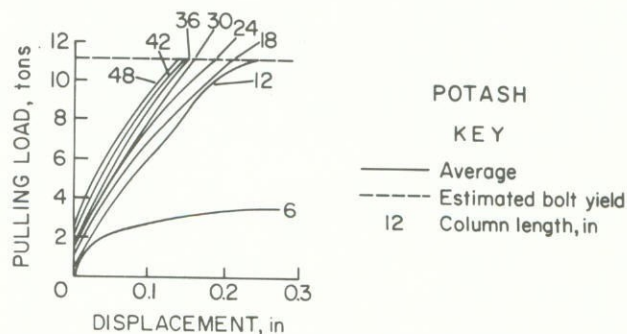
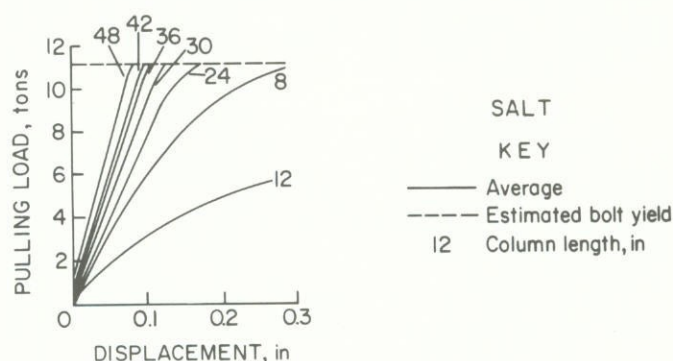


Figure 7. Massive, thickly bedded roof type.

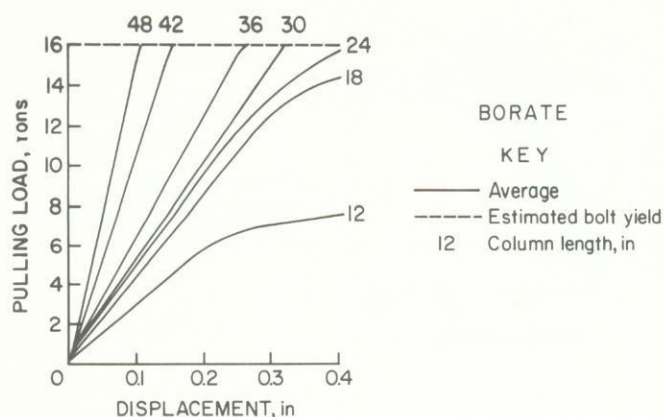


Figure 8. Massive, highly fractured roof type.

mines indicate differences in column length behavior. Figure 6 shows trona exhibited higher anchorage integrity than gypsum. Bolt yield (approximately 11 tons) was attained in both instances at 24 inches of column. The ECL for trona is 18 inches and 30 inches for gypsum.

The massive, thickly bedded roof type was found in salt and potash mines. Again, contrasting differences in column length behavior are apparent in both mine roofs, as revealed in figure 7. Bolt yield was achieved with only a 12-inch grout column in the potash roof, whereas an 18-inch column was needed in the salt roof for similar yield. Suggested ECL's for potash and salt are 12 and 24 inches respectively.

An example of a massive, highly fractured roof was evident in a borate mine. Because of the highly fractured nature of the roof rock, laboratory samples could not be obtained. Figure 8 shows high anchorage at the various column lengths when 1-inch-diameter rebar was used; however, the data do not reflect the high incidence of inadequate installations as evidenced by unhardened resin, which allowed the bolts to be pulled out by hand. Significant loss of resin into the fractures was the primary cause of improper installations. The use of at least 42 inches of grout dramatically reduced the number of bad installations at this mine. Under the ECL procedure, a full grout column (48 inches) is required.

An overall assessment of the average column length data for each test site supports the same trend; the stiffness of the grouted bolt system generally increases with an increase in the column length up to full column. Under applied axial loading conditions, longer grout columns can resist higher loading up to a level that equals or exceeds the yield strength of the steel rebar. In this study, the critical column length was estimated at 26 ± 9 inches. Longer

columns may become necessary in highly fractured rock. Since variations in this critical column length must certainly be attributed to the effectiveness of the resin-to-rock bond or anchorage, a better understanding of this resin-to-rock anchorage is required.

ROCK QUALITY AND ANCHORAGE CONSIDERATIONS FOR PROPER RESIN-GROUTED BOLT DESIGN

A practical approach to developing a procedure to more effectively evaluate resin-bolt performance is to understand the engineering parameters of the rock in which the bolts are anchored. Two areas of investigation are (1) loading characteristics of the rock-wall lining near the bolt and (2) general rock quality away from the bolt.

Some researchers have studied the relationship of physical rock properties such as the unconfined compressive strength of intact rock cores to the ultimate bearing capacity of a rock mass (10, 11, 12, 13). The ultimate bearing capacity of a rock mass is the load-per-unit area the rock can support without excessive failure. The term "bolt anchorage" has similar meaning in that the boundary at the resin-to-rock contact around a borehole is the bearing surface. The major difference between ultimate bearing capacity and bolt anchorage lies in the direction of loading.

The ratios of the ultimate bearing capacity of rock to the unconfined compressive strengths can range from 3 to 8 (10, 13). Research has shown that during pull tests, the resin and rock surrounding the steel rebar are subjected to compressive forces (8). To apply these relationships to the grouted bolt study, the author proposes comparing the average compressive strengths of intact rock core to a unit anchorage that adequately represents the host rock with minimal steel effects. A selected column length for testing unit anchorage was 24 inches. Pull-test results for 24-inch grouted bolts were used in this study as an index test for unit anchorage. By using the same column length in different rock types, valid anchorage comparisons were made with minimal steel yield influence. Shorter column lengths were evaluated but did not provide as good a correlation. The results of this analysis using 24-inch grout columns are presented in Table 1. The equation presented in the table reveals an inverse relationship of the compressive strength of the intact rock to the in situ anchorage measurements in the prediction of an ECL. The exact function between the compressive strength and the ECL is unclear because of this inverse relationship. Without additional testing, the applicability of the above should be

questioned and examined further at other mines in other rock types. Such a procedure, if perfected, would greatly reduce the large number of pull tests necessary under the old system of ECL determination.

Compressive Strength/Anchorage Relationship in Estimating-ECL

	A Unconfined Compressive Strength (psi)	B 24 inch Anchorage (tons)	C Effective Column Length (inches)	Predicted ECL (inches)	Difference (inches)
Potash	4244	10.4	36	35	1
	3321	10.9	30	26	4
	3689	12.6	12	17	-5
Salt	2754	11.2	24	21	3
	3625	11.9	24	21	3
Gypsum	2975	10.9	30	24	6
	3131	11.1	18	24	-6
Trona	4270	12.7	18	13	5
	6650	12.5	42	35	7

EQUATION to Predict ECL

$$80.9 + .0059A - 6.806B = C$$

$$r^2 = .81$$

Table 1. Compressive strength-anchorage relationship for estimating ECL.

Novel methods are needed to quantify the support needs of roof rock based on rock characteristics and index anchorage testing. Other methods such as rock mass classifications, which attempt to apply numerical or qualitative ratings to rock, offer additional opportunities to characterize roof rock. Only by applying such procedures can the column-length-to-anchorage relationship of resin-grouted bolts in weak stratified roof rock be better understood.

SUMMARY

Column-length anchorage effects attributed to long and short columns were evaluated for thinly to moderately bedded; massive, thickly bedded; and massive, highly fractured weak rock. There was no significant difference in bolt behaviors that can be attributed solely to rock or roof type in thinly to moderately bedded and in massive, thinly bedded roofs; pull tests showed repeated successes using partial and full columns. In highly fractured rock the outcome was not predictable; obtaining a good installation with a partial column was uncertain because of the likelihood of resin loss into fractures or improper resin-catalyst mixing due to oversized annulus. To ensure adequate bolt performance, the use of fully grouted columns is recommended when significant fracturing is expected.

The indiscriminate use of full columns could easily be considered a conservative approach for all mine roofs based entirely on anchorage studies; however, the fundamentals of sound roof design should consider the question of stiff or yielding roof support. Clearly, point-anchor systems would offer less stiff roof

reinforcement than is achievable with fully grouted systems. Under some weak roof conditions, yieldable roof support can be more stable and less likely to cause catastrophic roof failure; therefore, a prerequisite for deciding on full or partial resin columns should rest largely on the support design criteria, as well as the bolt pull-test performances for a specific roof rock.

Procedures are needed that can not only help measure the quality control of grouted bolt installations, but also provide input on the mechanical quality of the roof rock and the degree of frictional contact along the borehole boundary. Simple index testing, which can quantify the competency of the host rock and the mechanical interlocking force along the borehole wall, should be part of a bolt effectiveness program. Suggested methods for characterizing the roof rock are using material properties such as the unconfined compressive strength or some rock mass classification procedure.

A practical standard test for measuring the degree of frictional contact along the resin-to-rock boundary is a modified pull-test procedure using large-diameter or high-grade steel rebar with a short anchor inserted at a specific horizon in the borehole to be tested. Importantly, the borehole should be drilled with air instead of water in weak rock, which may have a tendency to "mud up" when drilled. If significant steel yield is avoided, then the actual bolt behavior can be largely attributed to bolt "slippage" in the borehole, assuming the rock is considerably weaker than the steel. Such data, if obtained in different geologic horizons, would result in more meaningful comparative unit anchorages. Roof coring or visual borescope inspection methods may be used to identify the different horizons to be tested.

Another promising tool, the ECL method, established guidelines on resin-bolt performance based on individual load-displacement requirements and the statistical success of a group of bolts with the same column length. The major drawback of the standard ECL method is the large number of tests needed to complete such a study. The investigation of tentative correlations between roof rock characteristics and unit anchorages may greatly reduce the number of tests to more quickly estimate an ECL. Continued research in this area is needed to provide data on other mines with different rock types.

CONCLUSIONS

The following conclusions are drawn from the information presented:

- a) The pull test is a valid procedure for checking axial-loading characteristics of installed resin-grouted bolts; however, this test by itself is not sufficient for determining a proper installation.
- b) The ECL method is a viable procedure for applying load-displacement criteria for safe bolt installations. The limiting criteria used to judge the bolt axial loads and displacements should reflect the yield characteristics of the steel used in the rebar and pull test unit.
- c) Comparison anchorage testing should be done using similar-length, short resin columns in different test sites or borehole horizons. Rebar effects should be minimized by pull testing at loads lower or near the steel yield.

The grout column selected should also be long enough to allow proper catalyst-adhesive mixing. For this reason, a 24-inch resin column was chosen for the study.
- d) Increasing column length generally results in increased bolt stiffness. In highly fractured roof media, fully grouted bolts may be needed. The need for lessening the stiffness of the roof support may necessitate the use of partial columns as found on resin point-anchor systems.
- e) Some roof rock characterization technique should be part of the bolt support plan. Tentative correlations of roof rock and borehole wall characteristics to anchorage integrity can evolve into more meaningful column-length design criteria.

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STRATA III
A FULL-SPECTRUM ROOF CONTROL CONCEPT

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ABSTRACT

Strata III is a full-spectrum roof control system which provides the mine operator the ability to upgrade his roof control in place. The procedure is easily adaptable with current roof bolting techniques. To use the system, the operator starts by installing a special hollow core mechanically anchored roof bolt during the roof bolting cycle. This bolt can be upgraded to a fully grouted tensioned rebar bolt by injecting StrataGee G-300, a special polyurethane grout, around the bolt through an injection port in the roof plate. This same bolt can then be used to inject RokLok binders for roof consolidation by pumping the binders through the core of the bolt.

This concept in roof control was developed to provide the mine operator a flexible way to maintain roof control in areas which deteriorate over time, or must be kept open for long periods of time. Possible locations of use would be: longwall set-up rooms, recovery rooms, headgates, intersections, and belt entries.

INTRODUCTION

Mobay Corporation has demonstrated its concern for improving the safety for U.S. coal mines and improving the productivity at the same time. In 1977 Mobay introduced polyurethane binders for the consolidation of highly fractured roof. This technology was first introduced in Germany in 1970 where it has been widely accepted. Polyurethane injections are now being used widely throughout Europe to control the strata on deep highly stressed longwall faces. The use of polyurethane has improved production of these European longwalls. Almost 40 million pounds of polyurethane binder is being used annually in the coal mines throughout the world.

In the United States, the use of polyurethane for strata control in underground coal mines has reached almost 2 million pounds annually. The main use is to facilitate and protect the recovery of longwall face equipment after panel depletion. It has been demonstrated that the planned application of polyurethane binder during the recovery of longwall panels can reduce the cost of the moves and increase the productivity of the mine. (1) Polyurethane is also used to reconsolidate broken roof strata which has stopped or impeded production from the longwall, most common areas are the head and tail gates and the longwall face.

Areas where the head and tail gates will begin to breakup and impede production can not be predicted. Thus, areas of application are difficult to plan for to make the most effective use of polyurethane binder. In most cases the roof has already begun to fall before management decides to inject polyurethane binder to reconsolidate and re-establish roof control. A shift or more of production can be lost to downtime while waiting for injection to begin. Additional time is lost to drilling and preparing the roof for the binder injection. Such delays in injection increases the amount of binder required to reconsolidate the roof because the fractures have continued to widen with the delay. The use of the Strata III bolt can improve the overall effectiveness of polyurethane injection.

STRATA III BOLT

The unique feature of the Strata III bolt is the high strength hollow core rebar. (Figure 1.) This hollow core rebar is 3/4-inches in diameter and is made from a grade 70 steel. The hole down the center of the bar is just under 1/4-inch. This rebar has rated ultimate strength of 34,000 pounds.

The other components which make up the Strata III bolt are, for the most part, standard roof bolt items. All are MSHA approved. The anchor shell currently being used is a F2B bail anchor for a 1 3/8 inch hole. Other anchor shells can be used. The roof plate is a Jenmar 6" x 6" x 1/4" bell plate. The plate has been modified to include a small one way rubber injection check valve. The nut is a Fraser Jones dome nut which is used with a standard 3/4-inch washer. The final component is the medium density foam seal which is placed between the roof plate and the roof. (Figure 1 shows the details of the Strata III bolt.)

STRATAGEE G-300 GROUT DEVELOPMENT

StrataGee G-300 grout was developed to meet most of the design criteria for pumpable grouts recommended by Amicon Corporation for the United States Bureau of Mines. (2) (Table I compares the recommended criteria to the properties of StrataGee G-300.)

RokLok binder was tested for use as a pumpable roof bolt grout but it could not achieve many of the recommended properties. The most critical was the ultimate strength. It could not reach the 25,000 lb/ft ultimate pull-out strength. It also performed poorly in wet hole tests, plus its gel time exceeded 1 minute.

During the development of StrataGee grout, factors were considered, such as, ease of mixing, and loss of grout into cracks in the roof. The lower the viscosity of the materials to be mixed, the simpler the pumping and injection equipment required. Because the viscosity of the materials are low, the grout can be quickly injected around the bolt. But the use of low viscosity materials increases the likelihood of grout being lost into the cracks in the roof. To reduce this likelihood, the gel time of the material was kept as short as possible (30 seconds at 68°F). Another advantage of short gel time is that the grout is less sensitive to water contamination enabling the strength of the grout, in wet holes to reach acceptable level.

Other advantages of StrataGee G-300 grout are its adhesiveness and flexibility. StrataGee has good adhesion properties which add to the pull-out strength of the grout. It is also less brittle than cement and polyester grouts. A bolt installed using StrataGee grout can be stressed and destressed without breaking the integrity of the grout.

Extensive pull-testing was done during the development of the StrataGee G-300 grout to determine its effective strength. Number 6 rebar bolts, about 16 inches long were grouted into 1 1/8 inch diameter holes 12 inches deep drilled into blocks of Indiana limestone. The grout was allowed to cure for different intervals of time, varying from 5 minutes to 24 hours, before the bolts were pulled. It was determined that the StrataGee grout will reach its ultimate strength in about 2 hours and half that strength in about 20 minutes. The StrataGee has a pull-out strength of better than 25,000 pounds. (Figure 2 shows that when a force of 30,000 pounds is placed on the bolt, the displacement is only 0.2 inches.)

PUMPING

A critical step in the use of the Strata III bolt is the placement of the StrataGee grout into the annular space between the bolt and the side of the bolt hole. The pump that is used to place the grout around the bolt is capable of injecting a minimum of 0.5 pints or pumping continuously for a much larger injection. The pump can be adjusted in increments of 0.5 pints. This flexibility permits the installation of all bolts 1 foot or longer, or a full scale injection of RokLok binder through the bolt into the strata.

The pump is designed to be easily transported underground, easy to operate and easy to maintain.

There are 2 versions of the pump, air operated and hydraulically operated. Each pump provides the same flexibility and ease of operation. Each pump consists of two equal volume pumping pistons. One for each of the two components of the StrataGee grout; a reciprocating pumping motor, either pneumatic or hydraulic; and a stroke counter to control the injection volume. The pump is mounted on pneumatic tires and has a place for two 5 gallon cans of grout on top. The cans can be hooked directly to the intakes of the pumping pistons. (Figure 3 is a drawing of the pump.)

From the pump, the components of the StrataGee grout are pumped through independent hoses to an injection manifold or injection gun. Here the components are brought together and pumped through a disposable static mixer. The static mixer doubles as the probe which is inserted into the check-valve

in the roof bolt plate. The injection manifold has provisions for the use of a nontoxic flush. The valves on the grout component lines can be closed and then the manifold flushed by opening a needle valve on the flush line. Right now the flush line is connected to a separate container from the pump. The pump and manifold can be temporarily stored by closing the valves on the pump piston intakes, closing the valves on the injection manifold and flushing the manifold. (Figure 4 shows the details of the injection manifold.)

It is also possible to inject the StrataGee G-300 grout using a two component caulking gun if it is impractical to use either the pneumatic or hydraulic pumps. The caulking gun has a mixer which is inserted into the check valve in the roof plate and the grout is squeezed into the hole around the bolt.

INSTALLATION OF BOLT

The installation of the Strata III bolt is almost identical to the installation of all other mechanically anchored bolts. First, a 1 3/8 inch diameter hole is drilled into the roof. This hole should be 1 to 2 feet deeper than the desired anchorage horizon. This additional depth will permit the future injection of RokLok through the bolt. The bolt assembly is inserted into the hole, making sure that the foam seal is properly positioned between the bell plate and the roof. This is to prevent the leakage of StrataGee grout from between the plate and any irregularities in the roof. The bolt is then torqued and set. A torque of 150 ft-lbs will result in a tension of 10,000 pounds. (Figure 5 shows the relationship between torque and tension.) The bolt is a compliance mechanical roof bolt at this stage.

The hollow core bolt can be grouted either immediately or months later, after it has been installed as a mechanical bolt. The grouting requires the use of the metering pump to assure that the proper amount of grout is pumped around the bolt. The grouting of the bolt is a simple procedure. First, the number of pump strokes required to grout the bolt up to the anchor shell is set. It will require 1.5 strokes per foot of bolt in a standard 1 3/8 inch bolt hole. A 4-foot bolt would require 6 pump strokes. Grouting is not done beyond the anchor shell because the core of the bolt must be kept clear for the future use of RokLok binder injection.

To do the grouting, the plastic mixing nozzle on the injection is held firmly

into the rubber check valve in the bell plate. The pump is started by pressing the trigger. When the pump stops its cycle, the mixer can be withdrawn from the check valve and moved on to the next bolt to be grouted or flushed and readied for the next application.

The final step in the use of the Strata III bolt is the injection of RokLok binder into the roof through the core of the bolt. Standard RokLok binder injection mixers and pumps can be attached to the Strata III bolts. Then RokLok binder would be injected into the roof as it would normally be done to reconsolidate the broken roof strata.

ANTICIPATED APPLICATION AREAS

The Strata III concept in roof bolting was developed to fill a need in the coal mining industry. It fills a gap between compliance primary roof bolts and supplemental roof support such as trusses, jumbo bolts, and cribs. Because its chief benefit is versatility, it can be upgraded in three steps from a mechanical bolt to a fully grouted and tensioned combination bolt to an injection packer for RokLok. Because of this versatility, it can be effectively applied to a wide variety of ground control problems.

Intersection Repair

Mine X has been experiencing a rash of intersection roof falls which occur some time after mining. The mine management has been using 1 inch diameter high-strength bolts in an attempt to hold the roof in these intersections. These bolts have been partially successful in supporting the intersections, but they can not re-establish the beaming effect needed for good roof control.

The mine could effectively re-establish roof control in these intersections by installing 5 Strata III bolts as part of the primary roof control plan. (Figure 6.) The bolts would be torqued and tensioned, then fully grouted. This would tend to prevent the bleed-off of tension and the related problems. If the roof does continue to work and breakup it can be injected with RokLok to re-establish the beaming effect needed for good roof control.

Longwall Head and Tail Gates

Mine Y has a large investment in its longwall. It has been a user of RokLok binder in the past and knows how many times RokLok succeeded in some very difficult ground conditions. They've

always waited until the last minute, when everything else had failed, to contact someone else to inject RokLok binder. As a result, the use of RokLok binder becomes an expensive proposition. They are in need of a more effective way of making use of the benefits of RokLok binder injection.

Mine Y could use the Strata III bolt in these head and tail gates where difficult conditions are anticipated. The mine could install Strata III bolts during the development of the head and tail gates. An idealized pattern for the installation of the bolts would be to make every fourth bolt a Strata III bolt. A Strata III bolt would be installed on one side of each row, alternating sides from row to row. (Figure 7.) The Strata III bolting pattern is determined by the anticipated RokLok binder injection pattern. Patterns will vary from mine to mine.

The mine could keep an injection pump along with several thousand pounds of binder on the section to be able to inject through the Strata III bolts.

With the bolts in-place, the mine can respond quickly with RokLok binder injection when portions of the entries begin to deteriorate. By responding quickly, the mine can reconsolidate the roof with less binder. With ability to upgrade roof support in-place on an as-needed basis through the use of the Strata III bolting system and RokLok binder the mine could reduce the amount of other types of roof control, such as trusses and cribs. An overall reduction in the cost for roof control could be realized.

Other Areas of Application

The Strata III bolt system can be used in any other area where other types of supplemental roof control are needed. One of these areas might be the main belt or track haulage which must be maintained for the life of the mine. With the Strata III bolt in-place in these entries it would be a simple matter to reconsolidate any area that begins to deteriorate.

The Strata III concept can also be applied to the control of water entering the mine through the strata. In areas of the mine which might pass under water-bearing strata, or creeks, Strata III bolts could be installed for possible injection of RokLok binder for water-stop. The RokLok binder would be injected only if water began to find its way into the mine.

TRIALS

Several trials of the Strata III concept have been conducted to demonstrate the bolt's effectiveness. The first trial was to determine just how easy it would be for an average bolting crew to install the bolt as part of their normal bolting cycle. The only additional training required was to show the bolter operator and his helper how to place the foam seal between the plate and the roof. This took approximately 5 minutes. The drilling and bolting proceeded as normal. The bolt being installed was 8 ft long made up of two 4 ft segments. The only increase in time was due to drilling the hole 1 to 2 ft deeper than the anchorage horizon.

Measuring the amount of time it would require to grout a lift of bolts was also done. At this mine 16 bolts were being installed per lift. Each bolt would require 12 strokes from the grout pump. It took about 15 seconds to place the grout around each bolt. All 16 bolts were grouted in less than 3 minutes. A second lift of bolts was installed with the same results.

The second trial was conducted in the headgate of a longwall which was experiencing roof control problems. The roof had sagged to the point of sitting on top of the stage loader. To prevent this problem from occurring in a section of the entry out by the problem area, the mine management decided to try Strata III bolts to keep the roof from falling.

It was determined that 40 four foot long Strata III bolts could be pre-installed for future RokLok binder injection to support the anticipated problem area. The bolts were installed as secondary bolts every 8 feet along the headgate entry down each side of the entry. This spacing matched the spacing of injection packers they would have otherwise used. Each row of bolts was about 4 ft from the entry center line. The bottom 3 ft of the bolts were grouted with StrataGee grout, making them ready for polyurethane binder injection.

When the longwall approached the problem area, the roof was injected with polyurethane binder through the Strata III bolts. The longwall was then able to pass through the area without any problems.

A third trial for placing Strata III bolts was done in the setup roof for a longwall panel. In this application, 200 bolts were pre-installed in the entry before the entry was widened to accept the longwall equipment. The bolts were

spaced 8 ft apart in two rows 4 ft from the entry center line. After the bolts had been installed a crew of 2 was brought in to grout the bolts.

The grouting of the bolts took less than a full shift. This included setup and cleanup. The grout pump and all grout was placed in the bucket of a scoop which was trammed down the entry as the bolts were being grouted. The actual time involved in grouting was about 4 hours.

About 10 percent of these bolts leaked grout as they were being grouted. The bulk of these leaking bolts were due to large cracks in the roof. The other locations for leaks were around the foam seal which did not fill all the irregularities in the roof, or the angle between the bolt and the roof was too far off, 90 degrees to permit a good seal.

Once the bolts had been installed and grouted, the entry was ready at any time for the RokLok binder injection prior to widening. About 3 weeks later, the entry was injected with RokLok binder in those areas which were thought to be potential problem areas. The setup entry was widened without encountering any roof control related problems.

During the injection it was determined that the spacing on the bolts could have been increased to 16 ft. It was found that the radius of migration of the RokLok binder was greater than first determined. Thus, the mine will be able to do the same procedure on the next longwall with 1/2 the number of Strata III bolts.

A fourth trial is in progress at another mine which has installed about 200 Strata III bolts in areas of belt entry which might require additional roof support in the future. If and when these areas begin to show signs of deterioration, they will be injected with RokLok binder.

HEALTH AND SAFETY

The installation of the Strata III bolts should be done in accordance with all roofing safety standards. At this stage, the Strata III bolt is no different than a standard mechanically anchored roof bolt.

During the use of the StrataGee G-300 grout and the injection of RokLok binder, certain health and safety precautions must be observed. The precautions do not interfere with the effective use of the materials underground. However, before working with any polyurethane material, you must read and become familiar with available information concerning its

hazards, proper use and handling. This information is available in several forms from the manufacturer i.e., Material Safety Data Sheets and Product Labels.

CONCLUSION

The Strata III bolting system can be an effective means of both primary and secondary roof support. This is especially true in areas which are suspected of experiencing future deterioration. For a minimum increase in initial bolting costs, the mine operator achieves the ability to upgrade his roof control in-place quickly by grouting in the Strata III bolt with StrataGee grout and then, if needed, injecting RokLok binder through the same bolt to reconsolidate the roof. This ability should allow for a reduction in the use of other roof control items such as cribs and posts and trusses to mention a few. By reducing the need for these items an overall reduction in roof control costs could be realized.

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TABLE I
CRITERIA FOR PUMPABLE GROUT

Property	Recommendations	StrataGee G-300
Pull-out Strength		
Ultimate	25,000 lb/ft	25,000 lb/ft
Wet Rock	12,500 lb/ft	12,500 lb/ft
Creep	Minimal	Minimal
Compression Strength	As good as existing grouts	7,000 psi
Gel Time (40 to 100 F)	Controllable to 1 minute	Yes
Flash Point	Above 140 F	Above 400 F
Flammability	None	Yes
Explosivity	None	None
Odor	Low	Low
Toxicity	Minimal	Minimal
Cleanout solvent	MSHA approved	Detergent-water
Shrinkage	None	None

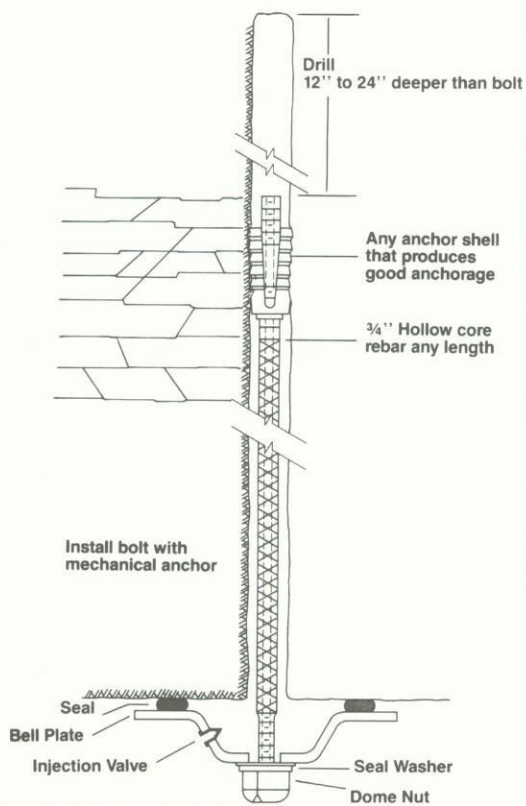


Figure 1. Details of Strata III Bolt

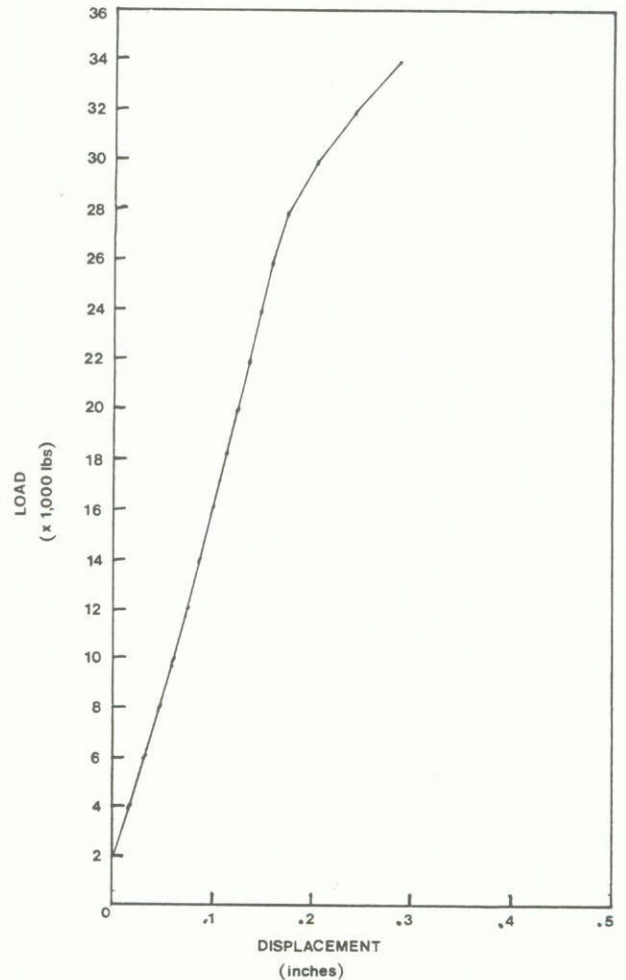


Figure 2. Pull Test of StrataGee G-300 Grout

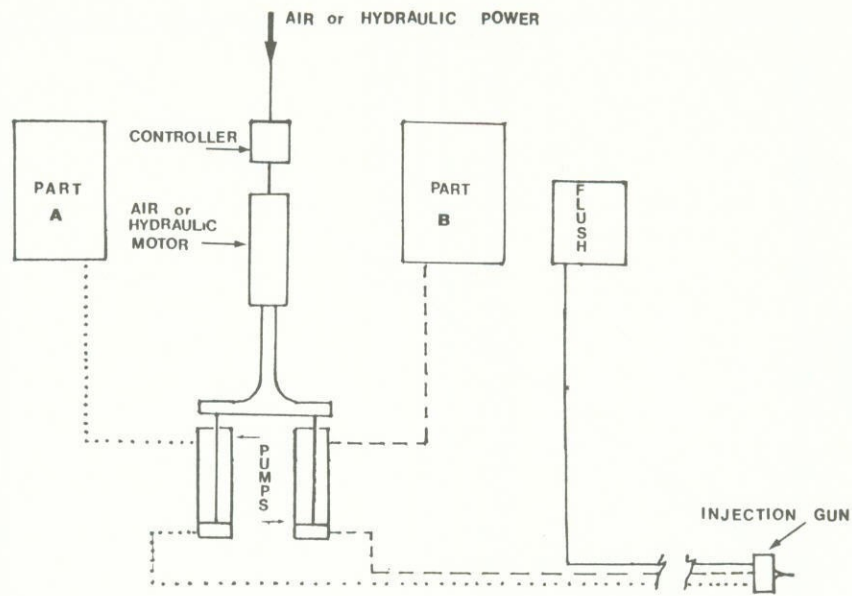


Figure 3. Schematic of Grout Pump

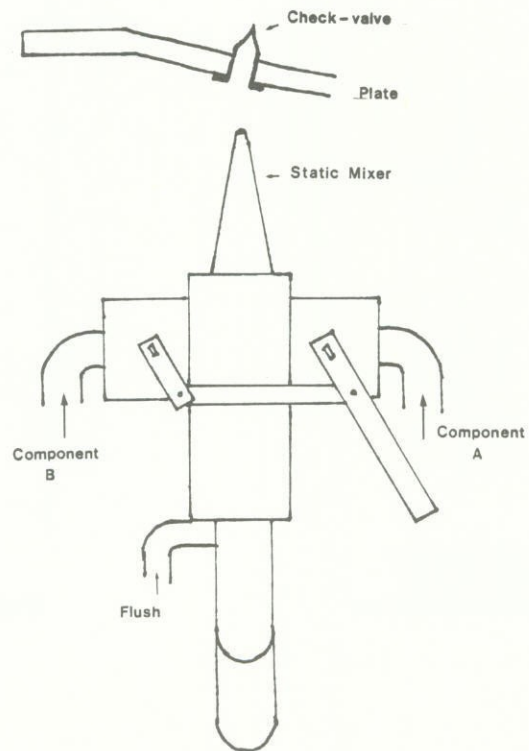


Figure 4. Details of Injection Gun

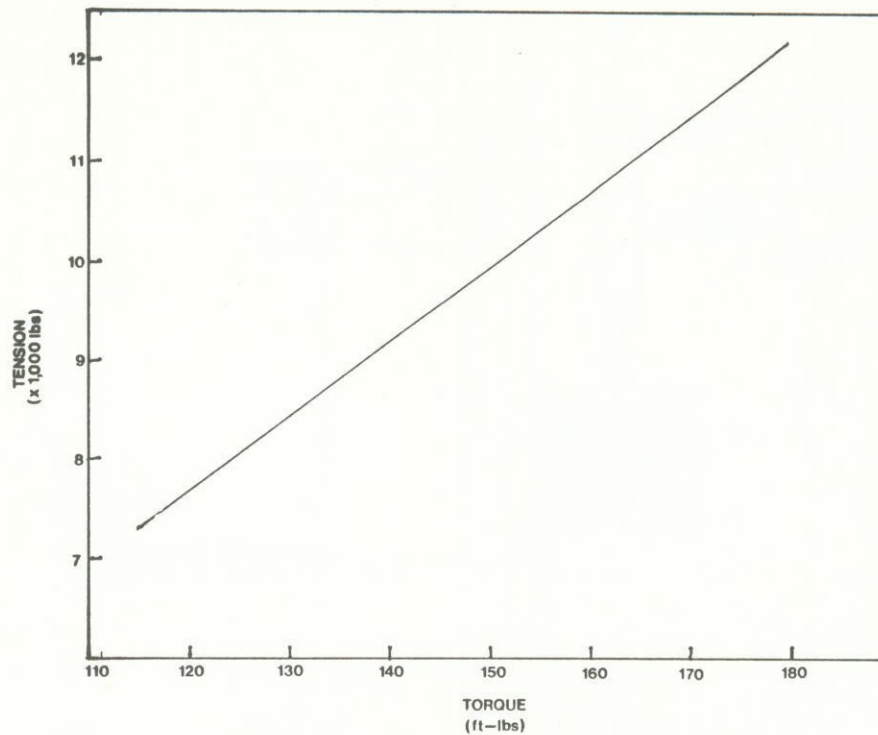


Figure 5. Torque vs. Tension for Strata III Bolt

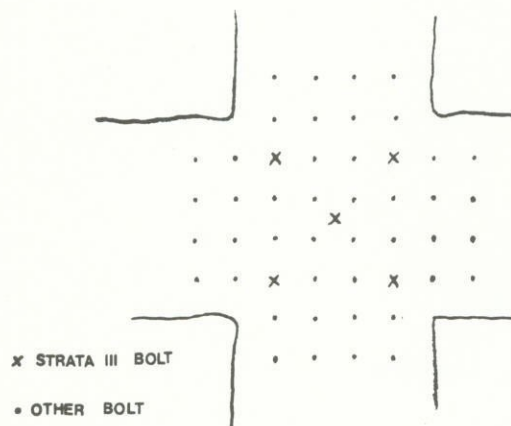


Figure 6. Proposed Installation for Strata III Bolts used in an Intersection

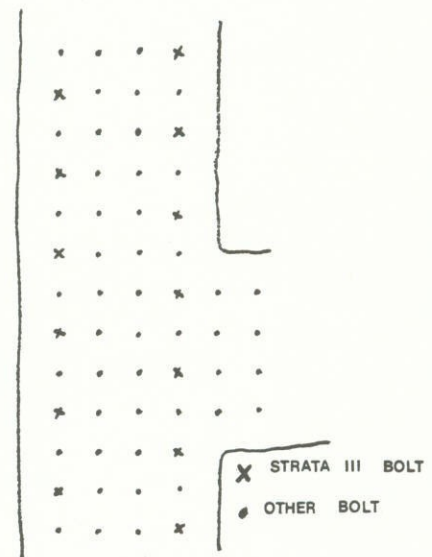


Figure 7. Proposed Installation Pattern for Strata III Bolts used in Head and Tail Gate Entries

STUDY OF GROUND MOVEMENTS OVER A LONGWALL MINE

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ABSTRACT

This paper presents an analysis of ground movements recorded from longwall operations in the Northern Appalachian region in West Virginia. The site chosen for the investigation was selected due to varying heights of overburden and topography and was instrumented for both surface and subsurface measurements. The instrumentation areas consisted of two panels covering various sections with unique geologic and topographic features.

Instrumentation included surveying monuments and shallow boreholes using an inclinometer for measuring horizontal strata displacements. Vertical strata deformation was monitored utilizing a unique design of the mechanical grouting method. Full length borehole profile extension was measured using Sondex displacement rings in the inclinometer installation. Subsurface displacements, i. e. caving zone, fractured zone, and highly deformed non-fractured zone were identified. The nature of surface disturbances with respect to varying geologic settings were compared, and an attempt was made to simulate final strata caving and bed separation using finite element analysis.

INTRODUCTION

Several investigators have tried in recent years to monitor and predict rock mass movements between the mining horizon and the surface above active longwall mining extractions, as evidenced by various workshops and symposia (1, 2, 3, and 4). Subsurface subsidence studies require the monitoring of strata from mine level to the surface to insure the best selection of longwall face supports. In specific situations it is imperative to know whether caving, or strata breakage, extends to the surface or, say to a significant water aquifer. In order to monitor the development of caving, instrumentation must be installed from the surface to mine level, however, very few systematic studies that actually monitor the caving mechanism associated with longwall mining have been conducted. To develop a better understanding of longwall strata behavior under

varying geologic and topographic conditions, a subsurface strata monitoring program was initiated.

Caving height of the immediate roof is a major concern when selecting face roof supports and has been determined from equations developed by many researchers. Most methods used for determining the proper load capacity of longwall roof supports are based on the assumption that a volume of roof material, dependent on the height of initial caving, must be supported. Caving height is usually estimated as a constant times the extraction height, the constant being determined by the bulking factor of the caved material. Various formulas have been developed by many researchers and as can be expected, vary considerably. For example, Wilson (5) assumes that caved material occupies 1.5 times the volume of the same rock in-situ; therefore the height of caving of the immediate roof is twice the extraction height. However, by measuring volumes of underground roof fall cavities in the Pittsburgh coal seam and comparing them to rock fragments piled up on the floor, Peng (6), found that the bulking factor for shale roof ranges from 1.25 to 1.30 with an average of 1.28.

To physically measure the caving horizons above a longwall extraction, two 6" boreholes were instrumented with a unique design of the mechanical grouting method along each panel. Each borehole was used for measuring vertical strata displacements and water level measurements in the water bearing formations. A number of subsidence monuments (stations) were placed both longitudinally and transversely to the direction of face advance. Surface elevation surveys were performed on these local monuments during borehole monitoring to differentiate between surface and subsurface displacements. Full length vertical displacements were also monitored within a borehole using a settlement probe, which provided insight into the vertical movements experienced adjacent to longwall extractions.

Lateral strata movements were studied adjacent to the active longwall extraction utilizing a full profile borehole inclinometer in hopes of detecting the assumed subsurface lateral movement directed

toward the extraction. The borehole was placed in a chain pillar in the tailgate development to protect the inclinometer casing from being ruptured due to the caving process and to fully utilize the inclinometers capability to detect the high horizontal movements associated with the panel's edge.

SITE DESCRIPTION

Many factors were considered during site selection, including topography and terrain, surface access (private land ownership), and environmental restrictions. However, the most important technical consideration was to choose the portion of the test site that would yield representative results over the entire length of the panels. The longwall panels under investigation are located in a northern region of West Virginia, where structural relief varies from medium to steep mountains to low-lying bottom land between them.

The locations of each 6-in. (15 cm) borehole monitoring station utilizing the mechanical grouting system and inclinometer installations are shown in Figure 1. The site exists in an area where the overburden rarely exceeds 300 feet (91 m) in the mountainous region and generally is near 110 feet (34 m) in the valleys. Mining is being performed in the Redstone Coalbed. The panels under investigation are 490–510 feet (149–155 m) in width and 3250–3850 feet (991–1173 m) in length. Mining height varies within each panel, but predominately is 72 inches (1.83 m).

The inclinometer borehole was strategically placed in the headgate section of the longwall panel as can be seen in Figure 1. This area was chosen to provide information into the mechanisms associated with lateral movements and rotational tilt as a function of panel advance, and to avoid distraction of the casing due to the large displacements, fractures, and cracks developing within the panel.

SITE INSTRUMENTATION

Each mechanical grouted borehole site consisted of a subsurface anchoring assembly, an above ground monitoring platform, and subsidence monitoring lines both longitudinally and transversely to the direction of mining. A pre-mining surface elevation survey was performed at each test site to establish an elevation datum, and successive vertical subsidence surveys were performed during face advancement. Vertical surveys were performed due to the anticipated subsidence to allow differentiation between surface ground movements and subsurface strata activity. Two boreholes were instrumented with an eight anchor assembly, as illustrated in Figure 2a, with the physical assembly shown in Figure 2b, which allowed measurements of vertical displacements within the chosen stratum. Vertical displacement measurements were made possible by direct

readings taken at the surface by graduated scales attached to the monitoring platforms, as shown in Figure 3. The intervals selected within each borehole for anchor placement were chosen following geologic analysis of drill logs. An attempt was made to place each anchor in a distinct stratigraphic member, and to avoid strata interfaces, as this was where caving and bed separation are considered most likely to occur.

The Sondex settlement probe system, which was used for measuring vertical strata deformation in the inclinometer installation, consists of a down-hole probe, electrical cable reel, batteries, controls, and steel surveying tape. The probe operates within flexible corrugated pipe with stainless steel sensing rings coupled to the surrounding soil. Electronic circuitry within the probe detects the location of rings by induction principle to the nearest 0.1 inch (0.25 cm). Lateral strata movements were calculated from inclinometer probe measurements. These measurements were obtained from a borehole which was drilled to a depth of 105 feet (32 m), through the Redstone Coalbed. Survey lines were also established in the vicinity of the existing inclinometer station to detect the amount of vertical subsidence and the total lateral movement surrounding the station. The inclinometer system used at the test site consists of four components including the inclinometer casing, sensor, control cable, and indicator.

LOCAL GEOLOGY

An assessment of the geologic and mechanical properties of the entire unit of strata overlying the longwall panels was conducted to relate these characteristics to caving height and lateral movement. To accomplish this, geophysical logging was carried out in borehole BH2 in an attempt to get a representative account of the lithology over both panels. In addition to geophysical techniques, a corehole taken in the area of the two boreholes was obtained for laboratory testing and used for the purpose of analytical study.

Borehole BH1 is located on Panel 2, as indicated by Figure 1, and was drilled to the top of the coal seam. The surface at BH1 had a mean sea level elevation of 1571.67 feet (478 m) with a borehole depth of 208 feet (63 m). This final depth was approximately 3 feet (91 cm) above the top of the Redstone Coalbed. The geologic column of the overburden was obtained from descriptive logging of the borehole as drilling was being performed. The strata are composed of interbedded shale, siltstone, and sandstone units with massive sandy shale formations. A generalized geologic column showing the location of each anchor is presented in Figure 4.

Borehole BH2 is located on Panel 3, as indicated by Figure 1, and was drilled to the top of the coal seam. The surface at BH2 had a mean sea level elevation of 1582.34 feet (481 m) with a borehole depth of 216 feet (66 m). This final depth was approximately 1 foot (2.54 cm) below the roof of

the Redstone Coalbed. The strata in BH2 are composed of interbedded shale, siltstone, and sandstone units with no extremely massive formations. The generalized geologic column showing anchor locations in Figure 5, indicates that lithology over the adjacent panels vary considerably in the immediate roof and lower main roof area.

The inclinometer borehole located in the headgate area between Panel 2 and Panel 3, as indicated in Figure 1, was drilled through a chain pillar and down into the underlying Weston Sandstone formation. The final depth was approximately 105 feet (32 m) or 4 feet (122 cm) below the floor of the Redstone Coalbed. The strata at this hole are similar to the lower strata found in the other boreholes and can be seen in Figure 6.

RESULTS AND DISCUSSIONS

Surface Ground Movements

A detailed analysis of surface ground movements associated with this site has been presented elsewhere (9), however, a brief analysis of monument lines established near subsurface instrumentation is presented in this report to differentiate between surface movements and subsurface activity. Final ground movements along Panel 3 section A-A', in Figure 1, are presented in a vertical subsidence profile as shown by Figure 7. The results indicate that the maximum subsidence observed on Panel 3 in the longitudinal direction are 4.325 feet (132 cm), with the average vertical displacement being 4.113 feet (125 cm). Using the average value for maximum subsidence and relating this to the mining height reveals a percent subsidence of 68.6 percent.

Transverse subsidence profiles measured in areas containing subsurface instrumentation are shown in Figures 8, 9, and 10, and correspond to Figure 1, sections B-B', C-C', and D-D', respectively. Each transverse subsidence profile reveals that the longwall panels under consideration are in supercritical conditions as revealed by the flat area present in each subsidence basin and indicates that maximum possible subsidence is being experienced by subsurface instrumentation. Each subsidence profile exhibited unusually sharp vertical movements at the edge of the panels creating unique subsidence basins unlike the commonly observed parabolic shape. The cause of such phenomenon is most likely due to the shallow depth of cover in the areas, as well as relatively weak strata in the overburden. This abrupt vertical movement resulted in the development of numerous tension cracks as observed at many locations near the panel's edge.

By observing Figure 9, it is apparent that an additional amount of vertical movement in the order of 5 inches (13 cm), occurred near the tailgate side of Panel 2. The cause of this additional subsidence is due to excessive

convergence on the pillars in the tailgate section between Panel 1 and Panel 2, and resulted in the non-symmetric subsidence profile.

Subsurface Ground Movements

BH1 - Initial anchor movement in borehole BH1 was detected in all anchors when the face has surpassed the borehole a distance of 3 feet (91 cm). This initial subsidence of all anchors, which was approximately 0.5 inches (1.27 cm), can be explained by overburden settling onto the shield supports and floor settling due to high stress and can be related to vertical subsidence evident on the surface, Figure 11. The piezometer tube data for the lower, middle, and upper levels in Figure 12 (7), correspond to subsurface anchors 5, 6, and 7, respectively, and reveal that subsurface activity is occurring even though vertical displacements have not yet begun. An initial dewatering of upper level strata was present even before the longwall face passed beneath the hole, indicating that minor fracturing of the overburden was present.

When the longwall face had passed the borehole a distance of 25 feet (8 m), an initial bed separation was evident between anchors 4 and 5, and was probably due to weak interfaces between strata lithology, as can be seen in Figure 13. Most likely, this failure occurred at a depth of 131 feet (40 m), or just below the massive lower Sewickley sandy shale formation. Only a few inches of vertical movement was detected from anchors 5-8 and suggests that minimal sagging of the upper level strata was occurring. Relating this small movement in the upper levels to dewatering information, it is evident that the effects of the extraction on upper levels are not being fully represented by vertical displacement data. Analyzing dewatering information in Figure 12 (7), reveals that major fractures have already developed in the upper levels of overburden at a face position of 20 feet (6 m) past the instrumentation site. At a face position of 20 feet (6 m) water levels at anchor 5 and 6 indicate that a drop in water level of magnitudes in excess of 30 and 20 feet (9 and 6 m), respectively. Using this transition zone as a possible caving horizon and relating that height to the immediate roof, an attempt to define the bulking factor of the caved strata for an extraction height of 6 feet (2 m) has been made using the following equation (8).

$$H = h_{im}(K - 1)$$

Where: H = mining height
 h_{im} = thickness of immediate roof or caving height.
 K = bulking factor.

Substitution yields:

$$K = \frac{H}{h_{im}} + 1$$

The caving horizon of 38 feet (12 m), which corresponds to the height of the bed separation from the coal seam, yields a bulking factor of 1.17. The bulking factor of 1.17 for the 38 feet (12 m) horizon does not correspond to the commonly used values of 1.60 - 1.80 for the bulking factor of sandy shale, or 1.40 for clay shale (8). Therefore, initial caving of the immediate roof is assumed to have been extremely uniform with little breakage and expansion of the strata. This can partially be explained by large blocks failure occurring along planes of weakness or changes in lithology resulting in massive failure of large portions creating an orderly arrangement of the gob material.

When the face had advanced a distance of 30 feet (9 m) beyond the instrumentation site, larger vertical movements were evident and the rate at which they were occurring was much higher. Observing Figure 12 (7), it is evident that extremely large lateral strata movements were occurring at this time, and as a result, completely pinched off the piezometer pipes preventing water measurements to be made.

When the face had advanced a distance of 50 feet (15 m) beyond the instrumentation site, the bed separation between anchors 4 and 5 was at a maximum, and magnitude was in the order of 15 inches (38 cm). Anchors 1-4 had average vertical displacements of 28 inches (71 cm), while anchors 5-8 averaged only 13 inches (33 cm) total vertical movement.

Although the maximum vertical subsidence measured on the surface was 47.65 inches (121 cm), total vertical movement of anchor 8 was 48.46 inches (123 cm). This discrepancy most surely is due to an apparent slippage of the anchors. The surface at BH1 after mining, as shown by Figure 14, reveals that numerous tension cracks developed in the area of the borehole and resulted in literally separating the borehole into two sections. This may have resulted in the slippage of anchors by reducing the bonding strength of the grout to the borehole wall.

BH2 - As the face approached BH2, readings on the surface began two days before the face reached the borehole. Initial anchor movement was detected in anchors 1-6 when the face was at a position of 2 feet (61 cm) past the instrumentation site, and was only in magnitude of 0.2 inches (0.5 cm).

Small anchor movements continued to be recorded hourly until the face was 14 feet (4 m) past the station. At that time, anchors 1-4 began caving at an extremely fast pace, while anchors 5-8 showed little movement, therefore an initial bed separation was evident. This high velocity of strata movement is opposite of what was observed in BH1, being more closely related to the actual theory of caving behind the face supports. The sudden drop of strata that is evident in the strata displacement curves in Figure 15, can be explained by underground observations of the caving sequence. In the area of the borehole, a small overhang developed behind the face

supports eliminating the effects of gob piling on the back of the gob shield. This resulted in the full void of 6 feet (2 m), corresponding to the extraction height, allowed for roof caving material to collect.

When the longwall face had passed the borehole by 30 feet (9 m), anchors 5-8 began to show some sagging of the main roof, as can be seen in Figure 15. The differential movement observed between the lower four anchors and the upper four anchors revealed a bed separation in the order of 43 inches (109 cm), and most likely was occurring just below the Lower Sewickley sandstone formation, as indicated by descriptive geologic logging of the borehole. This transition zone was evident by differences in strata lithology, i.e., progressing from the weak gray shale of the immediate roof to a more competent sandstone formation.

The caving horizon of 60 feet (18 m) corresponding to the bottom of the Lower Sewickley formation yields a bulking factor of 1.10, and also does not correlate to commonly used values for bulking factors of shale and sandy shale. This also leads to the conclusion that large blocks of strata were caving into the void left by the extraction and were collecting in an orderly manner.

When the longwall face had advanced a distance of 50 feet (15 m) past the instrumentation site, movement in anchors 1-4 had all but ceased, and only residual displacement of gob compaction were being detected. Anchors 5-8 had only displaced an average of 18.3 inches (46 cm), therefore the large bed separation had begun to close. The subsidence development curve shown in Figure 16, indicates that vertical movements of the surface began when the instrumentation site was undermined. The maximum vertical subsidence measured at the surface was 43.34 inches (110 cm), and total vertical movement of anchor 8 was 43.87 (111 cm), therefore anchor slippage is also present in this instrumentation site, as was BH1. The amount of anchor slippage is a considerable amount less than was observed in BH1 and can be attributed to less fracturing occurring in the vicinity of the borehole.

Inclinometer Borehole

The inclinometer probe measured the angle of tilt of the casing within the borehole in two directions, parallel and perpendicular to the centerline of the panel. The following discussion will refer to the A and B direction. The A direction is perpendicular to the direction of mining. Positive A deflections refer to movements in the direction of the extraction or toward the gob area and negative deflections are in the opposite direction toward the headgate side or solid coal side. The B direction refers to deflections parallel to the direction of mining. Positive B deflections are in a direction opposite to panel advancing direction and negative deflections refer to the mine advancing direction.

Initial readings were established 450 feet (137 m) in advance of the approaching longwall face. Readings were taken periodically as the longwall face approached the instrumentation site, and initial lateral movement was detected when the face approached within a distance of 50 feet (15 m) of the borehole. Figure 17a shows the initial lateral displacements measured within the inclinometer casing as a function of face advance, which was 50 feet (15 m) away from the instrumentation site when the displacements were recorded.

Figure 17b shows final lateral movements as affected by longwall mining, and only residual movements are being encountered. The large negative lateral movement in the A direction down to a depth of 25 feet (8 m) indicates that the surface topography was highly affecting the subsurface ground movements. The high topographical relief area near the borehole resulted in a zone of high compression in the topographical low area in which the instrumentation site is located. This compression zone caused major surface movement at a relatively shallow depth, opposite the direction of the caved area, and had little effect on the lower strata. By correlating this large movement to the lithology in the area, it is evident that the zone of weathered shale above the bedrock extended down to approximately 25 feet (8 m). This zone suffered the effects of the increased pressure created by the hillside moving to a lower elevation, and partially explains the phenomena of surface movement observed by other researchers. Khair (9), reported that while the crest of a hillside is subjected to extreme tension, the foothill experiences extremely high zones of compression. Pomeroy (10), found that ground movement seems to occur in areas where beds are a mix of weak zones and strong zones, or along beds of weathered shale. Figure 18 presents the displacement vectors observed from surface measurements performed on the subsidence monuments placed longitudinally and transversely to face advancing direction in the area of the inclinometer study. When the topography became relatively mild, the magnitudes of lateral movement decreased, but were still in a direction indicating that the hillside was creating the movement. By differentiating the horizontal movement of adjacent stations, the strain profile in Figure 19, section D-D' of Figure 1, reveals the high zone of compression at the foot hill near the borehole and substantiates previous conclusions of subsurface activity being dictated by topographical features.

The Sondex settlement probe was used for monitoring of full length borehole extension each time measurements were taken by the inclinometer. The Sondex was used to record any vertical strata movements taking place as the longwall face advanced. From the accuracy provided by the Sondex, to the nearest 0.1 inch (0.25 cm), no detectable movements were observed over the chain pillar adjacent to the active longwall panel. Past observation and measurements (9), experienced at the site revealed that strata breakage was easy along the panel's edge,

therefore, less load transfer was present on the chain pillar area which further decreased the chance of massive vertical movements. From the surface subsidence profile in the area of the inclinometer borehole, Figure 10, the fact that no vertical movements detected by the Sondex is substantiated by surface surveys.

FINITE ELEMENT ANALYSIS

An attempt was made to identify possible zones of strata separation using finite element method, employing the existing NASTRAN program (developed by NASA). A finite element simulation of each mechanical grouted borehole has been prepared. A typical grid used for modelling the overburden contains numerous isoparametric elements discretized by their related nodal points as can be seen in Figure 20. Material properties were assumed constant throughout individual horizontal layers. The elastic properties used were those evaluated by testing a number of core specimens as mentioned earlier in this study, and the properties used for the generalized strata are given in Table 1. In an attempt to best simulate the overburden at the test site, three finite element simulations were performed.

Simulation 1 (BH1), consisted of the traditional 2-D analysis where only a dome created by the caving process with a reduction of material properties to represent gob material was used. The simulation assumed a dome area created by initial caving was completely filled with gob material, and beam sagging and separation due to the weight of the overlying overburden was measured. Results from Simulation 1 (BH1), indicated that the caving process was not truly being represented due to small values being displayed as displacement vectors. The displacements of the isoparametric elements corresponding to the location of the mechanical anchors in the field study are shown in Table 2, and as can be seen, do not correspond well with actual field measurements.

From Simulation 1 (BH1), it was evident that the linear elastic solution provided by NASTRAN does not allow the actual failure along bedding planes to be simulated. Therefore, joint elements were introduced between the major strata to reduce the stiffness of the elements at the area where possible bed separation might occur. Extremely reduced mechanical properties were assigned to the joint elements, however, the mechanical properties were unchanged for the remainder of the model. These elastic properties for joint elements were set to be 1000 times less than those of the intact strata elements, which has also been used by other researchers (11). These reduced properties are associated with zones of weak weathered shale interbedding that have been recorded from field observations.

Results from Simulation 2 (BH1), can be seen in Table 3, and in contrast to Table 2, the introduction of weak joint elements between the major strata appear to closely resemble actual field

measurements. It must be noted that displacement of anchors 1-3 are slightly less than field measurements indicated, but as mentioned before, it is thought that block rolling during caving elevated the measured values to some extent. The bed separation that was evident from field measurements is simulated in terms of higher displacements by finite element analysis corresponding to the area between anchors 4-5.

Using the information obtained from Simulation 2 (BH1), analysis of the dome created by the caving process combined with the introduction of joint elements between major strata was performed in Simulation 3. Results from Simulation 3 (BH2), indicate that implementation of joint elements better simulate strata activity when trying to analyze the caving sequence created by a longwall extraction. Table 4 presents the displacement of the isoparametric elements corresponding to each anchor, and reveals the extremely large vertical displacements that were indicated by field measurements. The apparent bridging of the Lower Sewickley sandstone most surely developed due to its high tensile strength when compared to the other units present in the overburden.

CONCLUSIONS

The mechanical grouting method has proven to be a very useful and inexpensive technique for monitoring vertical strata movements and bed separations resulting from longwall extractions. Dual purpose measurements, made possible by piping, proved that more information can be obtained from such an installation, and thereby, increasing the feasibility of an underground study using the mechanical grouting method.

The bulking factors determined in this study are much less than work done previously for the same geologic material. The reason for this discrepancy is probably due to inexact identification of the caving zone caused by wide spacing between anchors.

The reaction of the overburden, and its caving characteristics, are highly dependent upon site specific geologic factors. An overburden characteristic that can strongly affect the caving response is the presence of a thick strong layer within the overburden. This stronger layer, when compared to other units in the overburden, creates a bridging affect over the upper strata and hence, limits the amount of vertical displacement over the extraction. When conditions are unfavorable, this bridging affect can cause undesirable high abutment pressures to be placed upon the adjacent solid coal.

Data obtained utilizing the inclinometer revealed another characteristic of longwall strata behavior, and suggests that topography also affects the movement of subsurface structures. A topographical high area can cause extreme compression zones on low lying areas, and affect subsurface lateral movements some distance below

the surface. When this condition exists in areas over the gate road entries, it is possibly a contributing factor in the unpredictable behavior of roof characteristics in gate entries, especially in the tailgate area. The instrumentation site used for this study was in the headgate section, and future studies should be performed in the tailgate area where roof problems are more prominent.

From finite element simulations, it is evident that the linear elastic solution does not allow the actual failure along bedding planes to be simulated. The introduction of joint elements with reduced material properties provide a better approximation when trying to analyze the caving sequence created by a longwall extraction.

ACKNOWLEDGEMENTS

The authors acknowledge the assistance of N. P. Reddy, S. J. Jung, Y. S. Ro, P. Molesky, and P. M. Lin, graduate students in the Mining Engineering Department, for their help in obtaining field data. This research was performed as Task No. 4 of an integrated project entitled "An Improved Longwall Mining Technique" directed by the Longwall Mining and Ground Control Research Center and sponsored by WVU's Energy and Water Research Center.

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Table 1. Material properties of each strata used in the finite element analysis, based upon laboratory testing.

Material	Young's Modulus (PSF)	Poisson's Ratio	Density (LBS/CFT)
Sandstone	3.96×10^8	0.11	160.00
Coal	1.29×10^8	0.44	90.00
Shale	1.58×10^8	0.19	160.00
Sandy Shale	2.22×10^8	0.39	156.00
Weathered Shale	7.20×10^3	0.45	100.00

Table 2. Vertical displacements of elements corresponding to the location of anchors in BH1, Simulation 1.

Anchor	Finite Element (IN)	Measured (IN)	Difference (IN)
1	32.6	55.6	23.0
2	29.8	53.8	24.0
3	27.9	52.5	24.6
4	26.5	51.8	25.3
5	22.1	50.2	28.1
6	21.5	48.6	27.1
7	20.3	48.1	27.8
8	19.4	47.7	28.3

Table 3. Vertical displacements of elements corresponding to the location of anchors in BH1, Simulation 2.

Anchor	Finite Element (IN)	Measured (IN)	Difference (IN)
1	53.7	55.6	1.9
2	52.5	53.8	1.3
3	50.8	52.5	1.7
4	49.3	51.8	2.5
5	46.7	50.2	3.5
6	46.1	48.6	2.5
7	45.5	48.1	2.6
8	44.9	47.7	2.8

Table 4. Vertical displacements of elements corresponding to the location of anchors in BH2, Simulation 3.

Anchor	Finite Element (IN)	Measured (IN)	Difference (IN)
1	68.1	79.4	11.3
2	66.9	72.2	5.3
3	64.9	68.4	3.5
4	60.2	65.1	4.9
5	52.7	47.9	4.8
6	48.1	45.1	3.0
7	47.5	44.8	2.7
8	45.6	43.9	1.7

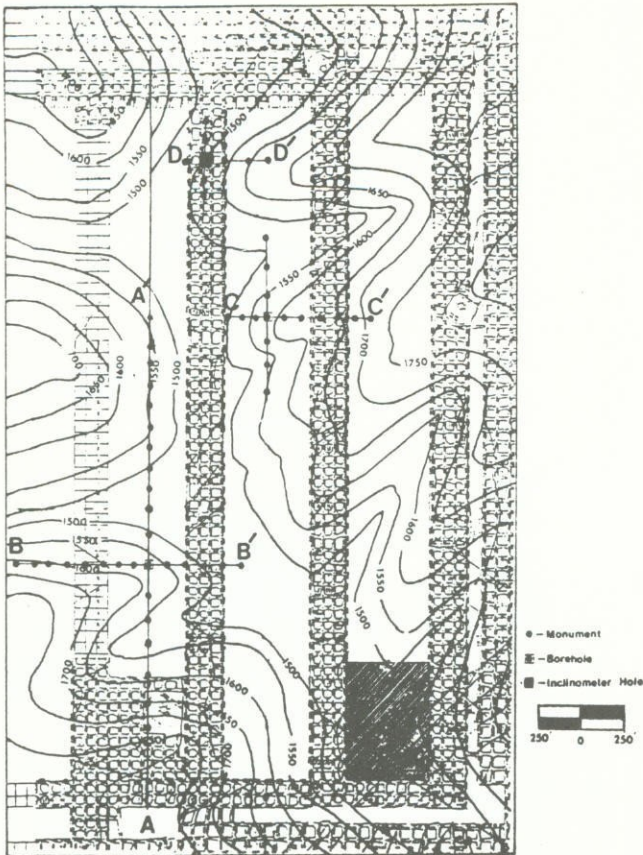


Fig. 1. Shows the panel layout and locations of monitoring lines and borehole installations.

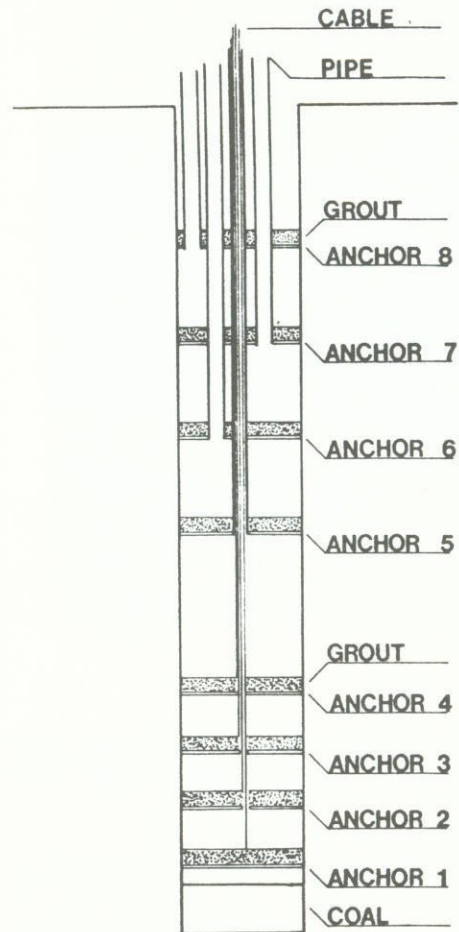


Fig. 2a. Illustrates the mechanical grouting assembly used for each instrumentation site.

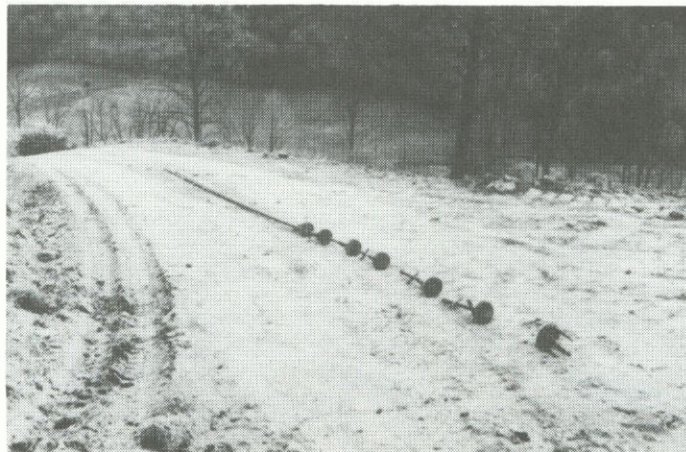


Fig. 2b. Shows the physical assembly used at each test site.

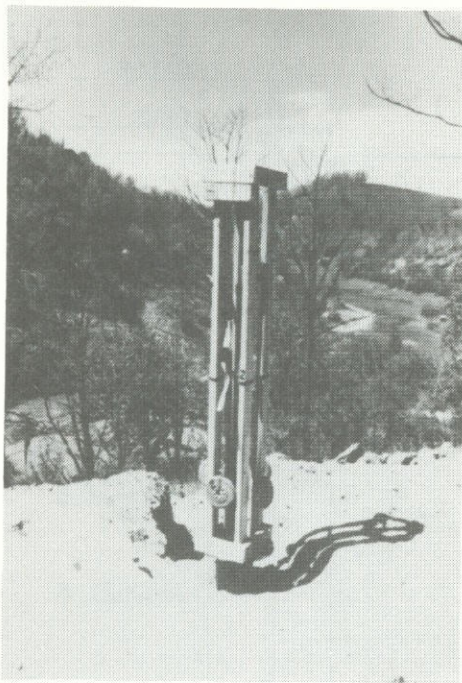


Fig. 3. Shows the typical monitoring structure used for support of measurement instruments.

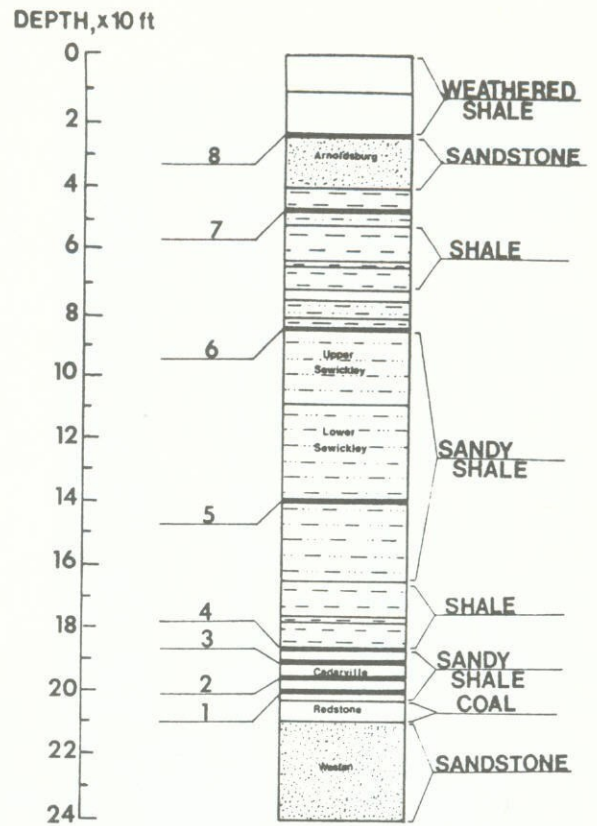


Fig. 4. Show the generalized geologic column and location of anchors in borehole BH1.

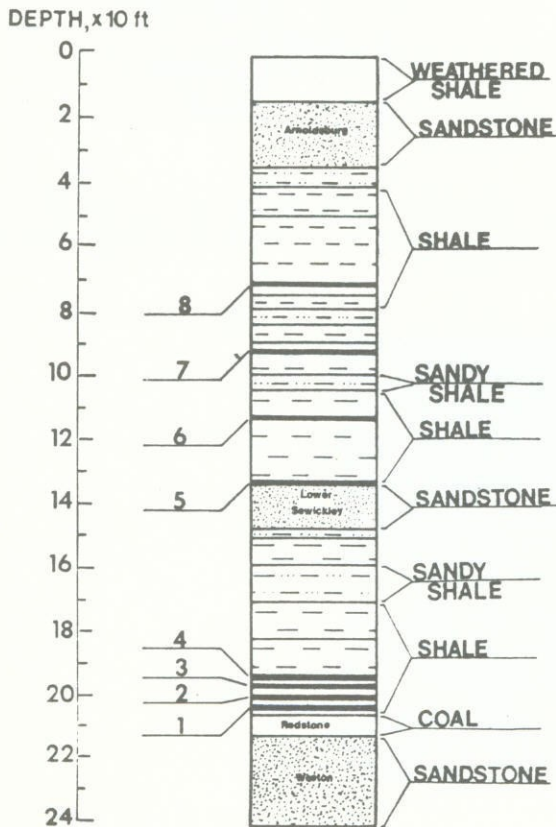


Fig. 5. Shows the generalized geologic column and location of anchors in borehole BH2.

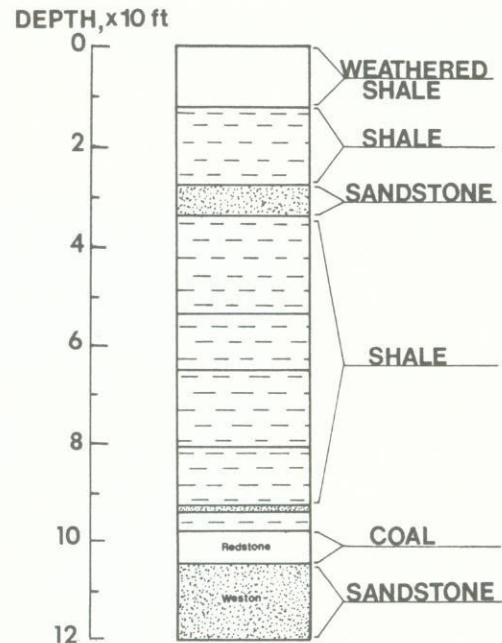


Fig. 6. Shows the geologic column corresponding to the area adjacent to the inclinometer borehole.

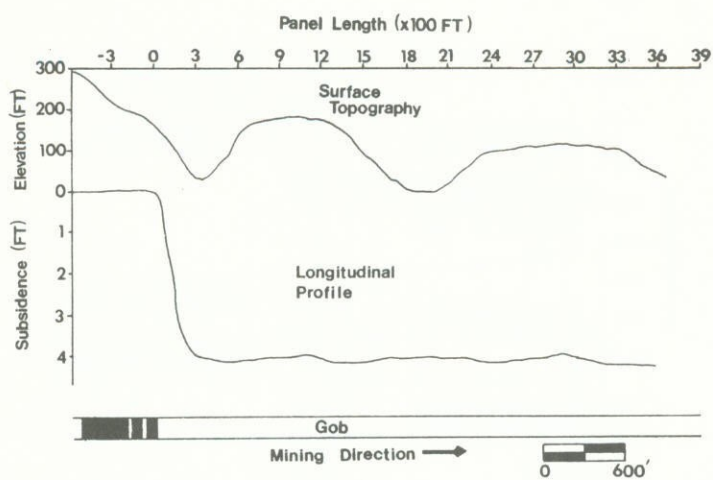


Fig. 7. Shows final vertical subsidence along Panel 3 section A-A', in Figure 1.

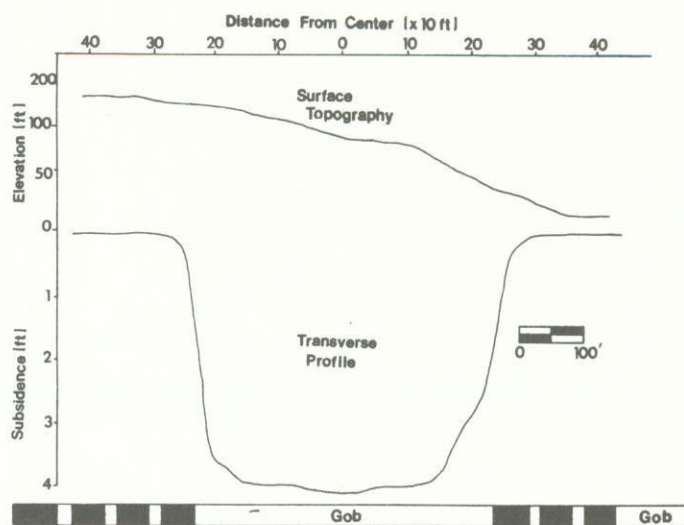


Fig. 8. Shows final vertical subsidence along transverse section B-B' (borehole BH1), in Figure 1.

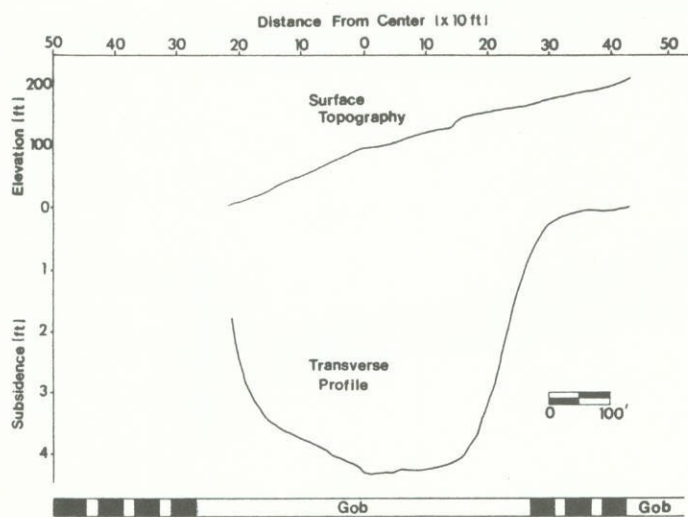


Figure 9. Shows final vertical subsidence along transverse section C-C' (borehole BH2), in Figure 1.

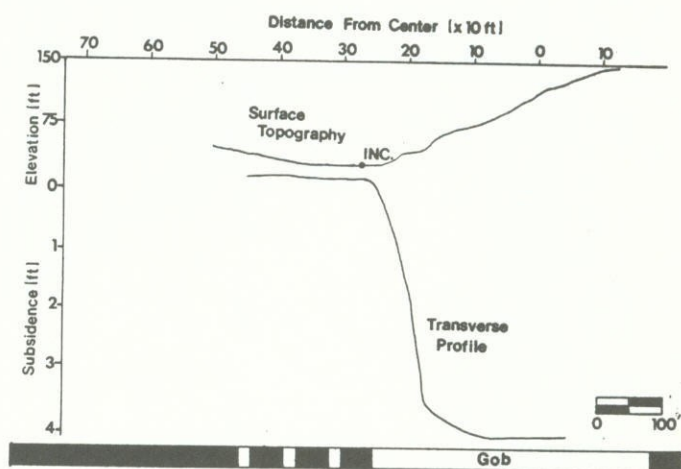


Fig. 10. Shows final vertical subsidence along transverse section D-D' (inclinometer borehole), in Figure 1.

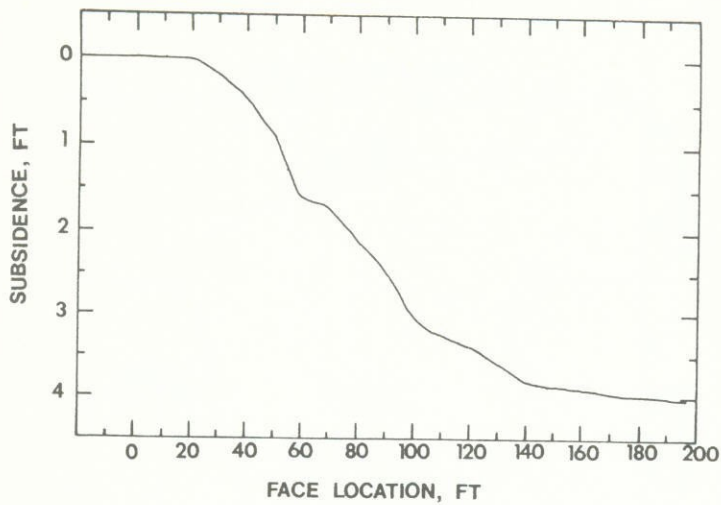


Fig. 11. Shows the vertical subsidence development curve as a function of face advance for BH1.

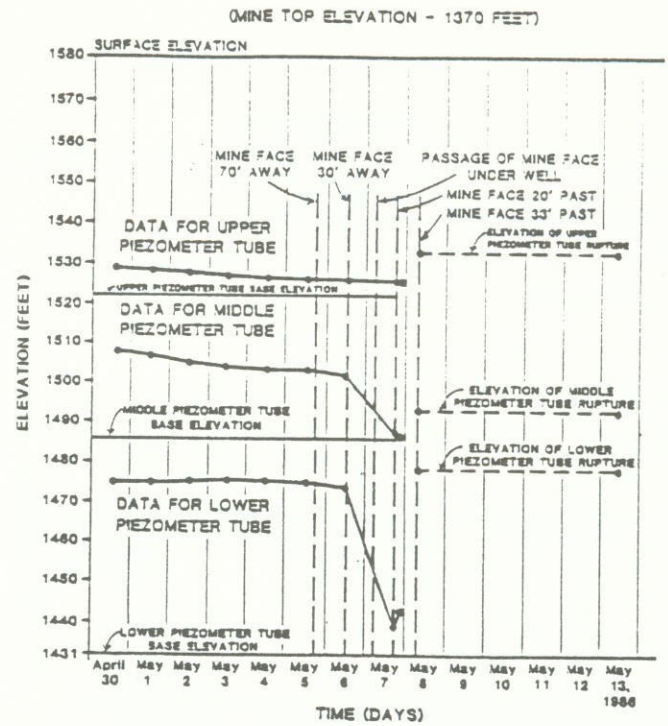


Fig. 12. Shows piezometer tube data collected from strata in BH1 (7).

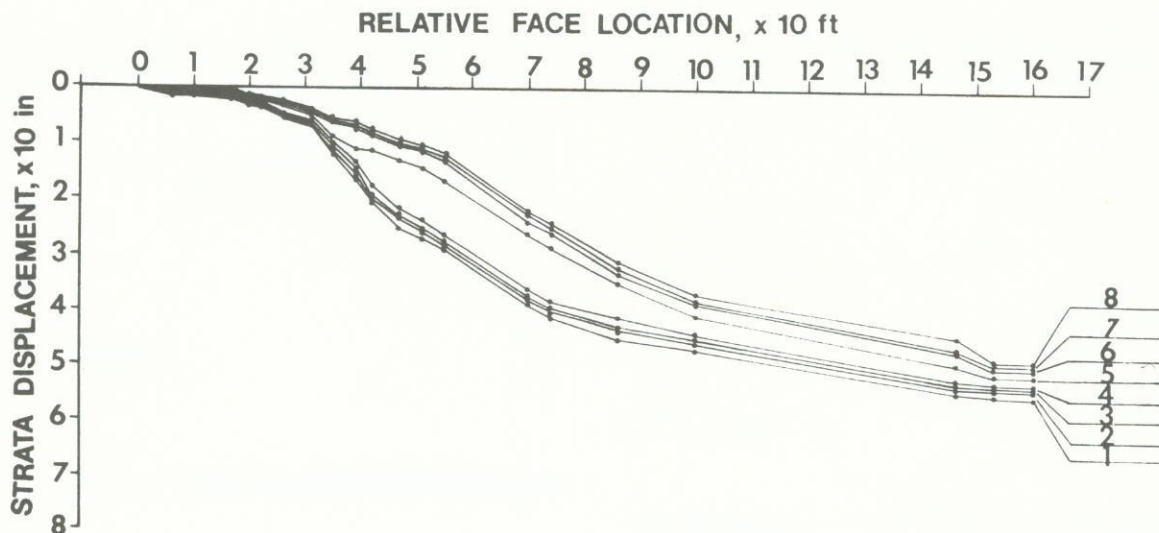


Fig. 13. Shows anchor displacement development curves obtained from the mechanical grouted assembly in BH1.



Fig. 14. Shows numerous tension cracks on the surface during monitoring of BH1.

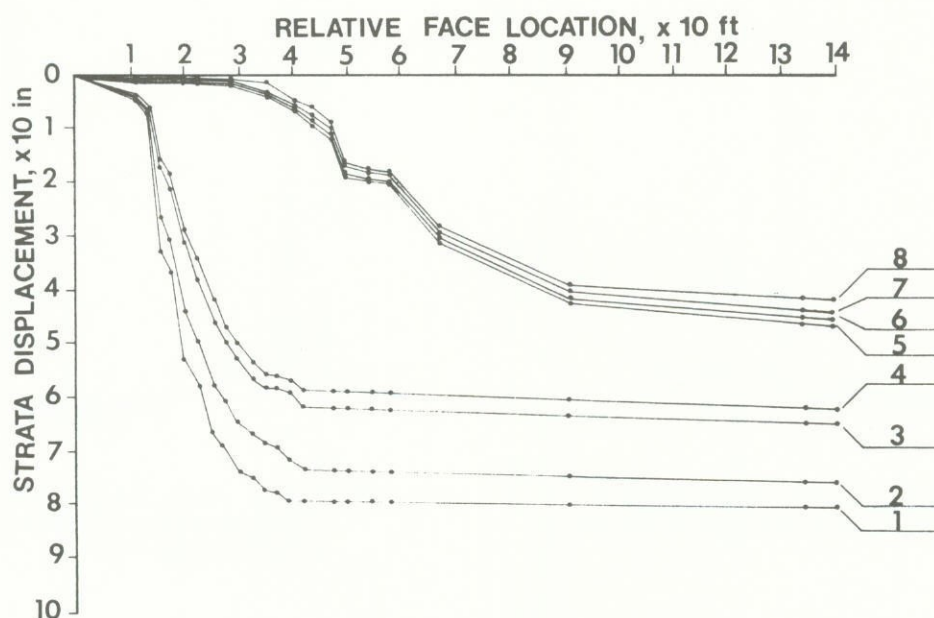


Fig. 15. Shows anchor displacement development curves obtained from the mechanical grouted assembly in BH2.

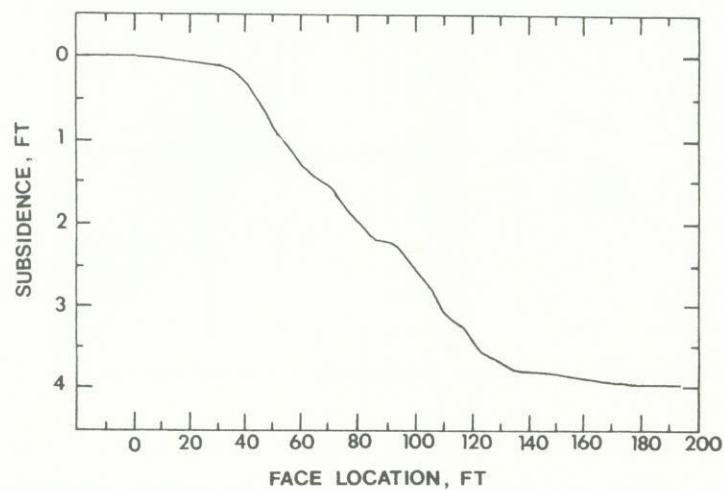


Fig. 16. Shows the vertical subsidence development curve as a function of face advance for BH2.

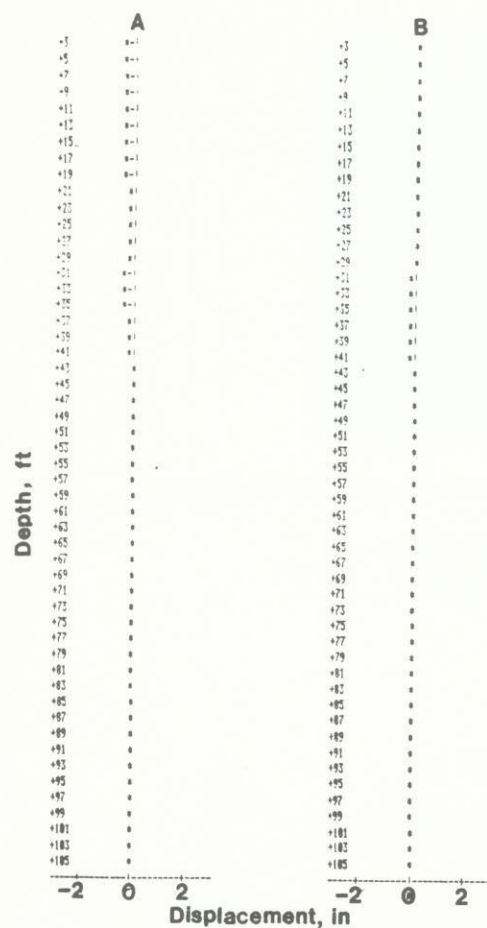


Fig. 17a. Shows initial lateral displacements at a longwall face position of 50-ft.

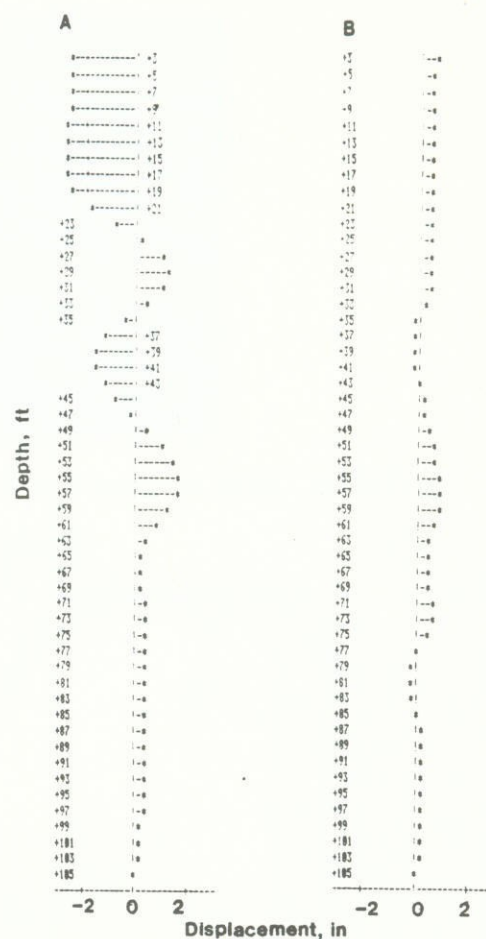


Fig. 17b. Shows final lateral displacements at a face position of 250-ft. past the inclinometer hole.

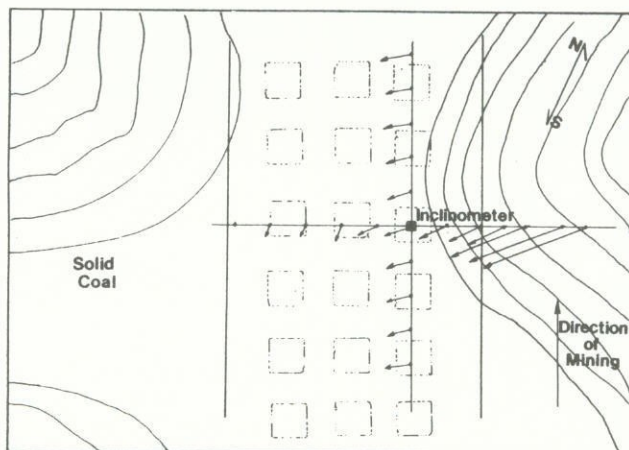


Fig. 18. Shows the displacement vectors of typical subsidence monuments near the inclinometer hole.

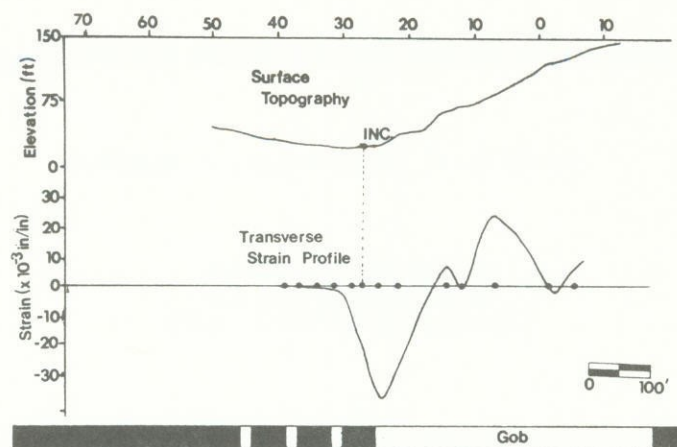


Fig. 19. Shows strain profile for section D-D' (inclinometer borehole), in Figure 1.

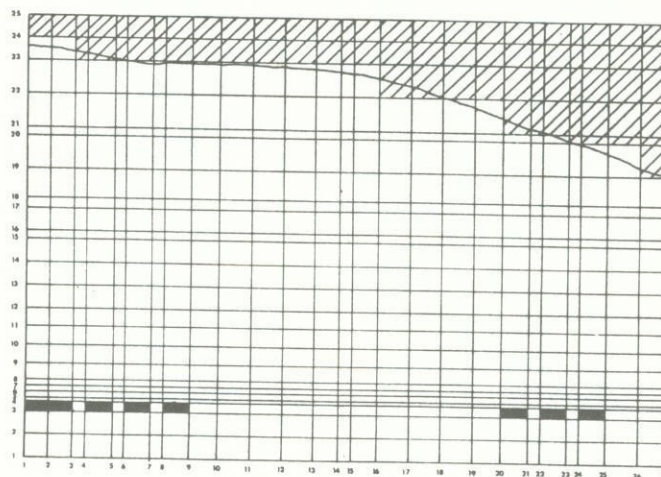


Fig. 20. Shows the finite element mesh used for simulation of caving in each borehole.

FIELD MEASUREMENTS OF OVERBURDEN AND CHAIN PILLAR RESPONSE TO LONGWALL MINING

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ABSTRACT

Current longwall pillar design relies heavily upon past experience and many assumptions about pillar and entry response to longwall abutment pressures. Caving characteristics of the immediate and main roof strata are known to play an important role in the overall pressure redistribution, which determines the maximum abutment pressure, abutment pressure at the T-junction, and the width of influence zone.

This paper presents the results of an extensive geomechanical study in a longwall panel at a West Virginia coal mine. The response of eight headgate pillars and the adjacent entries was monitored as the longwall face approached and passed by the instrumented locations. Overburden response to longwall mining was also monitored, and height of the highly fractured zone above the gob was determined by postmining drilling. Good agreement was observed between the measured stress changes and those computed using the known height of the highly fractured zone. The results provide a detailed picture of the response of overburden strata and chain pillars to longwall mining.

THE FULL TEXT IS IN PAGES 296-311

A NEW METHOD FOR SIZING LONGWALL PILLARS
BASED ON FIELD MEASUREMENTS

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ABSTRACT

A new method for the design of longwall pillars, called "Analysis of Longwall Pillar Stability" (ALPS), is described. The method is the final result of a research program that included two major field studies, extensive reanalysis of data from other in situ measurements, and detailed evaluations of available pillar design methods.

The key problem addressed by ALPS is estimation of the abutment loads applied to longwall pillars. Based on underground stress measurements made in mines in three states, equations for predicting the magnitude, distribution, and time-of-arrival of abutment loads are proposed. Stress measurements are also used to evaluate empirical pillar strength formulas for use in longwall pillar design. ALPS has been verified by application to case histories from several mining areas, and examples taken from the Pittsburgh and the Blue Creek seams are presented. Step-by-step guidelines for using this design method are also given.

INTRODUCTION

Longwall mining is increasingly being considered essential to the future of underground coal mining. While longwalls have tremendous productivity potential, the success of any given longwall face depends on adequate ground control in the gate entries, which are the lifelines of a mine. The gate entries also serve as emergency escapeways from the face.

The primary function of longwall pillars is to maintain the stability of the gate entries. The design of chain

pillars must address different stability requirements and different load conditions during the progression of longwall mining. A typical chain pillar system will be subjected to two sets of longwall loadings during its service life, first as a headgate and then as a tailgate. Other chain pillars, those located at the edges of multi-panel longwall blocks, will be subjected to only a single longwall loading as they protect headgate, tailgate, or bleeder entries. Longwall pillar design also includes sizing barrier pillars to protect the main entries from the adverse effects of "peep-through" longwall loadings.

Two basic philosophies of longwall pillar design have been described (Dahl, 1978). In conventional, or "stiff", design, the chain pillars are sized to carry all the abutment load to which they will be subjected. Conventional designs may employ equal-sized pillars, or combinations of large and small pillars. The other philosophy, called "yielding" design, uses only very small pillars which transfer load. Yielding designs are commonly used in Western longwalls, but have only recently been attempted in the Eastern coalfields (Gauna et al., 1986).

The key assumption of conventional longwall pillar design is that unstable pillars will result in unstable gate entries. Experience has in fact provided many examples of mines where undersized pillars have been associated with intolerable entry conditions. In many cases, once the chain pillar sizes were increased, ground conditions in the gate entries improved dramatically.

Undersized chain pillars are thought to cause ground control problems because they attract high vertical stresses, which in turn cause high shear stresses near the entry (Serata et al., 1986).

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Highly stressed pillars may also punch into the roof or floor, or they may suffer excessive rib spalling (Moebs and Stateham, 1985). Differential pillar displacements in response to vertical loads may also result in roof failure (Dahl, 1978). Larger pillars can improve ground conditions because they are better able to distribute abutment loads, resulting in lower average pillar stresses, lower bearing pressures, and smaller displacements.

In spite of the growing importance of longwall mining, relatively few quantitative guidelines for the sizing of longwall pillars have been developed (Choi and McCain, 1980; Carr and Wilson, 1982; Lu, 1983; Hsuing and Peng, 1985). Recent research at Penn State has attempted to fill this gap through the development of an empirical design approach for longwall pillars called ALPS (for "Analysis of Longwall Pillar Stability"). As a method for conventional longwall pillar design, ALPS allows for pillars to be sized to carry the abutment loads. There are three basic components of ALPS:

- . estimation of the load applied to the pillars
- . estimation of the pillar strength and load-bearing capacity
- . calculation of "safety factors" (SF) based on a comparison of the load to the load-bearing capacity.

LONGWALL PILLAR LOADS

The loads applied to longwall pillars may be divided into two parts. These are development loads, which are present before longwall mining, and abutment loads, which arrive during longwall panel extraction. Development loads are very similar to the loads applied to pillars in room-and-pillar mining. Abutment loads are unique to retreat mining, and are the key to longwall pillar design.

Development loads are due to the weight of the overburden directly above the pillars and the gate entries. The "tributary area theory" has been used to estimate longwall development loads (Wilson, 1972; King and Whittaker, 1971). The tributary area expression for the development load per foot of gate entry (L_d) is:

$$L_d = (H) (W_t) (\gamma) \quad (1)$$

Where: H = depth of cover (ft)

W_t = center-to-center width of the gate pillar system (ft)

γ = unit weight of the overburden (pcf)

To check the validity of the tributary area theory for longwall pillar design, a series of two-dimensional, linear-elastic, finite-element models were run at Penn State using the finite-element program ANSYS (Mark, 1987). The parameters that were varied in the models included rock mass properties, horizontal stress, extraction ratio, and section geometry.

The model results indicated that tributary area theory provides a satisfactory estimate of the development loads on typical longwall pillars. The accuracy of the theory decreased as the extraction ratio increased, but the theory's predictions were within 10% of the model results for two-dimensional extraction ratios of up to 50%. The development loads observed in the model were also affected by less than 10% when the stiffness of the roof and floor was increased to ten times that of the coal, when horizontal stresses twice as great as the vertical were applied, and when unequal-sized pillars were used.

Abutment loads occur as a portion of the weight of the overburden that had been supported by the excavated longwall panel is transferred to the pillars. Abutment stress increases begin to be experienced several hundred feet before the arrival of the face. From a mining standpoint, the most critical abutment loads are those experienced by the pillars at the face ends or "T-junctions". In this paper, the pillar loads at the T-junctions will be referred to as the "front abutment" (L_f). After the face moves past, there is additional load transfer from the panel, and pillar loads continue to increase. Eventually an equilibrium is reached, and pillar loads stabilize at a final, maximum value called the "side abutment" (L_s).

The side abutment is much easier to analyze than the front abutment, because it may be treated in two dimensions. Two, similar analytical approaches have been proposed for estimating the side abutment. Wilson (1972) proposed that the vertical stress in the gob increases linearly, from zero at the rib to the original overburden pressure at some point within the gob. He estimated that the distance required for the gob pressure to return to cover load was typically 0.3 times the depth of cover (H). The side abutment may therefore be visualized as shown in the lower portion of figure 1.

The second analytical approach for estimating the side abutment, first proposed by King and Whittaker (1971), begins with the concept of a "shear angle" or "negative angle of draw" which determines the pillar loading. As shown in figure 1, the side abutment is represented as the wedge of strata defined by the shear angle β . King and Whittaker presented two equations for quantifying the side abutment, one for critical and supercritical panels (equation 2) where the panel width P exceeds twice ($H \tan \beta$), and the other for subcritical panels (equation 3).

$$L_s = H^2 (\tan \beta) (\gamma/2) \quad (2)$$

$$L_{ss} = \left[\frac{HP}{2} - \frac{P^2}{8 \tan \beta} \right] \gamma \quad (3)$$

Choi and McCain (1980), slightly modified King and Whittaker's method, and presented subsidence data which indicated that for the Pittsburgh seam $\beta=18^\circ$.

Although developed using different lines of reasoning, the methods of Wilson, King and Whittaker, and Choi and McCain become very similar in application. All assume that the weight of the overburden between the ribline and a point ($H \tan \beta$) into the gob is evenly split between the pillars and the gob. For Wilson's $0.3 H$, β works out to 16.7° , which is very near Choi and McCain's 18° .

While there undoubtedly is a relationship between subsidence and abutment load (Heasley and Saperstein, 1986), it seems that relationship may not be so straight-forward as is implied by King and Whittaker. Like its cousin the angle of draw, the negative angle of draw can be extremely difficult to determine in actual subsidence profiles. Nevertheless, the concept of an "abutment angle" equivalent to β is very useful for design. The abutment angle should not be considered a physical reality, but as an approximation which defines the side abutment in equations 2 and 3. To make these equations useful for design, it is necessary to determine the value of β which is appropriate for Eastern U.S. conditions. Field measurements of longwall pillar stresses provide the means of determining β .

The magnitude of the front abutment (L_f) is much more difficult to determine analytically than that of the side abutment. The load transfer at the face ends is a complicated three-dimensional problem, because the overburden load is shared between the pillars, the solid coal, and the gob. While three-dimensional finite-element modeling has

been used to analyse this problem (Hsuing and Peng, 1985), the mechanical properties of the coal and gob can be very difficult to determine.

An alternative approach begins with the concept that the front abutment is some fraction of the side abutment, and can be represented as:

$$L_f = F (L_s) \quad (4)$$

where F is a "front abutment factor" with a value of less than one. If it could be shown that F is approximately constant from panel to panel, then equation (4), in combination with equations (1), (2) and (3), may be used to determine the pillar loads at the T-junctions. In fact, different values of F may be necessary for the first and second front abutments. Stress measurements may again be used to determine appropriate values of F for design.

Another aspect of the abutment load prediction problem is the distribution of the load. When the first panel adjacent to a gate section is mined, the pillars may not carry the entire abutment load because some portion of it may be taken by the nearby barrier pillar or unmined panel. Peng and Chaing (1984) analysed field measurements to determine the width of the abutment influence zone (D). They found that abutment stress increases could be detected up to a distance from the panel edge of:

$$D = 9.3 \sqrt{H} \quad (5)$$

If the shape of the stress distribution within the abutment influence zone was known, then it would be possible to estimate the percent of the total front or side abutment load that is carried by the chain pillars. Airey (1977) proposed that within the influence zone the stress decay approximately follows an inverse square law. Field measurements of longwall pillar stresses can be used to test the hypothesis of an inverse square stress decay.

FIELD MEASUREMENTS OF LONGWALL ABUTMENT STRESSES

Stress measurements in longwall pillars may be used to evaluate abutment loads if the average pillar stress change occurring during longwall mining may be determined. Measurements from an individual stressmeter may not be representative of the average pillar stress change, because the stress distribution within a pillar is generally non-uniform. One solution to this problem is to install enough stressmeters in each instrumented pillar

that the average stress change can be estimated with some confidence.

The above approach has been adopted in several field studies performed during the last several years. At Mine X in Northern W. Va., Penn State instrumented pairs of headgate pillars in two successive longwall panels (Mark and Bieniawski, 1986a; Mark, 1987). The pillars in the first panel headgate were 42 ft wide, and in the second they were 22 ft wide. Arrays of 15 vibrating wire stressmeters (VWS) were installed in each pair of pillars. Other instrumentation included convergence, roof stress, horizontal pillar expansion, and differential roof sag. The instrumentation program for the second array is shown in figure 2.

In the Eastern U.S., other studies in which sufficient stressmeters were installed to estimate the average abutment stress have been reported by Schuerger (1985) and Allwes et.al., (1985). An early version of ALPS (Mark and Bieniawski, 1986b) was based largely on analyses of only the published data from these two studies. Recently the complete original data sets from both studies were made available to Penn State for more detailed analysis, the results of which are presented below.

The study described by Schuerger was conducted in Mine Y, located in Eastern Kentucky. Four pairs of pillars near the first longwall panel at the mine were instrumented with a total of over 80 VWS. Four pairs of longwall pillars were also instrumented with VWS in the study reported by Allwes et. al. Their study was conducted in Mine Z, located in Eastern Ohio.

The data from the Penn State studies and from the other two studies supports the assumption that the average pillar stress change can be estimated if several stressmeters are used. Stress distribution profiles from many of the instrumented pillars (figure 3) showed that at low stress levels the greatest stress increases were measured near the rib. Later, the stressmeters near the rib often lost load while the stress peak shifted to the pillar core. These results show that while no single stressmeter could have been expected to accurately portray the average pillar stress increase, a reasonable portrait of a pillar's stress history could be obtained using multiple VWS measurements.

In the three studies, data from a total of 10 stressmeter arrays are available for analyzing the magnitude of

the abutment loads. It was found that in all 10 cases enough data was collected to characterize the average stress change at the T-junction, called the front abutment stress (σ_f). Once the longwall had passed an array, individual meters were often lost and in some cases access to the entire array could not be maintained. As a result, valid data on the side abutment stress (σ_s), measured long after the face was passed, was only available from three arrays. Measurements of the second front abutment in the tailgate proved even more difficult to obtain, and were only available in one case.

The first aspect of the abutment load problem is to determine the distribution of the load between the chain pillars and the adjacent solid. Because the front abutment data set is the most complete, it was used in the analysis. First, the extent of the abutment zone D was calculated for each array using equation (5). Then the normalized location of the pillar centerline, relative to D, was determined for each pillar. Next, the magnitude of the front abutment was calculated using equations (2-4). Theoretical pillar loads (L_s) were then calculated as shown in figure 4, and converted to average pillar stress (σ_c) by dividing by the pillar width (w). Ratios of the measured average pillar stress changes (σ_m) to the theoretical were then calculated. These ratios are plotted on figure 5, along with a hypothetical inverse square stress decay function. It can be seen that field measurements seem to follow the trend of the stress decay function very well. It was therefore concluded that the fraction R of the total side abutment carried by the chain pillars may be estimated as:

$$R = 1 - \left[\frac{D - w_t}{D} \right]^3 \quad (6)$$

where w_t is less than D. Where w_t is greater than D, then $R=1$.

The next step towards characterizing the magnitude of abutment loads is determining the abutment angle β . Data from the three arrays in which stress measurements were continued until long after the face had passed are shown in Table 1. The final, maximum average stress change measured in the pillars was taken as the side abutment stress (σ_s). The side abutment pillar stress can be related to the side abutment load per foot of entry as:

$$\sigma_s = (L_s) (R) (C) / (A_t) \quad (144) \quad (7)$$

where: C = crosscut spacing (ft)
 A_t = the total load-bearing area
of the pillars (sq.ft)

After substituting from either equation (2) or (3) into equation (7), equation (7) can be rearranged to solve for the abutment angle β . Analysis of the data in Table 1 indicates that the calculated values of β are somewhat higher than the 16-18° used by earlier researchers. It appears that a value of $\beta=21^\circ$ may be more appropriate for longwall pillar design in the Eastern coalfields.

The final step is determination of the front abutment factors for first and second panel mining. Data on the front abutment stresses, measured at the T-junctions as the first panel adjacent to the instruments was mined, are shown in Table 2. Table 2 also contains estimates of the side abutment stress calculated assuming $\beta=21^\circ$. The values of the front abutment factor for headgate loading, F_n , were calculated after rearranging equation (4). The calculated values of F_n are remarkably consistent for field data, and it appears that a value of $F=0.5$ provides a reasonable estimate of the percent of the side abutment that must be carried by the pillars at the face ends of typical headgates.

Unfortunately, data is only available from a single array, array Y-1, for determination of a second front abutment factor (F_t) for tailgate loadings. Although one array may be insufficient for a strong conclusion, it appears from the data that longwall pillars may experience relatively greater front abutment loads in the tailgate phase than in the headgate phase. It seems that a value of $F=0.7$ may be appropriate for estimating tailgate loads until more data is available.

LONGWALL PILLAR STRENGTH

The other component of the longwall pillar design problem is estimation of the pillar strength. For multi-entry gates, it is first necessary to determine the strength of the individual pillars used. Then the individual pillar strengths are used to derive an estimate of the load-bearing capacity of the longwall pillar system.

The strength of coal pillars has been the subject of much research. To date most success in predicting pillar strength has been achieved with empirical pillar strength formulas. These formulas have been developed from the results of numerous laboratory and in situ tests, and have been validated by mining

experience. Three empirical pillar strength formulas have been identified as being most applicable to U.S. mining conditions (Bieniawski, 1984). These are the modified Obert-Duvall equation (equation 8), the modified Holland-Gaddy equation (equation 9), and the Bieniawski equation (equation 10).

$$S_p = \sigma_1 (0.78 + 0.22 w/h) \quad (8)$$

$$S_p = \sigma_1 \sqrt{w/h} \quad (9)$$

$$S_p = \sigma_1 (0.64 + 0.36 w/h) \quad (10)$$

where: S_p = pillar strength (psi)

σ_1 = in situ coal strength (psi)

w = pillar width (ft)

h = pillar height (ft)

The in situ coal strength σ_1 may be determined by scaling the compressive strength of laboratory specimens using the relationship proposed by Hustrulid (1976):

$$\sigma_1 = \sigma_c \sqrt{d}/6 \quad (11)$$

where: σ_c = strength of laboratory specimens (psi)

d = dimension of specimen (in.)

If reliable laboratory strength data is not available, then an average in situ strength of Eastern coal of $\sigma_1=860$ psi may be used as a first approximation (Bieniawski, 1982).

The empirical pillar strength formulas were developed primarily for room and pillar mining at shallow depth. They are therefore most applicable to pillars with moderate w/h ratios. Longwall pillars, which are typically at greater depth and subject to abutment loads, often have w/h ratios of eight or more. Although it is generally believed that the empirical formulas underestimate the strength of very wide pillars (Bieniawski, 1984; Holland, 1973), only one in situ test of a pillar with $w/h>4$ has ever been conducted in situ. In that study over 80 VWS were installed in a pillar which was later mined on retreat (Wang et al., 1976).

Stress measurements in longwall pillars offer another opportunity for determining the in situ strength of wide pillars. The three longwall field

studies described earlier provided seven examples of pillars that apparently approached or reached their ultimate load-bearing capacity during the time that measurements were being conducted. In four of these seven pillars, failure was indicated by a significant drop in the average pillar stress, yielding of the pillar edges, and a clear shift in the stress peak to the pillar core. In the other cases, some but not all of these indications of failure could be clearly discerned in the data.

The total load on these pillars at failure was the sum of the longwall abutment loads, as measured by the stressmeters, and the development loads. Two-dimensional finite-element modeling was again used to determine the development loads. The results are shown in Table 3. Estimates of the in situ strength of the coal at each of the three study sites are also listed in Table 3, and these were used to determine normalized pillar strengths.

Figure 6 compares the normalized pillar strengths from the seven longwall pillars with the predictions of the three empirical pillar strength formulas. An eighth data point is provided by the pillar studied by Wang et al. (1976). Although there is considerable scatter, all three equations seem to follow the trend of the data. The best fit is apparently obtained with the Bieniawski equation. This equation is the least conservative of the three. It may therefore be concluded that the Bieniawski equation is preferable for safe estimates of the strength of longwall pillars with width to height ratios up to 9, after which the equation is over-conservative.

Once the strength of the individual pillars has been calculated, the next step is determination of the load-bearing capacity of the longwall pillar system which may be taken as the simple sum of the individual pillar resistances:

$$B = [\Sigma (S_p) (A_p)] (144 / C) \quad (12)$$

where: A_p = the load-bearing area of the pillar (ft^2)

Because longwall loadings are not in general evenly distributed between the pillars, equation (12) rests on two hypotheses. The first is that a pillar's load bearing capacity remains essentially constant after it has been loaded to its limit. For wide pillars which do not fail violently, the coal near the pillar edge is expected to provide enough constraint to maintain most of the load bearing capacity of even a yielded inner core (Wilson, 1983). Field measurements

also seem to support this hypothesis. Figure 7 shows the stress histories of the four pillars under deep cover at Mine Z. Three of these pillars apparently yielded, but none apparently suffered a loss in strength of more than 20% during the time the measurements continued.

The second hypothesis is that overloading an individual pillar will not lead to instability in the entries adjacent to that pillar. Although in some instances entry stability problems have been attributed to the improper location of moderately-sized pillars (Peng, 1976), there are many examples of successful longwall mining with combinations of large and small pillars (Choi and McCain, 1980, Artler, 1985; Mucho, 1986; Carr and Martin, 1985). In at least one instance, a headgate pillar was observed to yield during longwall mining without causing any adverse entry conditions (Mucho, 1986). In another example, a yield zone near a pillar edge was observed to expand significantly without noticeably affecting the entry (Mark and Bieniawski, 1986). While this is clearly an area which merits further study, at present it appears that combinations of pillar sizes may be used so long as the entire pillar system is strong enough to resist the applied load, and so long as large, stiff pillars are placed near enough the smaller pillars to prevent excessive deformations of the roof.

GUIDELINES FOR LONGWALL PILLAR DESIGN

The empirical approaches to predicting pillar loading and pillar strength developed above have been combined into a design method called Analysis of Longwall Pillar Stability (ALPS). To use ALPS to analyse a given longwall pillar design, it is first necessary to collect the geometric and coal strength data used in equations 1-12. Then the following parameters are calculated:

- L_d (equation 1)
- L_d or L_{ss} (equation 2 or 3)
- R^S (equation 6)
- B (equation 12)

The next step is to determine the maximum loading (L) to which the pillar system will be subjected. Three loading conditions may be defined. The loading experienced by pillars at the T-junctions in the headgate, or in the tailgate during first panel mining, is called "headgate loading". Headgate loading (L_h) consists of the development loads plus the first front abutment:

$$L_h = [L_d + (L_s) (F_h) (R)] \quad (13)$$

Pillars which are expected to protect bleeder entries will be subjected to the development load and the first full side abutment, or "bleeder loading" (L_b):

$$L_b = [L_d + (L_s) (R)] \quad (14)$$

Barrier pillar loads may also be determined from equation (14), except that $R=1$. The most severe longwall loading is "tailgate loading" (L_T), experienced during the mining of the second and subsequent panels. Tailgate loading consists of the development load, the first side abutment, and the second front abutment:

$$L_T = [L_d + L_s (1 + F_t)] \quad (15)$$

Once the load has been determined, a "safety factor" (SF) may be calculated as:

$$SF = B / L \quad (16)$$

The final step in the analysis is comparison of the SF calculated in equation (16) to a design criteria. Based on a survey of 174 pillar case histories from U.S. room-and-pillar mines, Bieniawski (1984) suggested that a range of $SF = 1.5-2.0$ was appropriate for his pillar strength formula. For new mines where no previous experience is available, it appears that an SF of at least 1.5 should be used for design of chain pillars with ALPS, while an $SF = 2.0-2.5$ should be adequate for sizing barrier pillars.

In many cases it may be possible to "calibrate" ALPS with actual field experience. Where several case histories are available from a given mine or mining area, an SF may be calculated for each pillar design and compared with the field observations. The two examples below indicate how this may be done.

The first example is taken from the Pittsburgh seam, which has been extensively longwallled in Pennsylvania, Ohio, and Northern W. Va. Table 4 contains the necessary data for ALPS analysis, and the actual longwall experience, taken from six published reports. The case histories are ranked approximately in the order of SF as calculated by ALPS. An in situ coal strength of 900 psi (Hustrulid, 1976) was used in all cases. The results indicate that serious problems were experienced each time the SF was less than 1.0, whether during first or second panel mining. On the other hand, when the SF exceeded 1.3, entry stability was apparently satisfactory.

A second set of case histories is available concerning longwall experience in Alabama's Blue Creek seam. The results of the ALPS analysis are shown in Table 5. It appears that chain pillar designs with SF less than 0.8 have been unsatisfactory, while those with SF greater than about 1.0 have functioned adequately. The Blue Creek seam is considered to be somewhat weaker than the Pittsburgh (Carr and Wilson, 1982), and so an in situ coal strength of 750 psi was assumed in these analyses.

The SF calculated from back-analysis of case histories can be used as design criteria for those specific areas. One major advantage of the ALPS method is that it may be applied to two, three, and four entry systems employing any combination of pillar widths. In fact, ALPS predicts that the use of a single large pillar flanked on one or both sides by small pillars results in a smaller total pillar width, and thus a more efficient design. The greater load-bearing capacity when a single large pillar is used results because pillar strength increases rapidly as the width-to-height ratio increases. The case histories in Tables 4 and 5 provide several examples of large pillar-small pillar combinations which had higher SF than equal-sized pillar designs which used a greater total pillar width.

CONCLUSIONS

The ALPS method is a valuable tool for the design of longwall pillars. The method is an empirical one, based entirely on field measurements performed in Eastern U.S. longwall mines. With ALPS, mining engineers can predict both the strength of a particular longwall pillar system, and the loading that may be applied to it. ALPS may be used to size pillars for protection of headgate, bleeder, and tailgate entries, and it may also be used in the design of barrier pillars. The method can analyze designs using either equal sized pillars or combinations of large and small pillars. Finally, ALPS is easy to use, and it may be adjusted on the basis of local experience.

The essential purpose of ALPS is to predict whether the pillars are adequately sized to carry the vertical overburden loads. However, adequately sized pillars will not necessarily result in adequately stable entries. Other factors, such as horizontal stresses, floor failure, or poor roof rock quality, may disrupt entry stability even when the abutment loads are controlled. The importance of such

factors, and the means to control them, are the subject of on-going research.

ACKNOWLEDGMENT

The work reported here is part of an on-going research project in the Standard Oil Center for Excellence in Mining Technology at the Pennsylvania State University. The Center is supported by a grant from Standard Oil. The funding and assistance from Standard Oil and The Pennsylvania State University are greatly appreciated.

Thanks are also due to U.S. Steel Research and to the Bureau of Mines for making additional field data available. In particular, the assistance of Mr. Al Terchik and Mr. Rich Allwes is greatly appreciated.

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TABLE 1. FIELD MEASUREMENTS OF THE SIDE ABUTMENT STRESS

MINE/ STRESSMETER ARRAY	DEPTH OF COVER(H)	PILLAR WIDTHS (w) (ft)	CROSSCUT SPACING (c) (ft)	PANEL WIDTH (P) (ft)	ABUTMENT FRACTION (R)	MEASURED AVERAGE STRESS Increase(σ_s) (psi)	CALCULATED ABUTMENT ANGLE (β) (DEG)
X/1	760	42, 42	90	1000	0.85	1380	18.5
Y/2	520	34, 94	110	470	0.99	637	21.8
Z/3	630	72, 72	110	500	0.99	664	20.3

TABLE 2. FIELD MEASUREMENTS OF THE FRONT ABUTMENT STRESS

MINE/ STRESSMETER ARRAY	DEPTH OF COVER(H) (ft)	PILLAR WIDTHS (w) (ft)	CROSSCUT SPACING (c) (ft)	PANEL WIDTH (P) (ft)	ABUTMENT FRACTION (R)	MEASURED AVERAGE FRONT ABUTMENT STRESS INCREASE(σ_f) (psi)	CALCULATED SIDE ABUTMENT (FOR $\beta=21^\circ$) (psi)	CALCULATED FRONT ABUTMENT FACTOR (F_h)
X/1	760	42,42	90	1000	0.85	758	1858	0.48
X/2	640	22,22	130	1000	0.71	650	2275	0.40
Y/1	450	34,94	110	470	0.99	241	400	0.61
Y/2	520	34,94	110	470	0.99	234	619	0.38
Y/3	610	34,94	110	470	0.97	371	735	0.52
Y-4	760	34,94	110	470	0.95	486	1099	0.47
Z/1	1560	72,72	110	500	0.87	1719	2886	0.68
Z/2	1410	52,92	110	500	0.89	1290	2535	0.57
Z/3	630	72,72	110	500	0.99	327	695	0.48
Z/4	480	52,92	110	500	0.99	184	414	0.45

TABLE 3. FIELD MEASUREMENTS OF COAL PILLAR STRENGTH

MINE/ STRESSMETER ARRAY	IN SITU COAL STRENGTH (S_1) (psi)	PILLAR WIDTH (w) (ft)	PILLAR HEIGHT (h) (ft)	ESTIMATED DEVELOPMENT STRESS (psi)	MAXIMUM MEASURED STRESS CHANGE (psi)	PILLAR STRENGTH (S_p) (psi)
X/1	900	42	5.3	1354	802	2156
X/2	900	22	5.3	1527	1485	3012
Y/1	900	34	6.5	926	1258	2184
Z/1	1050	72	10.5	2557	1890	4447
Z/2	1050	92	11	2218	1774	3992
Z/2	1050	52	11	2497	1081	3578

TABLE 4. ANALYSIS OF LONGWALL PILLAR STABILITY: PITTSBURGH SEAM

REFERENCE	DEPTH OF COVER (ft)	PILLAR WIDTHS (ft)	STABILITY FACTORS		REPORTED MINING CONDITIONS
			HEADGATE LOADING	TAILGATE LOADING	
Dangerfield (1981)	800	32,32,32	0.91	- - -	"Falls spread completely across entry" during first panel mining
Mucho (1986)	650	42,42	1.43	0.75	"Tailgate failure in front of face continued throughout the panel"
Katllic (1970)	900	68,68	1.43	0.83	Major roof falls and floor heave during second panel mining
Artler (1985)	800	52,62	1.59	0.87	Tailgate problems: "Roof lowering, excessive sloughage, floor heave"
Artler (1985)	800	34,94	2.36	1.32	Design "More than adequate"
Choi and McCain (1980)	750	32,82	2.38	1.33	"Good roof conditions in tailgate" during 2nd and 3rd panel mining
Mucho (1986)	650	20,93	2.39	1.33	Tailgate "Showed no signs of deterioration"
Dangerfield (1981)	800	85,85,85	2.23	1.47	"No problems experienced"
Lu (1985)	900	93,93	2.47	1.48	No problems reported

TABLE 5. ANALYSIS OF LONGWALL PILLAR STABILITY: BLUE CREEK SEAM

REFERENCE	DEPTH OF COVER	PILLAR WIDTHS	STABILITY FACTORS		REPORTED MINING CONDITIONS
			HEADGATE LOADING	TAILGATE LOADING	
Carr & Wilson (1982), Brass (1981)	1500	80,80	1.07	0.58	Satisfactory for 1st panel, but problems in tailgates of subsequent panels
Carr & Martin (1986), Carr & Wilson (1982)	1500	64,64,64,64	0.95	0.63	"Strata control in the tailgate was a notable problem"
Carr & Martin (1986)	2000	105,105,105	1.07	0.70	Floor heave, rib sloughage, and roof deterioration "unacceptable" in tailgate
Gauna et al., (1985)	2000	115,115,115	1.22	0.78	Tailgate damage, expensive support required
Jim Walter Resources(1986)	2000	20,200,20	1.65	0.97	Design now in use
Jim Walter Resources(1986)	1500	20,180,20	1.93	1.15	Design now in use
Carr & Wilson (1982),Brass (1981)	1500	130,60,130	1.97	1.32	No load transfer to tailgate, but large amounts of reserves sterilized

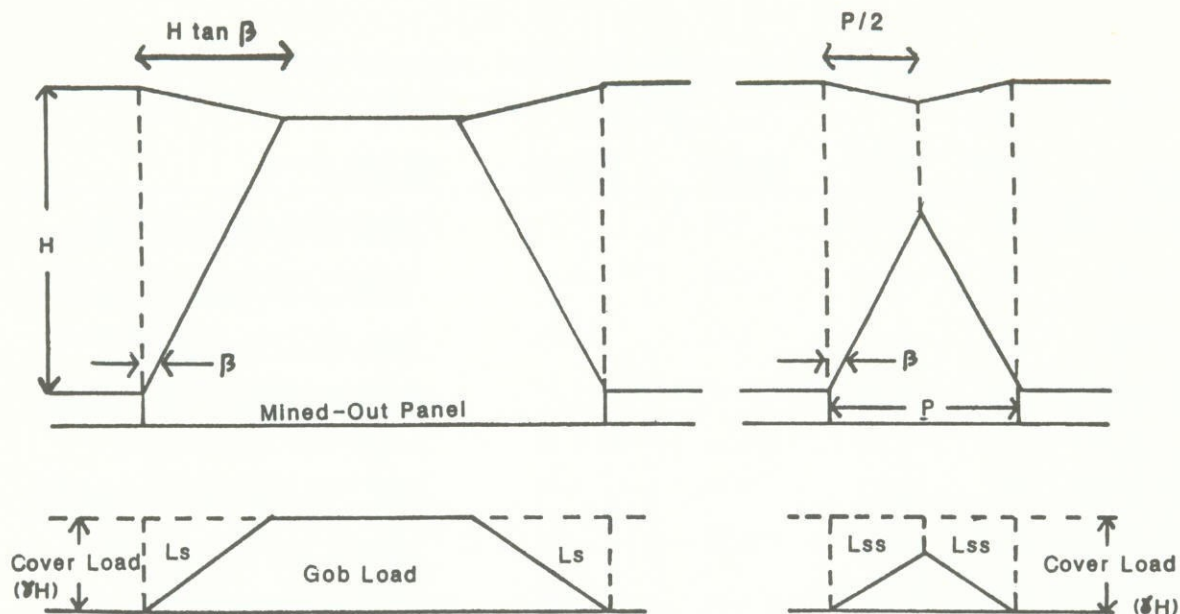


Figure 1. The side abutment load. The upper sketches illustrate the King and Whittaker (1971) approach, the bottom sketches show the Wilson (1972) approach.

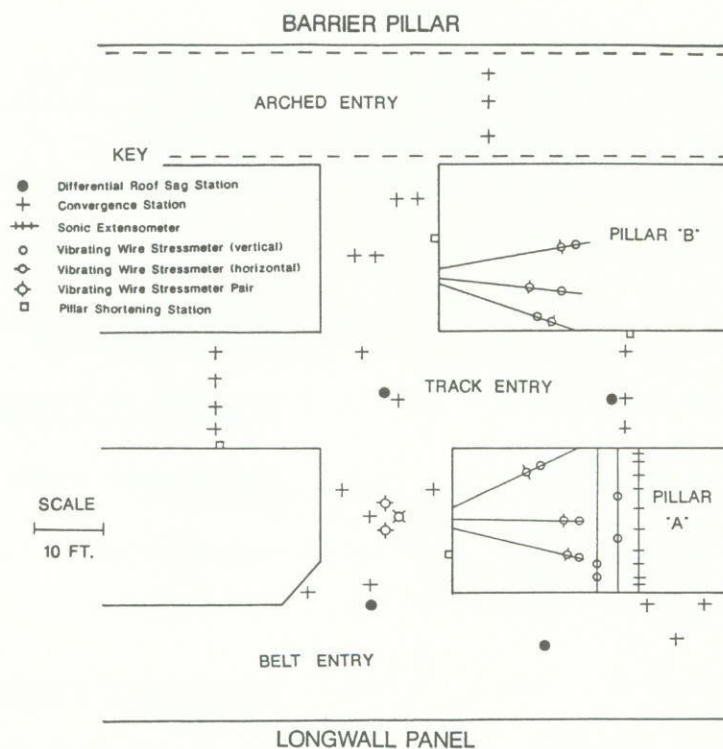


Figure 2. Instrumentation layout for the second Penn State study at Mine X.

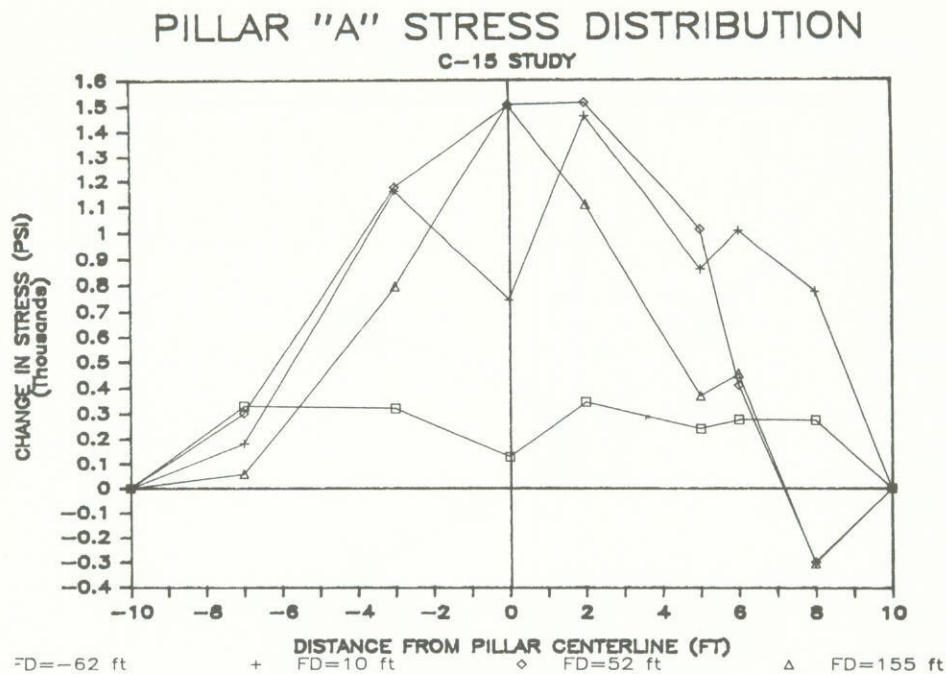
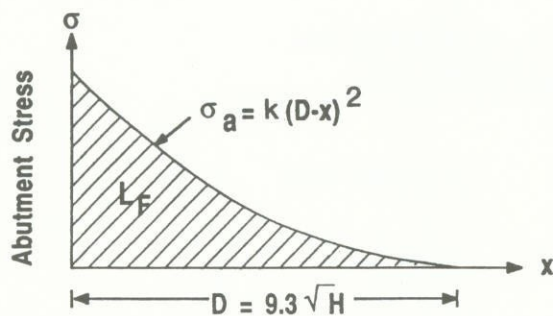


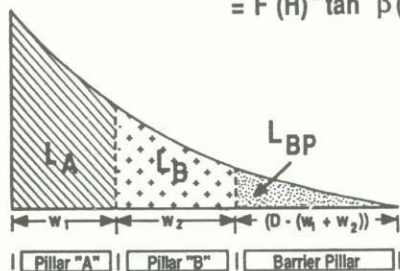
Figure 3. Vertical stress distribution profile measured in the pillar nearest the face at four times during the second study at Mine X.



Distance from Longwall

$$L_F = k \int_0^D (D-x)^2 dx = \left(\frac{k}{3} \right) D^3$$

$$= F(H)^2 \tan \beta \left(\frac{\gamma}{2} \right)$$



$$\frac{L_A}{L_F} = \frac{L_F - k \int_{w_1}^D (D-x)^2 dx}{L_F} = 1 - \left(\frac{D-w_1}{D} \right)^3$$

Figure 4. Calculation of the fraction of the total side abutment carried by the pillars. The upper sketch shows the inverse square stress distribution, and the lower sketch shows how the load is distributed between the pillars. Note that k is a constant equal to $L_F/3D^3$. See text for definitions.

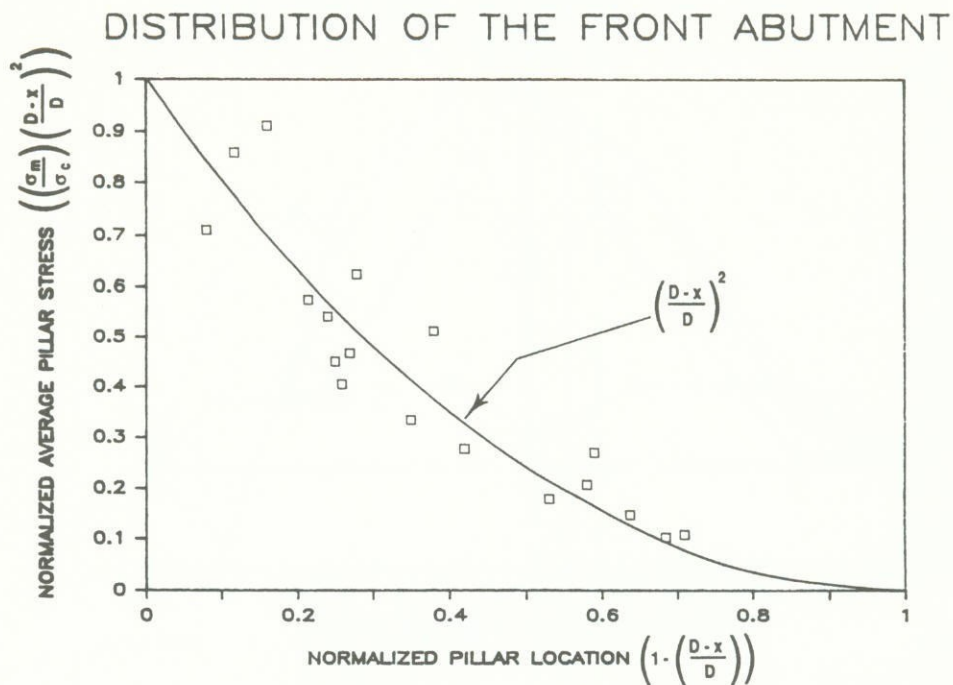


Figure 5. Comparison of the measured average front abutment pillar stresses with the inverse square stress distribution.

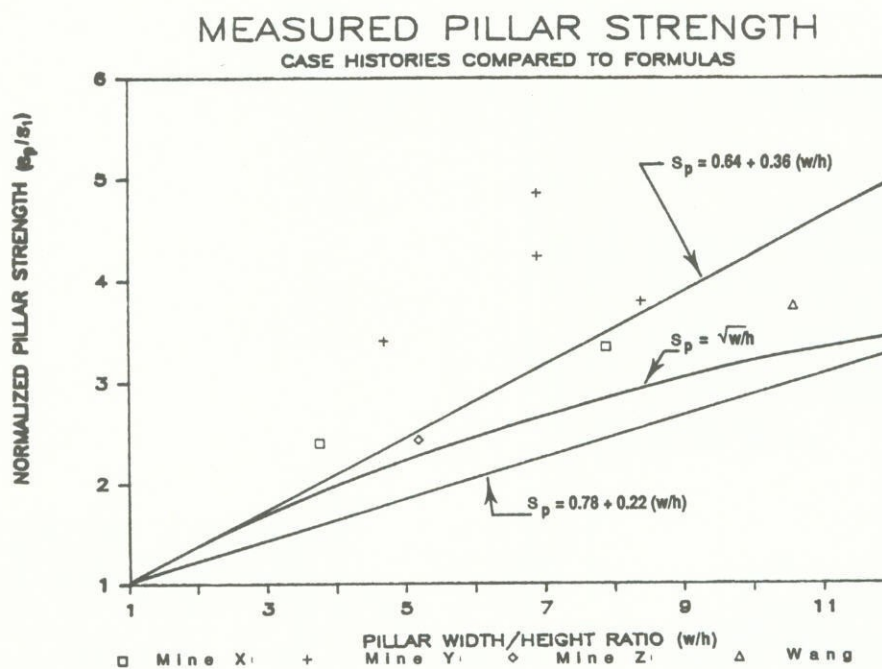


Figure 6. Comparison between the observed strength of eight large coal pillars with the predictions by three empirical formulas.

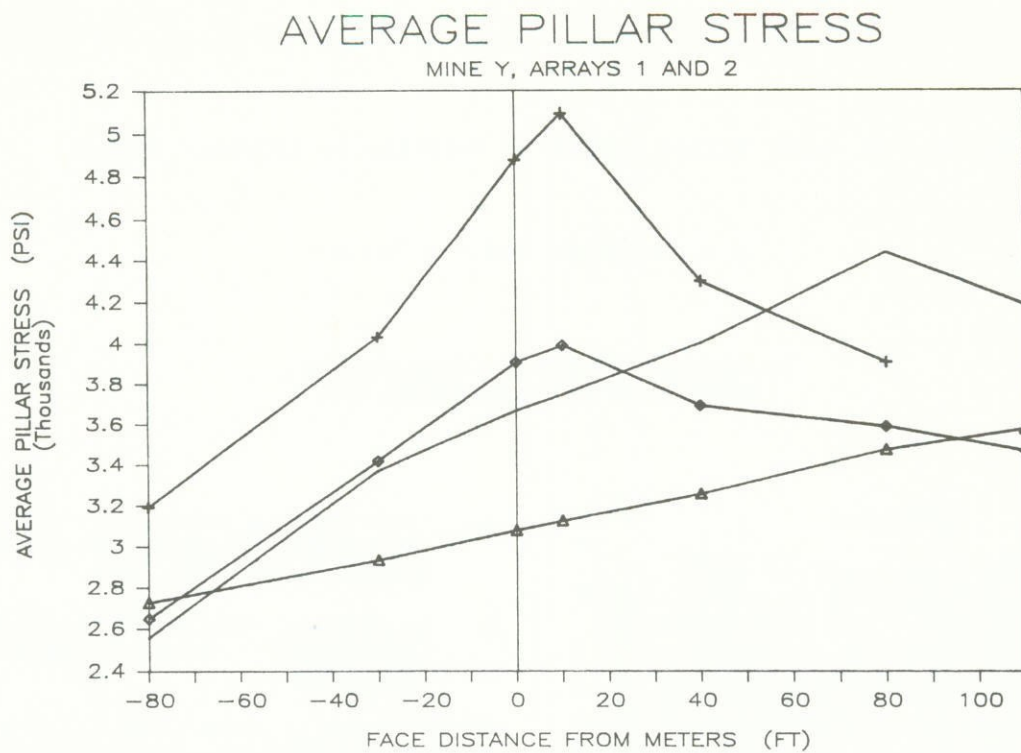


Figure 7. Average stress (including development stress) measured in four longwall pillars at Mine Y.

Aspects of Chain Pillar Design in Relation to Longwall Mining

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ABSTRACT

The paper examines the role of chain pillars in longwall mining layouts as used in several countries. The application of chain pillars in relation to their expected design role is discussed. An important feature is the specification of the pillar design to meet strata loading conditions as imposed by the mining and geological setting. The paper relates longwall width and chain pillar configurations to pillar size to satisfy anticipated loading and strength expectations. Laboratory work is referred to which examines criteria for pillar stability.

Comments are made regarding the application of the findings presented in the paper, to different situations involving the use of chain pillars in longwall and shortwall mining systems.

INTRODUCTION

Pillars are commonly left between longwall extractions for a number of reasons, the more important ones being listed as follows:

1. to provide protection of the gate roadways against the full potential effects of caving behind the longwall,
2. to protect roadways against strata deformation produced by high abutment stresses acting on the pillars,
3. to provide scope for isolation of successive longwall extractions, which may be desirable in connection with gas, water and spontaneous combustion hazards,
4. to protect current longwall extractions in respect of working too close to geological disturbances such as faulting and igneous intrusions,

5. to provide a measure of protection to the surface against the effects of mining subsidence, and
6. to improve roof control around the face-end of current longwall extractions.

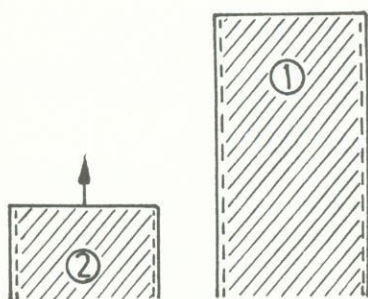
In European layouts, the pillars left between successive longwall workings are in the form of a continuous rib. The width of a rib pillar is generally around 0.1 (depth) since this provides an optimum for the factors listed above, especially against excessive deformation induced by heavy strata loading in deep workings where longwall advance mining is employed. Narrow rib pillars can result in destructive effects to mine roadways in deep layouts, so that choice of pillar size is important to overall success, safety and economics.

Longwall operations in North America, South Africa and Australasia usually employ chain pillars between successive extractions and this form of mining layout has several advantages in respect of ease of development where relatively shallow conditions exist (usually not deeper than 300 m).

Choice of pillar size needs consideration of the longwall extraction width, since the longwall operation transfers additional strata loading to the intervening pillars.

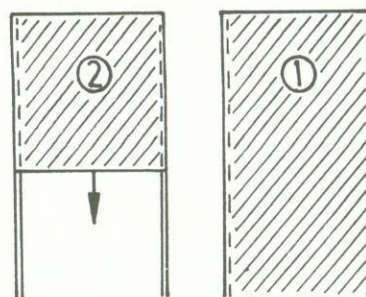
MINING LAYOUTS IN RELATION TO PILLARS BETWEEN LONGWALLS

Figure 1 shows common forms of longwall mining. Figure 1(a) illustrates longwall advance operation which is the most popular form of underground mining in the UK. Rib pillars serve an important function in protecting the gate roadways against the effects of strata loading in such layouts. The longwall retreat form of layout is also widely used in European coalfields, but where weak strata exists in deep workings



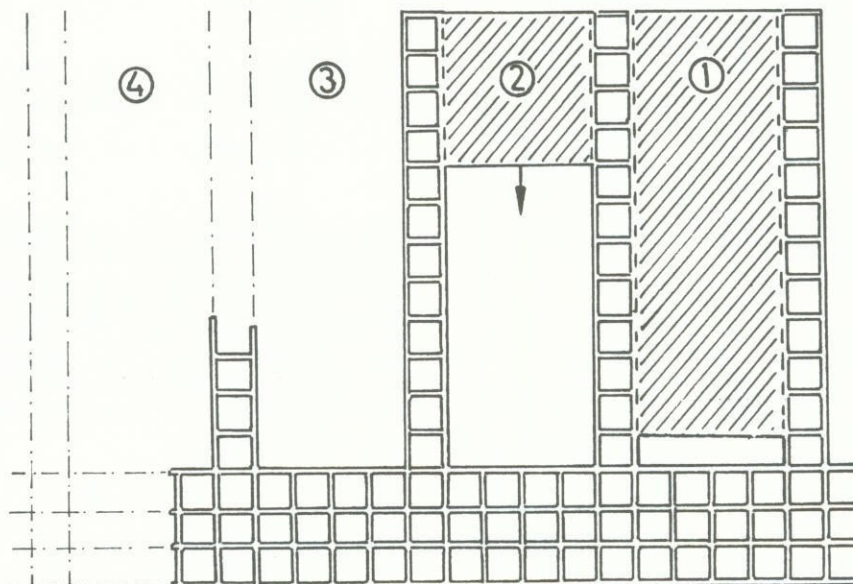
(a) Longwall advance

Rib pillar required for shielding the gate of panel 2, and possibly the LH gate of panel 1 during salvage.



(b) Longwall retreat

Rib pillar required to ensure RH heading (gate) of panel 2 is sufficiently displaced from flank strata pressure abutment of panel 1.



(c) Longwall retreat with chain pillars

Chain pillars allow convenient development using continuous miners; the pillars are required to provide protection of roadways against strata pressure abutment loading effects. Although a single row of chain pillars is shown, there can be several rows between longwalls.

Figure 1 Main forms of pillar configuration between longwalls (after Whittaker [3])

the gate roadways may experience stability problems and result in unacceptable levels of closure to the extent of impeding mining operations. Consequently, longwall retreat layouts of the form shown in Figure 1(b) tend to be selective in their application since it is usual to use rectangular openings for the gate roadways. In the case of Figure 1(a), European practice is to employ arch-shaped roadways for stability purposes.

Figure 1(c) shows a typical form of longwall retreat mining layout involving the use of chain pillars. Although only a single chain of pillars is shown, double or more rows of pillars can be used depending upon mining requirements. The use of 4-way junctions in chain pillar configurations needs careful consideration since there is a major increase in stress disturbance around such openings when compared to 3-way (T) junctions. Junctions in such layouts are the main localities for potential roof control problems and this is accentuated when the junction is within 30 m of an active longwall owing to the rapidly changing stress conditions.

With reference to Figure 1(c), there is an increasing transfer of strata loading to the chain pillars, and consequently their ability to accept loading needs to have an ample factor of safety against potential failure to take the effects of time and load fluctuations into account.

NATURE OF CHAIN PILLAR LOADING

The mechanics of load transfer from areas mined by longwalling to an intervening continuous rib pillar has been examined by King and Whittaker [1]. In the case of the chain pillar, load transfer needs to be assessed for individual pillars or lines of pillars depending upon their layout. The load that is transferred from the adjacent goaf areas is deemed to be dependent upon the mining depth to satisfy subsidence compatibility. The general representation of load transfer for a single line of chain pillars is shown in Figures 2(a) and 2(b).

Two subsidence situations need to be identified prior to the calculation of loads, these being:

- (i) Subcritical,
where $W/h < 2 \tan \phi$, and
- (ii) Critical and Supercritical,
where $W/h \geq 2 \tan \phi$.

These are represented in Figures 2(a) and 2(b) respectively.

The symbols used in the following analysis are defined as follows:

W	width of longwall face (m)
h	depth below surface (m)
γ	average density of overburden (kg/m^3)
ϕ	average angle of shear ($^\circ$)
p	pillar width (m)
m	pillar height (m)
B	bord (heading or room) width (m)
$\bar{\sigma}$	average pillar stress (N/m^2)
$\bar{p}$	better estimate of p (m)
L	length of rectangular pillar (m)
x	aspect ratio of rectangular pillar (-)
F	factor of safety (-)

Single Line of Square Chain Pillars

Load on pillar (N) =

$$9.81 \gamma [(p+W)h - W^2 \frac{\cot \phi}{4}]. (p+B) \quad \dots (1)$$

where $W/h < 2 \tan \phi$

In equation (1) the total volume of rock resting on the pillar is:

$$V = [(p+W).h - W^2 \frac{\cot \phi}{4}]. (p+B)$$

The goaf triangle is assumed to be completely relieved and its loading transferred fully to the extracted region. The angle of shear ϕ has been suggested by King and Whittaker [1] to be approximately 31° for British Coal Measures rocks since this allows the relationships to be compatible with established subsidence principles.

For $W/h \geq 2 \tan \phi$,

Load on pillar (N) =

$$9.81 \gamma [p.h + h^2 \tan \phi]. (p+B) \quad \dots (2)$$

Following on from this the average stresses applied to the pillars ($\bar{\sigma}$) are given by:

$$\bar{\sigma} (\text{N/m}^2) = 9.81 \gamma [(p+W)h - W^2 \frac{\cot \phi}{4}]. \frac{(p+B)}{p^2} \quad \dots (3)$$

for $W/h < 2 \tan \phi$

$$\bar{\sigma} = 9.81 \gamma [ph + h^2 \tan \phi] \frac{(p+B)}{p^2} \quad \dots (4)$$

for $W/h \geq 2 \tan \phi$

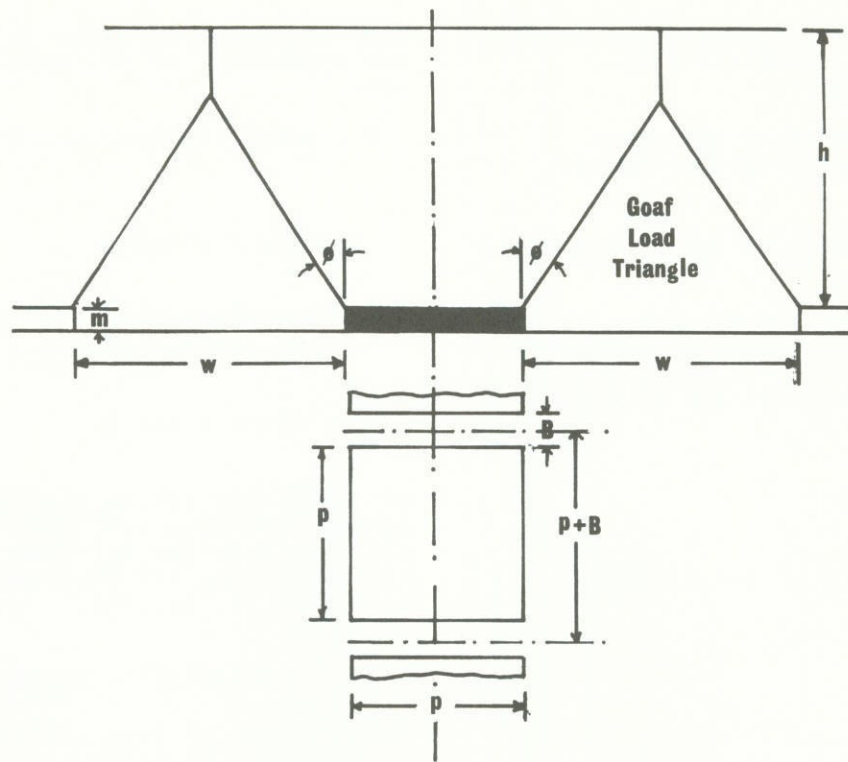


Figure 2(a) General representation of theoretical model for the subcritical subsidence situation

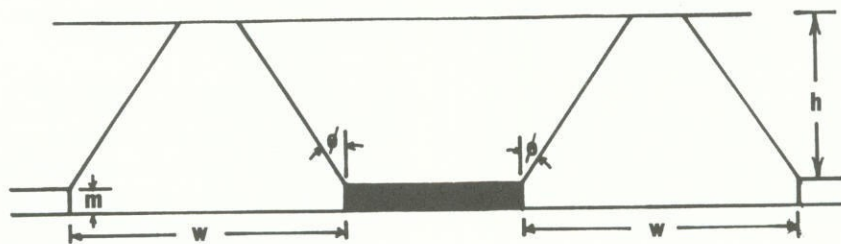


Figure 2(b) Critical/supercritical subsidence situation

Double Line of Square Chain Pillars

The mining situation for this pillar layout is shown in Figure 3, with the only physical changes being:

- (a) the effective width of the chain pillar support area becomes $(2p+B)$, and
- (b) the load is carried by two pillars of total area, $2p^2$.

The average pillar stresses are given as:

$$\bar{\sigma} = 9.81\gamma[(2p+B+W).h - W^2 \frac{\cot \phi}{4}] \frac{(p+B)}{2p^2} \quad \dots (5)$$

for $W/h < 2 \tan \phi$

$$\bar{\sigma} = 9.81\gamma[(2p+B).h + h^2 \tan \phi] \frac{(p+B)}{2p^2} \quad \dots (6)$$

for $W/h \geq 2 \tan \phi$

Single Line of Rectangular Chain Pillars

Rectangular chain pillars are employed extensively and frequently in preference to square pillars as less junctions are formed during development. This will inevitably result in increased stability and speed of development of panels for subsequent extraction. It also decreases the chances of roof falls occurring at junctions.

The general dimensions for a rectangular pillar are shown in Figure 4(a). A further parameter is thus defined, this being the pillars "aspect" ratio $x = L/P$ or $L = xp$. The average pillar stress is given by:

$$\bar{\sigma} = 9.81\gamma[(p+W).h - W^2 \frac{\cot \phi}{4}] \frac{(xp+B)}{xp^2} \quad \dots (7)$$

for $W/h < 2 \tan \phi$

$$\bar{\sigma} = 9.81\gamma[ph + h^2 \tan \phi] \frac{(xp+B)}{xp^2} \quad \dots (8)$$

for $W/h \geq 2 \tan \phi$

Double Line of Rectangular Chain Pillars

This situation is shown in Figure 4(b) and the equations for the average pillar stress are:

$$\bar{\sigma} = 9.81\gamma[(2p+B+W).h - W^2 \frac{\cot \phi}{4}] \frac{(xp+B)}{x2p^2} \quad \dots (9)$$

for $W/h < 2 \tan \phi$

$$\bar{\sigma} = 9.81\gamma[(2p+B).h + h^2 \tan \phi] \frac{(xp+B)}{x2p^2} \quad \dots (10)$$

for $W/h \geq 2 \tan \phi$

NB Equations (7) to (10) apply to the square case when $x=1$, thus the average pillar stress for any size pillar in any mining setting can be found by using one of equations (7) to (10).

STRENGTH OF SQUARE PILLARS

The strength of coal pillars has been investigated by Salamon [2]. An estimate of coal pillar strength of square geometry is given by the Salamon formula which states that:

$$\text{Pillar strength (kN/m}^2\text{)} = \frac{7180p^{0.46}}{m^{0.66}} \quad \dots (11)$$

where the pillar width (p) and extraction height(m) are in metres.

The formula was established using detailed observations of pillar behaviour within room and pillar coal mines of South Africa. However the relationship would appear to be applicable to coal seam workings in other countries including the USA, where the coal strength and geological setting has some similarity to that of South Africa. The formula was established from full-scale studies and therefore, rock mass reduction factors are inherent in it.

It is considered by the authors that equation (11) is probably the most meaningful coal strength relationship for application to chain pillars situations in view of the fairly symmetrical transmission of loading to the pillar configuration.

CALCULATION OF IN-SITU PILLAR DIMENSIONS

An assessment of the required pillar size for any underground situation can be made by equating pillar strength given by equation (11) to the average pillar stress

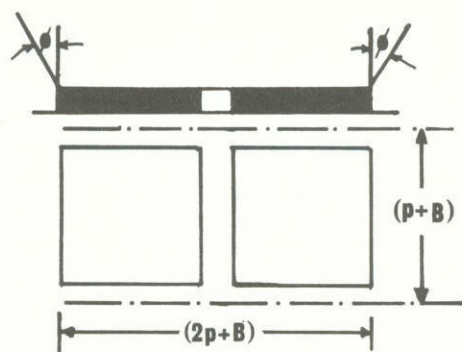


Figure 3 Double row of square chain pillars

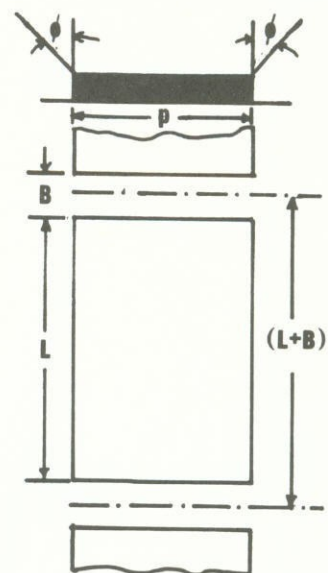


Figure 4(a) Single row of rectangular chain pillars

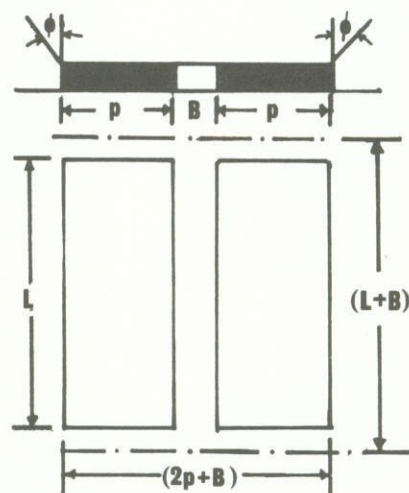


Figure 4(b) Double row of rectangular chain pillars

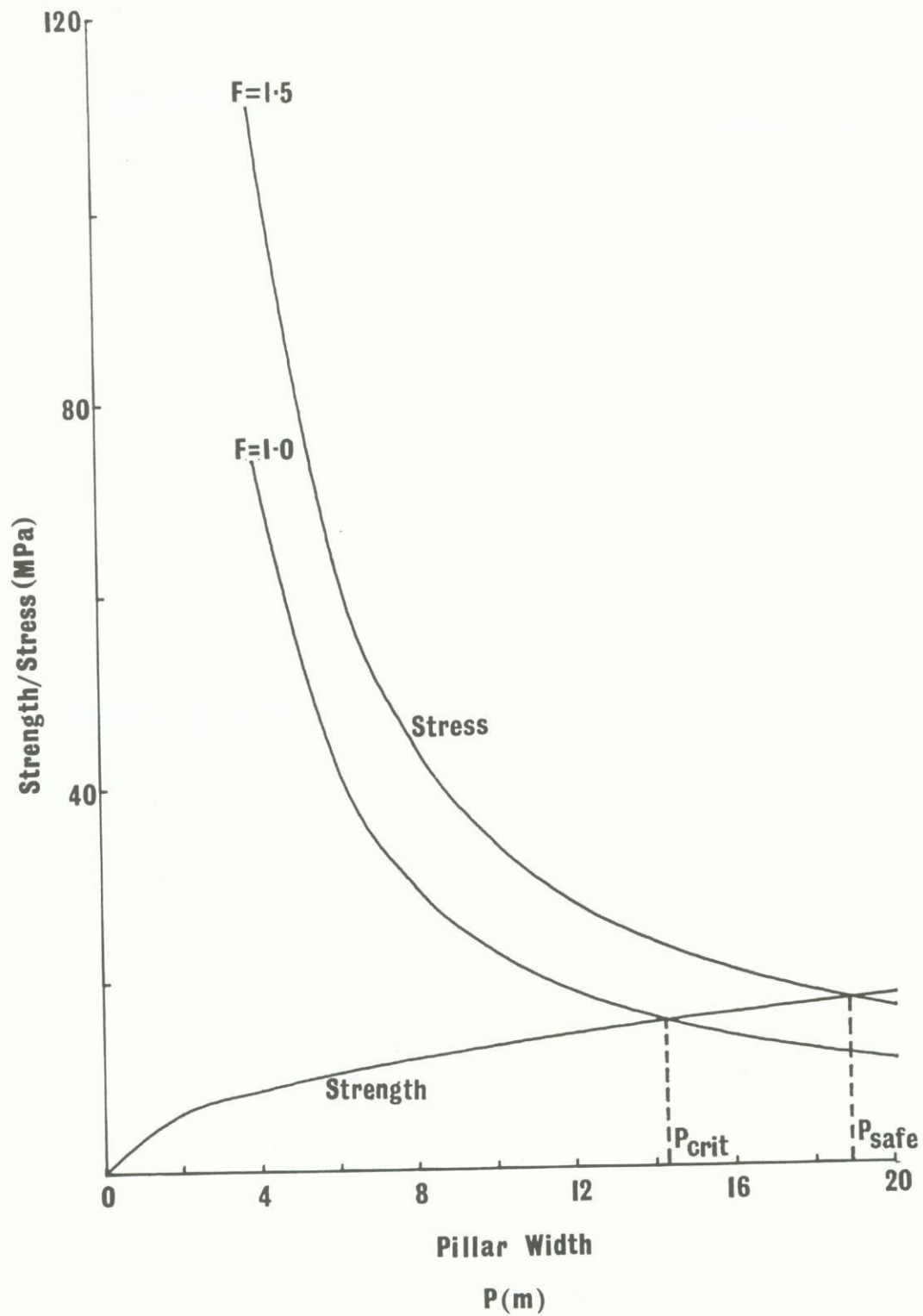


Figure 5 Relationship between stress and strength with pillar width

given by equations (7) to (10), according to whether the line is single or double, and if the subsidence situation is subcritical, critical or supercritical. The following example relates to a single line of square pillars ($x=1$), with a critical/supercritical subsidence situation ($W/h \geq 2 \tan \phi$).

A "critical" pillar condition is defined whereby the pillar strength equals the average pillar stress. Using equations (11) and (8):

$$\frac{7180p^{0.46}}{m^{0.66}} = \frac{9.81\gamma}{1000} \cdot [ph+h^2 \tan \phi] \frac{(p+B)}{p^2} \quad \dots (12)$$

Similarly, a factor of safety F can be applied to equation (12) so that:

$$\frac{7180p^{0.46}}{m^{0.66}} = F \left[\frac{9.81\gamma}{1000} \cdot [ph+h^2 \tan \phi] \frac{(p+B)}{p^2} \right] \quad \dots (13)$$

Equation (13) represents a realistic design situation with F being between 1.2 and 1.5.

Equations (12) and (13) can be solved both graphically and numerically to give the critical pillar size, p_{crit} and the safe pillar size p_{safe} respectively. The following example is used to illustrate both techniques using the following data:

$h = 100 \text{ m}$
 $m = 2 \text{ m}$
 $B = 4 \text{ m}$
 $\phi = 31^\circ$
 $\gamma = 2350 \text{ kg/m}^3$

A unique solution to equations (12) and (13) can be found by plotting pillar strength versus p and pillar stress versus p as defined by equations (11) and (8) with the appropriate factor of safety. Figure 5 illustrates this method for the example given.

From Figure 5,

$$p_{crit} = 14.3 \text{ m}$$

$$p_{safe} = 19.0 \text{ m}$$

Although this graphical method is simple it is time consuming, thus a numerical solution has been derived for equations (12) and (13).

A more useful solution to a designer would be a simple formula for p in any mining situation, derived from equations (12) and (13). However, due to the complex nature

of the equations, no simple formulae are readily available. Thus an iterative scheme is now presented to solve them for p .

Both equations (12) and (13) need to be rearranged into a form whereby p appears on both sides and, the scheme converges to the values of p found from the graphical method. The following equations have been found to satisfy these conditions.

$$\bar{p}_{crit} = \left[\frac{9.81\gamma [ph+h^2 \tan \phi] (p+B)m^{0.66}}{7180 \times 1000} \right]^{0.407} \quad \dots (14)$$

$$\bar{p}_{safe} = \left[\frac{F \cdot 9.81\gamma [ph+h^2 \tan \phi] (p+B)m^{0.66}}{7180 \times 1000} \right]^{0.407} \quad \dots (15)$$

$\bar{p}$ represents a better estimate of the inputted value of p . A final value of p_{crit} and p_{safe} can be found by repeatedly using the better estimate $\bar{p}$ as the input value p until $\bar{p} = p$. Figure 6 shows the convergence of this scheme for an initial estimate of $p_0 = 2 \text{ m}$.

From Figure 6,

$$p_{crit} = 14.3 \text{ m}$$

$$p_{safe} = 19.0 \text{ m}$$

These values agree with those found using Figure 5. Similar equations to (14) and (15) can be easily derived for all of the general pillar stress equations (7) to (10) to give iterative schemes, solvable on small micro computers.

MODEL PILLAR INVESTIGATIONS

Whittaker [3] undertook a series of model pillar tests to verify certain factors effecting pillar behaviour. Although the results of these tests have been reported previously, they are, however, of sufficient importance for further discussion in this text.

The work was involved with considering Salamon's original suggestion that the strength of rectangular coal pillars could be calculated by using his original equation with the value of p being that of the shortest side of the rectangle. It was also compared with the use of the "equivalent square" dimension. Whittaker concluded that Salamon's original recommendation of using the smaller pillar side holds good for values of L/p or x up to 5. This value of x is possibly the maximum that would be used in any realistic design, therefore it is quite appropriate

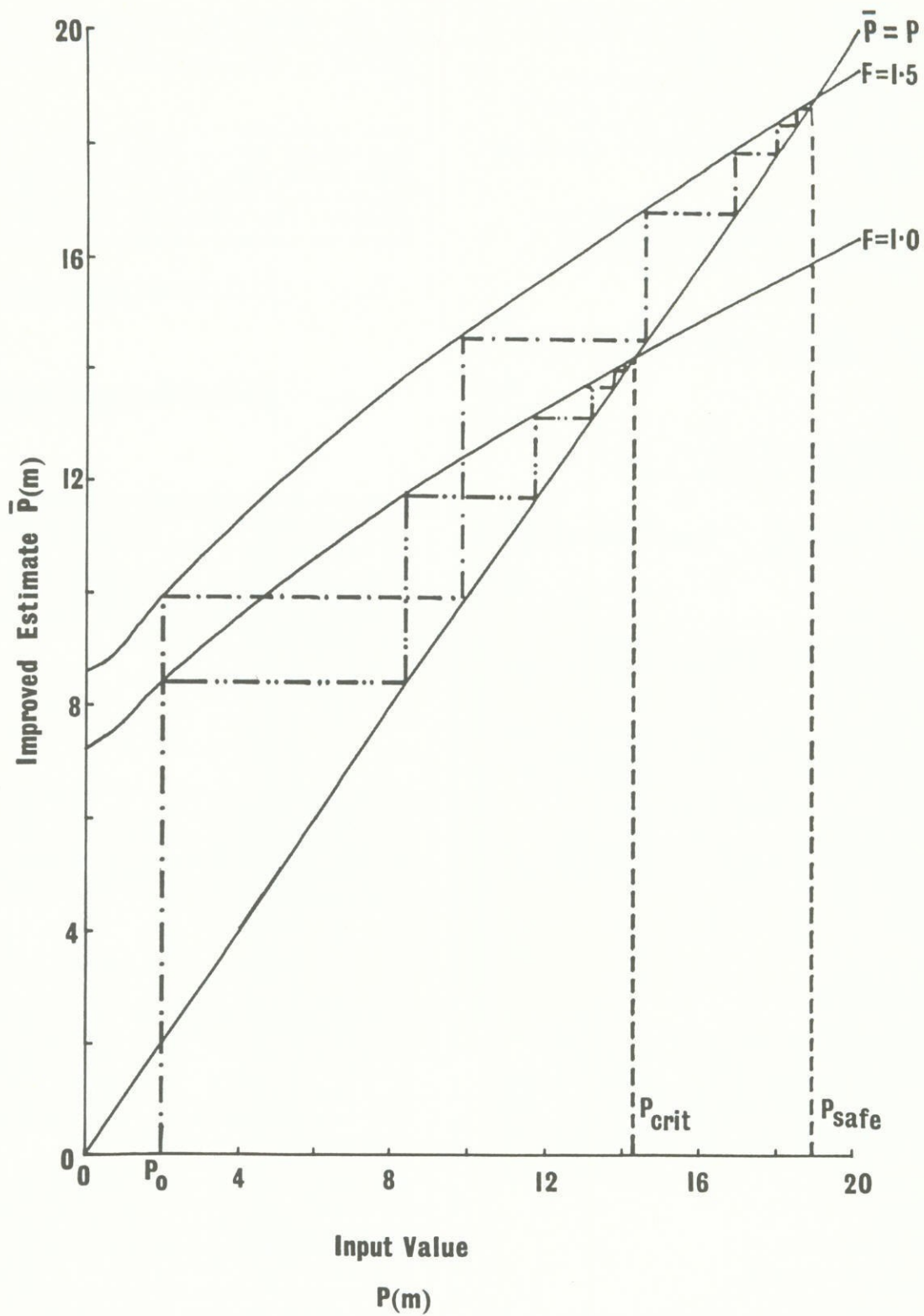


Figure 6 Convergence of the numerical scheme used to solve equations (14) and (15)

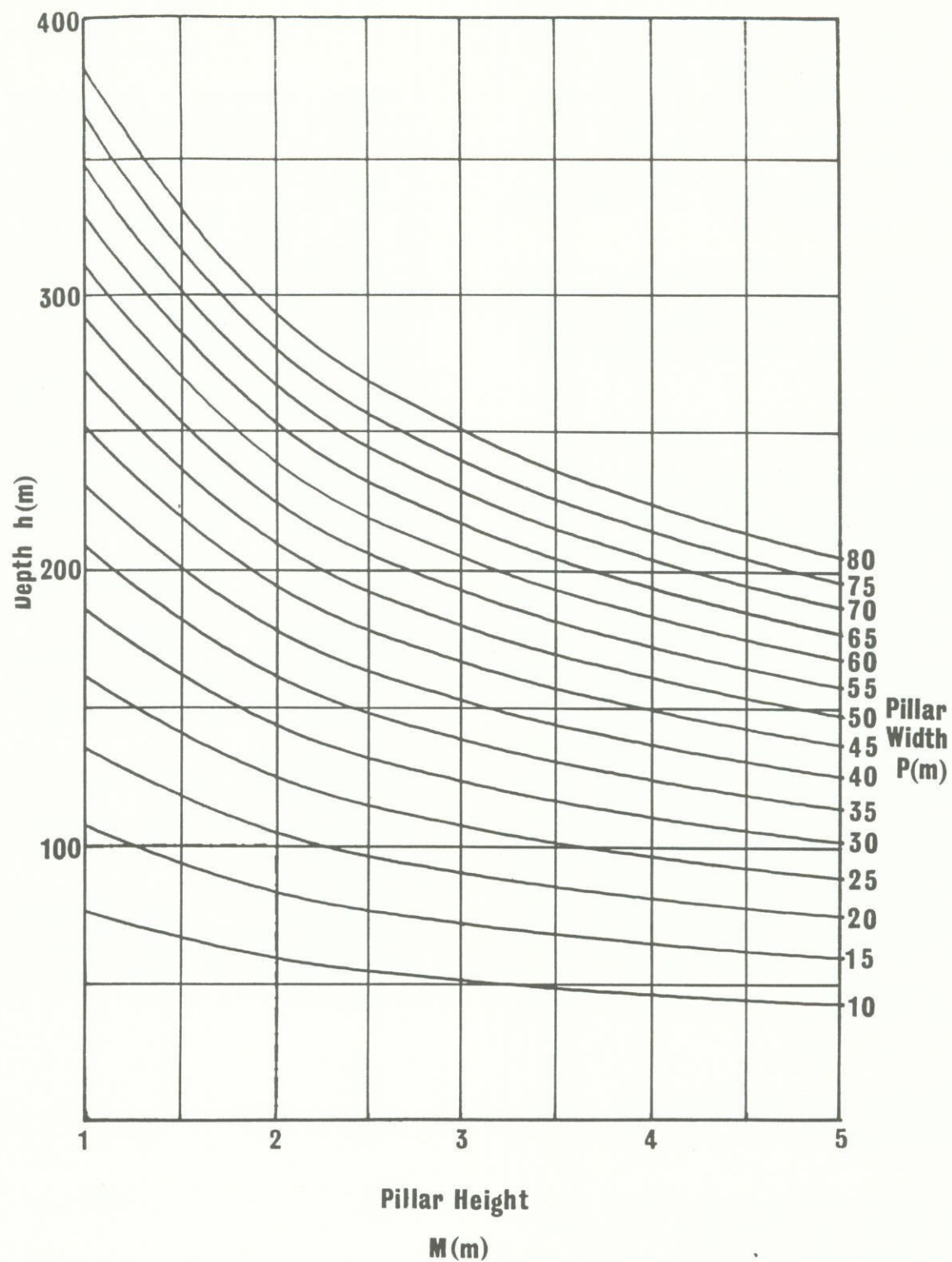


Figure 7 Design nomogram for a single line of chain pillars with $B = 4$ m, $x = 1$, $F = 1.5$ and a supercritical subsidence situation

to use equation (11) in the way outlined earlier in the text of this paper.

The test work also investigated the effect of the m/p ratio on the strength of a pillar. The results showed that for a pillar of given dimensions, its strength increases higher than given by equation (11) for $m/p \leq 0.25$. Thus in chain pillar situations where $m/p < 0.25$, there is a substantially higher factor of safety against pillar failure than that apparently predicted by equation (11).

The development of a pillar core, providing post-failure strength was seen during the test work on rectangular pillars as opposed to square pillars. This suggests another reason for using rectangular as opposed to square pillars.

DESIGN NOMOGRAMS FOR CHAIN PILLARS

The two methods of determining a pillar dimension p , given earlier in the paper, although quite correct are either time-consuming or involve the use of a computer. A more useful solution to a designer would be one or a series of nomograms, which would instantly relay the design options available.

There are several variables involved in the calculation of pillar size p which cannot all be represented on a single nomogram. Therefore for any single one, some variables need to be held constant. The nomogram presented here relates depth h to pillar height m and pillar size p for a given factor of safety F , bord width B , longwall length W , aspect ratio x and general pillar layout. These variables are deemed to be dictated by other mining factors apart from pillar considerations. Hence, once they have been decided upon, an appropriate pillar design can be found.

Figure 7 gives a mining situation using a single line of pillars

$F = 1.5$ (design value)
 $B = 4$ m
 $W =$ independent parameter as the
subsidence situation is
supercritical
 $x = 1$

Using Figure 7, the required pillar dimension for a 2 m extraction at a depth of 100 m is approximately 19 m. This agrees with the values of p_{safe} found from Figures 5 and 6.

At present Figure 7 is the only nomogram that has been produced, but it is intended to produce a series of them for different factors of safety between 1.0 and 2.0, bord

widths B , and pillar layout and should provide a useful aid to the mine designer.

CONCLUSIONS

A design criteria for pillar size can be derived by equating expressions for pillar strength to average pillar stress, and incorporating a reasonable factor of safety of between 1.2 and 1.5. The equations used within the text have been Salomons equation for coal pillar strength [2] and those derived by Whittaker [3] for pillar stress. Two methods of solving the resultant equation have been presented, namely graphical and numerical schemes. As a logical continuation of this a series of design nomograms for chain pillars have been proposed, an example of which has been given. It is felt that the nomogram idea represents the most practical solution to the design equations defined in the text as it is easy to use and presents an overall picture of the available design options. Neither the graphical or numerical technique explained have these advantages.

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THE INITIAL COLLAPSE OF THE OVERBURDEN OVER LONGWALL PANELS

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ABSTRACT

Documented case histories are studied where data were found on the initial collapse of the overburden above the longwall panels. Under this scenario, the most controlled conditions exist for assessing the span capacity of the overburden (i.e., before failure the roof spans to firm coal abutments on two to all four sides).

In an attempt to correlate theory to field observations, data are used on the overburden lithology, and the depth, width, and face advance of the panel, as well as, the subsidence conditions noted on the surface. The subsidence measured on the surface is used to determine when complete failure in the overburden rock has occurred.

THIS PAPER WILL NOT BE AVAILABLE

MONITORING AND PREDICTION OF GROUND MOVEMENTS ABOVE UNDERGROUND MINES IN THE EASTERN UNITED STATES

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INTRODUCTION

Surface deformations above underground mines and their prediction and control have been the topics of an extensive research effort by Virginia Polytechnic Institute and State University over the past few years. During the initial stages of this research, a number of case studies were collected from the literature, the mining industry and various government and state agencies and, based on the analysis of this information, appropriate prediction methods were proposed (Goodman, 1980; Webb, 1982; Hasenfus, 1984; Karmis et al., 1984).

Although the prediction methods were based on sound concepts, it became evident that their accuracy and implementation were a reflection of the size and quality of the collected case studies. In this respect, the data bank was rather insufficient and posed certain questions regarding the development of subsidence information due to inappropriate length and spacing of monuments used in subsidence surveys, density and accuracy of such surveys, lack of monitoring horizontal movements, etc.

In an effort to alleviate these problems, a comprehensive monitoring program was initiated in the coal mines of southwest Virginia, which included:

- Two longwall and five room and pillar mines.
- In total, seven longwall and sixteen room and pillar panels.
- Monitoring lines in excess of 35,000 feet.
- Vertical as well as horizontal measurements.

A high accuracy digital computer tacheometer was used in all surveys and the type and spacing of all monuments were in accordance with well accepted guidelines.

The data from this monitoring program completed the initially collected data bank and, thus, enabled a much needed statistical analysis between

mining factors and representative ground movement parameters.

Based on this analysis, the previously developed prediction methods were modified and tested for their applicability in the Appalachian coalfield. Finally, in order to facilitate the implementation of the prediction methods, as well as the data processing of the monitoring program, a number of computer software were developed and will be discussed in the paper.

PREDICTION METHODS

Three prediction methods were pursued under this research based on the theories of the profile functions, influence functions and the zone area method.

Profile Functions

Profile function methods describe the distribution of subsidence values along a half-profile, orthogonal to the longitudinal axis of the underground excavation, by means of a mathematical function or tables. These methods have been utilized in many coal mining regions around the world (NCB, 1975; Brauner, 1973; Munson and Eichfeld, 1980; Webb, 1982) by establishing the best fit function based on local field data.

The function recommended as most applicable for the Appalachian mining and geologic conditions (Karmis et al., 1984; Schilizzi, 1987) was given as follows:

$$s(x) = 0.5 S_{\max} (1 - \tanh(cx/B)) \quad (1)$$

where,

- $s(x)$ = subsidence at x ;
- x = distance from the inflection point;
- $S_{\max}$ = maximum subsidence of the profile;
- B = distance from the inflection point to point of $S_{\max}$; and
- c = constant.

Influence Functions

The influence function methods were developed primarily in the central European coalfields. They have been discussed in detail by Litwiniszyn (1957), Knothe (1957), Brauner (1973) and Kratzsch (1983).

In this approach, a so-called function of influence is assigned for every point of the rock strata, such that the integral of this function over a given area represents the subsidence (vertical displacement) caused at that point by the excavation of the seam below.

After reviewing the functions available in the literature, it was decided to utilize the bell-shaped Gaussian function, often also referred to as "Knothe's method of prediction" (Knothe, 1957). The mathematical development of this function, for vertical and horizontal movements, has been discussed in detail elsewhere (Schilizzi, 1987; VPI&SU, 1987). Briefly, subsidence at any point $S(s)$, can be expressed in this case by the following integral:

$$S(s) = \int_{-\infty}^{+\infty} \frac{S_0(x)}{r} \exp \left[-\pi \frac{(x-s)^2}{r^2} \right] dx \quad (2)$$

where,

- r = radius of principal influence - $h/\tan \beta$;
- h = mining depth;
- β = angle of principal influence;
- s = location of the point where subsidence is considered
- x = location of the infinitesimal element of excavation; and
- $S_0(x)$ = convergence of the roof of the infinitesimal element of excavation.

The convergence of the roof can be expressed by:

$$S_0(x) = m(x) * a(x) \quad (3)$$

where,

- $m(x)$ = height of excavation (void); and
- $a(x)$ = roof convergence (subsidence) factor.

If $m(x)$ and $a(x)$ are constants (they do not change along the x coordinate), then:

$$S_0(x) = S_{\max} = m a = \text{constant for } x_1 \leq x \leq x_2 \quad (4)$$

where, x_1 and x_2 are the limits of the excavation.

Finally:

$$S(s) = \frac{S_{\max}}{r} \int_{x_1}^{x_2} \exp \left[-\pi \frac{(x-s)^2}{r^2} \right] dx \quad (5)$$

The Zone Area Method

The original work on the zone area method is accredited to both German and Dutch engineers (Bals, 1932; Grond, 1953; Sinclair, 1963; Marr, 1975; Shadbolt, 1978). According to this principle, superposition is also used to calculate ground settlement. Complete extraction of the influence area will cause maximum subsidence, whereas partial extraction will produce partial subsidence of the point in question. The current, refined approach of the zone area method is accredited to Marr (1975). This technique was primarily designed to compliment the Subsidence Engineers' Handbook (NCB, 1975) for predicting ground movements over non-uniform extractions.

The method can be presented mathematically as (Goodman, 1980):

$$s/m = z_1 a_1^n + z_2 a_2^n + z_3 a_3^n + z_4 a_4^n + z_5 a_5^n + z_6 a_6^n \quad (6)$$

where,

- z = zone factors;
- n = influence constant; and
- a = proportional extracted areas.

The zone factors and the influence constant can be derived from field measurements.

FIELD DATA

The monitoring program included a total of sixteen room and pillar and seven longwall panels in seven mines located in Wise, Dickenson and Buchanan counties in Virginia.

Because of the large number of subsidence monuments above each mine, the number of panels under investigation and the mountainous terrain, a high accuracy, total station system was selected, utilizing a recording computer tachometer for measuring both vertical and horizontal surface movements. The operator was able to measure up to 150 points, over a six hour period, utilizing five to eight traverse stations.

The basic system included the surveying instrument, a number of reflectors, three tripods, a number of reflector rods, an HP86 computer with two disc drives, a data converter, a printer, a plotter and a digitizing table.

Specially designed adaptors, similar to those used by other researchers (Allgaier, 1981), were made for the secure attachment of the reflector

rods on the monuments, as shown in Figure 1. These special adaptors proved to be necessary for avoiding errors due to movements during the time required for distance measurement (about 5 seconds).

The monuments used in this study were of one inch diameter hot-roll steel rods, with a sharp machined-edge on one end, which were driven into the ground using either a sledge hammer or a gasoline-powered jackhammer, depending upon ground conditions. Monument lengths ranged from 2 to 5 feet, penetrating the ground in all instances to a depth of at least 1 foot beyond the frost line. A special adaptor was placed at the top of each monument to prevent deformation during installation.

A number of longitudinal and transverse monument lines were installed above each panel. The lines were extended well beyond the maximum expected area of influence on either side (0.8 times the depth beyond the edges of the extraction) to ensure the determination of the angle of draw. A set of reference points was also established outside of the area of influence for each case study. Following the guidelines suggested by the British National Coal Board (N.C.B., 1975), which are also accepted in this country, the spacing of the monuments was approximately 0.05 times the depth, to allow the accurate calculation of the distribution of horizontal strains along the monitoring lines.

The final effort included approximately 1,200 stations along 35,000 feet of monitoring lines.

BASIC RELATIONSHIPS FOR LONGWALL AND HIGH EXTRACTION ROOM AND PILLAR PANELS

Data from longwall and high extraction room and pillar case studies were combined, taking into account a correction factor for the extraction ratio. Thus the sample size was increased and the resulting relationships were used for both longwall and room and pillar case predictions. Information from different panels in the same mine was averaged, if their mining, lithologic, topographic and subsidence parameters were comparable. Otherwise these panels were treated as separate case studies. However, although the basic mining parameters were easily determined for all the monitored case studies, certain subsidence parameters, i.e. angle of draw, inflection point position and, most importantly, values of horizontal movement, were not available in many of the collected cases.

A number of relationships were established based on the data collected from all case studies, and these relationships were used for the regional application of the above mentioned prediction methods, including:

- Correlation of maximum subsidence, S_{max} , to mining and geological factors pertaining to the excavated area.

- Evaluation of ribside subsidence value and location of inflection point, from mining factors.

Figure 2 shows the maximum subsidence factor, i.e. S_{max}/m , where m is the mining height, as a function of the panel width-to-depth ratio and the percent of hard rock material in the overburden for the longwall case studies (hard rock has been defined in previous publications; Karmis et al., 1984; VPI&SU, 1987).

Figure 3 shows the maximum subsidence factor, i.e. S_{max}/m , divided by the extraction ratio, as a function of the panel width-to-depth ratio and percent hard rock in the overburden, for high extraction room and pillar case studies. Since, under typical mining practices, the maximum extraction ratio may be taken as 85 percent, this upper limit was assumed even for case studies claiming a higher extraction ratio.

The similarity in the relationship between the location of the inflection point and the width-to-depth ratio for longwall and room and pillar case studies allowed for all data points to be considered collectively as shown in Figure 4. An envelope line, (Figure 5), however, was preferred for prediction purposes, despite its potential for underpredicting the distance of the inflection point to the rib, thus resulting in a higher value of ribside subsidence and a wider area of influence. This nomogram is recommended when using the influence and profile function methods. The distance of the inflection point to the rib, however, can be accepted to be directly proportional to the depth for supercritical conditions, only within the limits of the data base used in this analysis, i.e. for depths ranging from about 300 to 1000 feet. A maximum value of $d = 230$ feet is, therefore, recommended.

Ribside subsidence, in terms of maximum subsidence, is important for the application of the zone area method. Figure 6 presents this relationship, with respect to W/h ratio, for all case studies. A minimum value of $S_r/S_{max} = 0.12$ is recommended when calculating the zone factors and the exponent n for supercritical cases.

The angle of principal influence (β or, usually, $\tan \beta$) is one of the basic parameters used in the influence function method. The value of this parameter has a great significance on the distribution of the deformations on the surface, as well as on the predicted values of the deformation indices, such as subsidence, slope and strain.

This parameter may be related to the geological characteristics of the overburden; however, a statistical relationship could not be established in this analysis for prediction purposes. An average value was, therefore, calculated from the case studies monitored in this study, as well as from other collected data pertaining to the Appalachian coalfield, as follows:

$$\tan \beta = 2.31 \pm 0.40 \quad (7)$$

Factor B was used to calculate horizontal strain as a function of curvature. In the original stages of this research a direct relationship between strain and curvature was sought which could describe B independently of any other mining parameters (Karmis et al., 1983). As more case studies became available through this project, it became apparent that such a relationship would be difficult to establish. As a result, a different approach was adopted, based on the work of Awershin (1947), Budryk (1953) and Akimov and Zemicev (1970), which has suggested that the magnitude of the horizontal strain factor (B) is a function of the excavation depth or the radius of principal influence (r).

For each of the monitored case studies, as well as for two literature case studies where horizontal measurements were available, factor B was determined by comparing the measured strain and the fitted curvature profiles. Using the established values of parameter B and the corresponding values of r, h and $\tan \beta$, a statistical relationship was found (Figure 7), expressed by the equation:

$$\begin{aligned} B &= (0.35 \pm 0.05) r \\ &\text{or} \\ B &= (0.35 \pm 0.05) h / \tan \beta \end{aligned} \quad (8)$$

where,

r = radius of the principal influence;
h = depth of the excavation; and
 β = angle of the principal influence.

The second equation is more convenient for prediction purposes.

COMPUTER SOFTWARE

The Profile Function Prediction Program

A computer program was developed for the application of the profile function prediction method. The functions describing the statistical correlation between mining and subsidence factors were included in the program, and, therefore, only the mining parameters need to be entered.

The program has been written in BASIC for the HP and IBM personal computers. Adaptations for site specific application will also require minimal changes. The results can be presented in a tabulated form or by delineating the profiles on a plotter (Figure 8).

Computer Software Based on the Influence Function Method

The distribution of subsidence related indices on the surface is described by a number of compli-

cated mathematical equations requiring considerable computational effort, and thus necessitating the use of computers. Therefore, the development of computer software to facilitate all calculations was a major factor in the utilization of this prediction approach.

Three sets of programs were developed; the first for the preparation of the data involving mining parameters, the second for the calculation of the deformation indices and the third for the graphical presentation of the results. A number of commercially available software packages may also be used for data preparation and graphical presentation, since special attention was given to software compatibility.

For the preparation of the input mining data, the following programs can be used:

- A. **COORDS:** (PacSoft Inc., 1983) This program creates the coordinate file of the prediction points on the surface, which can be used as an input file for the prediction programs. The data is entered manually using the keyboard.
- B. **7580 DIGITIZING:** (PacSoft Inc., 1983) This program creates the coordinate file of the prediction points on the surface directly from a topographic map of the surface, using a digitizer or a plotter.
- C. **MINDIG:** This program creates a new data file or edits an existing file describing the mine plan, including information about the parcel boundaries as well as their mining parameters (mining height elevation, subsidence factor). It is used for polygonal parcels with the use of a digitizer or plotter.
- D. **MINDIGr:** Similar to MINDIG, to be used for rectangular parcels.

For the calculation of the deformation indices six programs were developed, according to the shape of the parcels and to the required output:

- A. **DEFPRED:** This program calculates the values of the deformation indices at any point on the surface (Prediction Points File) and creates files (xxPREdyyy) containing the calculated data. The shape of the parcels may be polygonal or rectangular.
- B. **DEFPREDg:** This program calculates the values of the deformation indices at the grid points defined on the surface and creates files containing the calculated data. The shape of the parcels may be polygonal or rectangular. The output files (xxTOPOyyy) are compatible to PacSoft Inc. software, which can be used for plotting profiles or contours from the results.
- C. **DEFRECT:** This program calculates the values of the deformation indices at any point on the surface (Prediction Points File) and creates files (xxPREdyyy) containing the calculated

data. The shape of the parcels must be rectangular and the sides parallel to the axes of the coordinate system.

- D. **DEFRECTg**: This program calculates the values of the deformation indices at the grid points defined on the surface and creates files containing the calculated data. The shape of the parcels must be rectangular and the sides parallel to the axes of the coordinate system. The output files (xxTOPOyyy) are compatible to PacSoft Inc. software, which can be used for plotting profiles or contours from the results.
- E. **DEFRECTl**: This program calculates the values of the deformation indices at points along a curvilinear line defined on the surface. It may be used for prediction along roads, pipelines, railroads or power lines. It creates files (xxPREYyyy) containing the calculated data. The shape of the parcels must be rectangular and the sides parallel to the axes of the coordinate system.
- F. **DEFOPT**: This program calculates the values of the deformation indices at grid points defined on the surface and creates files containing the calculated data. It is recommended for analysis leading to the optimal shape of the excavation front, in the case of staggered or concave fronts. The shape of the parcels must be rectangular and the sides parallel to the axes of the coordinate system. The output files (xxTOPOyyy) are compatible to PacSoft Inc. software, which can be used for plotting profiles or contours from the results.

For the presentation of the results two programs were written for the plotting of profiles (DEFGRAPH) and three dimensional projections (DEF3-D). Contour plotting software developed by third parties may also be used.

These programs were initially developed in BASIC for HP computers. Use of a digitizer or of a plotter with digitizing capabilities of appropriate dimensions is recommended. Versions of the programs for use with the IBM-PC are also available, but input parameters and coordinates must be entered manually.

The Zone Area Method

The development of the zone area computer model required a number of assumptions. The pillars and panels involved in the modeling were assumed to be rectangular in shape. This facilitated development of an algorithm for computing the necessary proportional extracted areas for each zone. The requirement of rectangular pillars and panels is a reasonable assumption, approximating most mining layouts. Panels and pillars were also oriented parallel to a generated north-south coordinate grid, in that two opposite sides were situated in the north-south direction. This allowed easy designation of panel and pillar boundaries by a series of linear equations.

Critical to the calculation of subsidence by the zone area method is the determination of the proportional extracted areas within each zone. To determine these quantities, a subtractive method was employed, where all pillars and panels were checked for an intersection with a particular zone. The intersected areas were summed and then subtracted from the area of the zone. This difference divided by the zone's area yielded the proportional extracted area.

To determine all intersections between zones and either panels or pillars, it is necessary to represent the zones in a numerical format. Because the zones are circular in nature, they can, therefore, be represented by the equation of a circle. The intersections between the various boundaries are easily defined algebraically by solving, simultaneously, a zone equation and the pillar/panel equations. Knowing the intersection coordinates between panels or pillars and each of the six zones, the proportional extracted areas were calculated from the resultant sectors. At most three sectors were formed by the intersection. The areas of these three sectors were calculated analytically using a double integration method, which accounts for the circular boundary of each zone.

The computer version of the zone area model is written in MS Fortran for use on the IBM personal computer or a compatible.

ACCURACY OF PREDICTIONS

In order to demonstrate the accuracy of the prediction methods, a number of case studies were used. It should be mentioned that these case studies were included in the statistical analysis.

The parameters, as given by the mining companies, were used as input for the prediction methods. The resulting subsidence and strain profiles were then compared to the field data.

Figure 9 presents measured and predicted subsidence profiles, for one case study, using the three prediction methods.

It may be observed that the prediction of $S_{\max}$ was very accurate, and, as expected, all three prediction methods resulted in very similar prediction profiles.

On the strain profile, derived from the influence function method, the measured profile appeared to have been translated outwards from the center of the panel (Figure 10). This phenomenon may be attributed to the high surface slope in this location which can influence horizontal movements, by downslope translation, much more significantly than vertical settlement. This phenomenon has been observed by other investigators (Franks and Geddes, 1986). However, the limited strain data did not permit for a quantitative analysis of the effects of surface topography on strain distribution.

Figures 11 and 12 present a comparison between field data and predicted profiles for randomly selected case studies, using the profile function method.

A comparison of predicted and field values for $S_{\max}$ and d is presented in Tables 1 through 4. From Table 1 it may be observed that the prediction of $S_{\max}$ for longwall panels is very accurate (average error 16 percent), with the exception of a number of panels within the same mine (LPPA4 a, b, c) which were not monitored in this study. For room and pillar cases the average error was 31 percent. This is due to the fact that the extraction ratio often reported by mining companies, which is of great significance to the analysis of the results, was not always representative.

The predicted values of d were generally higher than the measured values (Tables 3 and 4). This result was expected, since the envelope line was used for prediction. It should be emphasized that an error of 35 percent, reflects a horizontal distance on the surface of less than 45 feet, for a typical mining depth of 600 feet, which is well within acceptable limits. In some cases, for design purposes, it may be preferable to predict d using the average (Figure 4) rather than the envelope relationship.

CONCLUSIONS

Surface movements due to underground mining exhibit regional characteristics and, as a result, the development of prediction methods must rely on the collection of local field data.

The research showed that the profile function method can be used for rapid calculation of vertical settlements on a line above a mine panel. This method is recommended for subsidence prediction in cases where a general distribution of the expected subsidence values above a mine is required.

The influence function approach is designed to yield a more detailed and complete calculation of ground movements, in any mode and at any depth, on a grid or at any specific point, than any other method. The study showed that the Budryk-Knothe function is a formulation applicable to the eastern United States mining conditions. It can calculate all types of surface deformations and, furthermore, can be easily adapted to mine sections of complex geometrical configuration.

The zone area method has merits similar to those of the influence function. Predictions in this case, however, are limited to the vertical settlement component only.

The methods developed in this study are recommended for general prediction purposes. The empirical relationships derived from the field data were based on simple statistical analyses, which, in many instances, resulted in envelope lines due

to insufficient data. As a result, a margin of safety is associated with these predictions (e.g. calculation of $S_{\max}$ or position of inflection point). For cases where the accuracy of the ground movement predictions is of critical importance, e.g. where the impact of ground movement on certain structures is likely to be severe, it is strongly recommended that the required field parameters used in the calculations be re-established from a monitoring program on the site in question. The general prediction methods described in this report can, subsequently, be easily adjusted to become site specific.

ACKNOWLEDGEMENTS

This paper has presented a summary of results developed under a comprehensive research effort conducted by Virginia Polytechnic Institute and State University in the past several years.

A number of agencies and companies have supported various parts of this research, primarily the Powell River Project, the Office of Surface Mining, Reclamation and Enforcement and the MMRRI program of the Bureau of Mines. Any opinions, findings, conclusions or recommendations expressed in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

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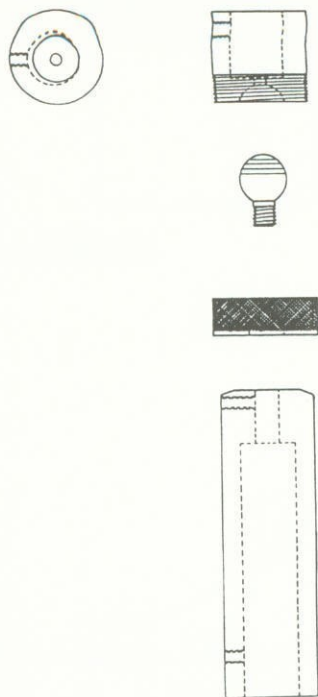


Figure 1. Adapter for Centering the Reflector Rod on the Monument.

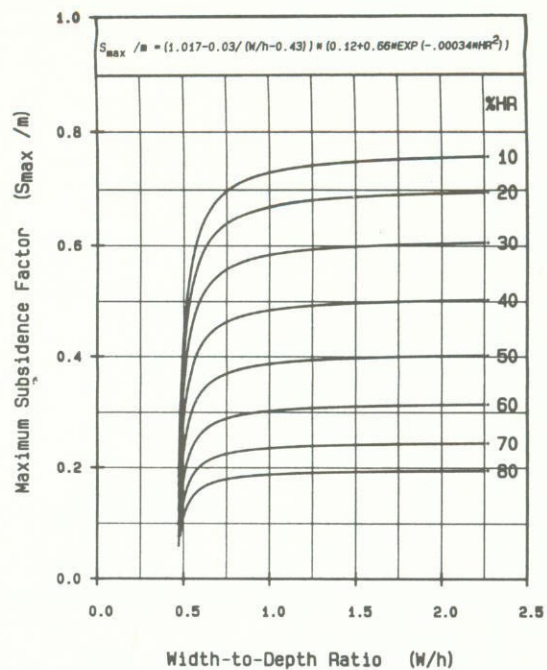


Figure 2. Chart for Predicting the Maximum Subsidence Factor for Longwall Mines.

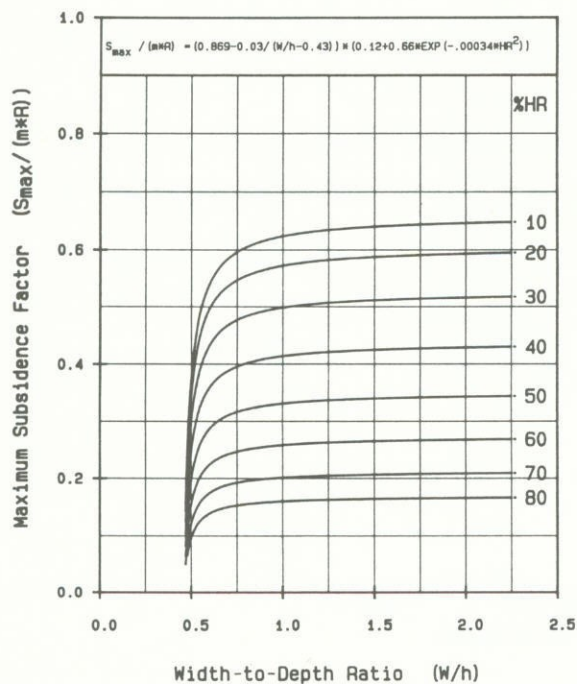


Figure 3. Chart for Predicting the Maximum Subsidence Factor for Room and Pillar Mines.

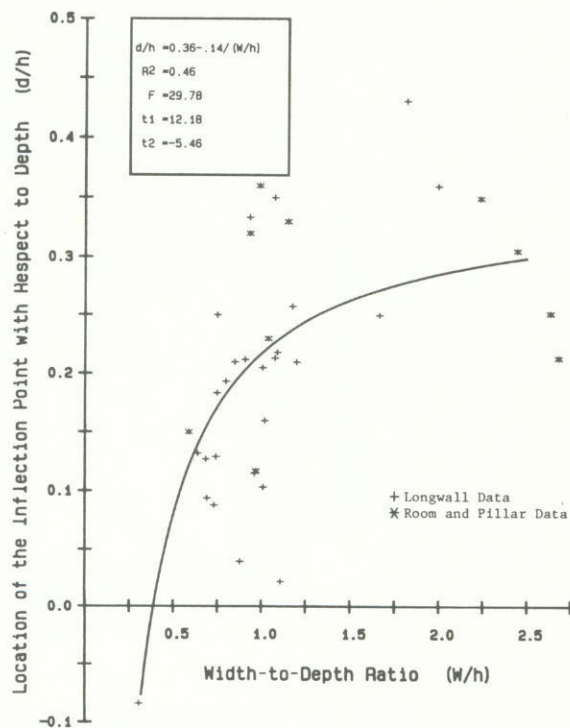


Figure 4. Effect of Width-to-Depth Ratio on the Location of the Inflection Point.

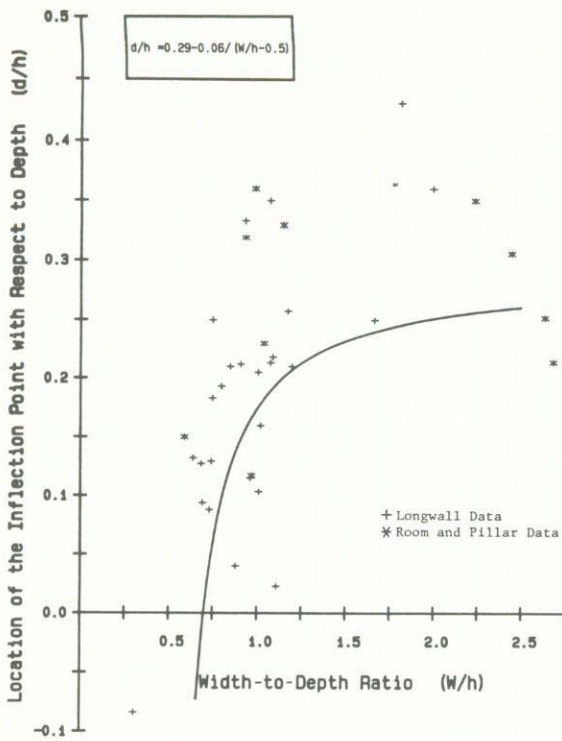


Figure 5. Location of the Inflection Point Using an Envelope Line.

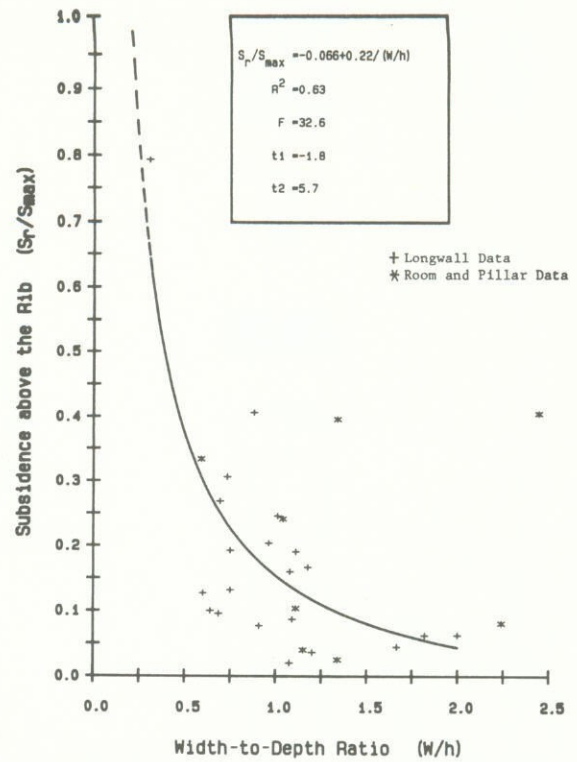


Figure 6. Prediction of Ribside Subsidence.

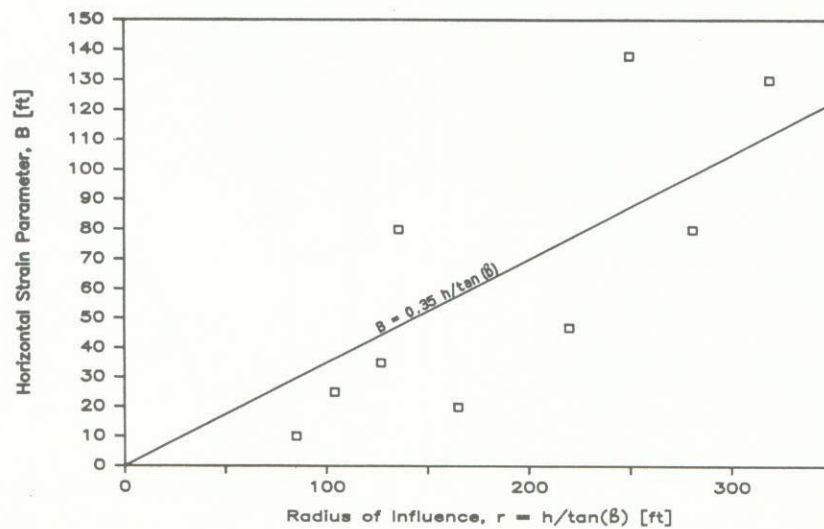


Figure 7. Effect of Radius of Influence on the Horizontal Strain Parameter.

Subsidence Prediction
Hyperbolic Tangent Profile Function
 Developed at VA TECH

INPUT PARAMETERS:

Seam thickness (m): 5 ft
 Panel width (W): 560 ft
 Depth of excavation: 600 ft
 Percent hardrock in the overburden: 50
CALCULATED PARAMETERS

Smax = 2.37 ft
 d = .16 ft
 B = 279.84 ft
 Panel is subcritical
 Distance (X) from panel center

X	Subsidence	X	Subsidence
0	2.4	350	0.8
14	2.3	364	0.8
28	2.3	378	0.7
42	2.3	392	0.6
56	2.3	406	0.6
70	2.2	420	0.5
84	2.2	434	0.4
98	2.2	448	0.4
112	2.1	462	0.3
126	2.1	476	0.3
140	2.0	490	0.3
154	2.0	504	0.2
168	1.9	518	0.2
182	1.8	532	0.2
196	1.8	546	0.2
210	1.7	560	0.1
224	1.6	574	0.1
238	1.5	588	0.1
252	1.4	602	0.1
266	1.3	616	0.1
280	1.3	630	0.1
294	1.2	644	0.1
308	1.1	658	0.1
322	1.0	672	0.0
336	0.9	686	0.0
350	0.8	700	0.0

Figure 8. Typical Printout--Profile Function Method.

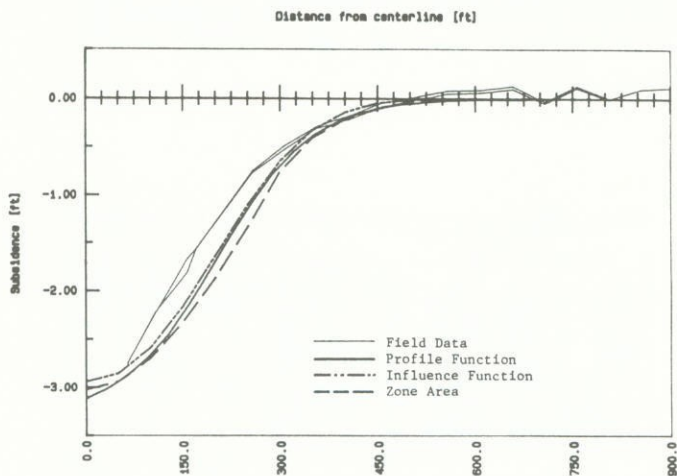


Figure 9. Predicted and Measured Subsidence Profiles for Case Study LUVA-VT1.

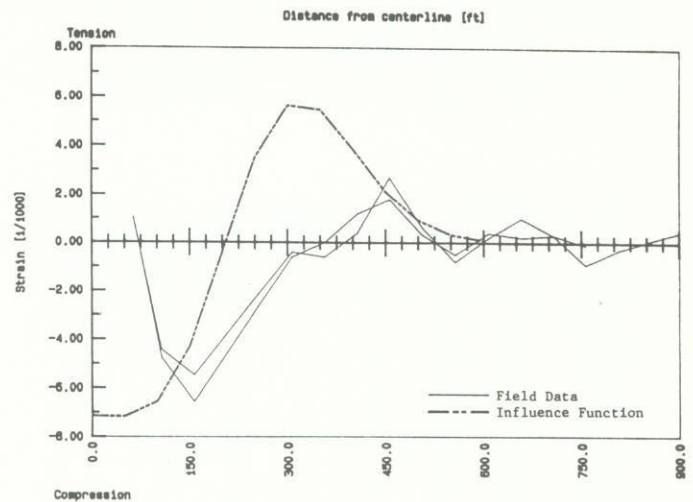


Figure 10. Predicted and Measured Strain Profiles for Case Study LUVA-VT1.

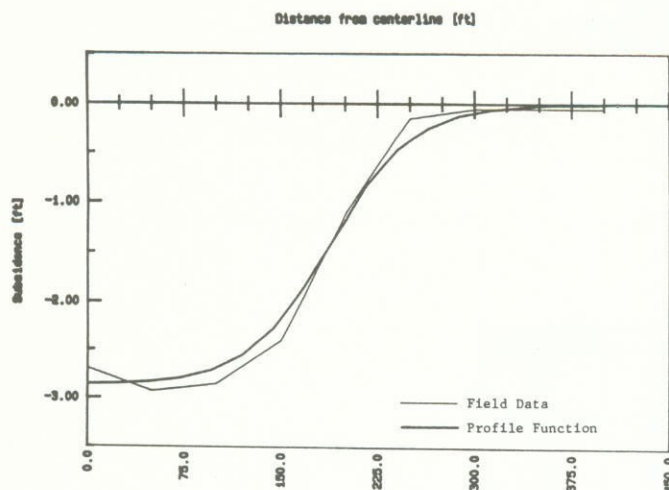


Figure 11. Predicted and Measured Subsidence Profiles for a Room and Pillar Case Study in West Virginia (Case Study RUWV4a).

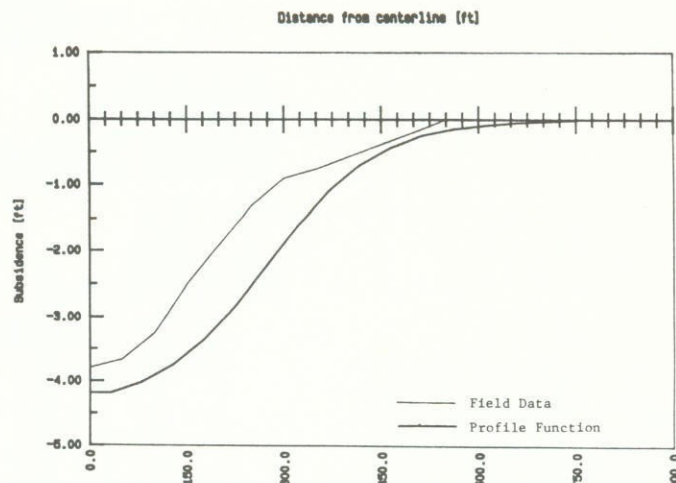


Figure 12. Predicted and Measured Subsidence Profiles for a Longwall Case Study in Ohio (Case Study LPOH1a).

Table 1. Comparison of Measured and Predicted Smax/m values for Longwall Case Studies and Relative Error (for $h < 1500$ ft).

Case Study	h (ft)	W (ft)	m (ft)	HR (%)	Smax/m measured	Smax/m predicted	Error (%)
LPAL1	500	600	4.5	26	0.611	0.630	3
LPOH1a	750	485	6.5	23	0.615	0.591	4
LPOH1c	700	480	7.25	24	0.648	0.594	8
LPOH2	700	485	4.3	23	0.581	0.605	4
LPPA2	550	610	4.5	41	0.467	0.479	3
LPPA4ab	510	450	4.15	31	0.32	0.566	77
LPPA4c	455	460	4	39	0.305	0.496	63
LPPA4d	290	445	4	26	0.563	0.638	13
LPPA7	400	470	3.7	21	0.649	0.672	4
LPWV1a	600	450	5.5	37	0.473	0.493	4
LPWV1b	600	450	5.5	37	0.345	0.493	43
LPWV1c	660	600	5.5	23	0.591	0.641	8
LPWV1d	550	600	5.5	23	0.733	0.652	11
LPWV1e	600	605	5.5	37	0.736	0.516	30
LUWV3	480	450	7	23	0.591	0.643	9
LUWV4a	300	500	4.9	32	0.596	0.582	2
LUWV4b	250	500	4.9	32	0.524	0.585	12
LPWV5	620	450	6	41	0.367	0.452	23
LPWV7a	750	600	5.5	25	0.618	0.612	1
LPWV7b	680	600	5.5	29	0.581	0.585	1
LPWV9ab	275	500	5	58	0.29	0.329	13
LPWV10a	650	450	--	55	0.38	0.321	16
LPWV10b	600	335	--	51	0.35	0.309	12
LUVAVT1	700	520	7.25	46	0.497	0.406	18
LUVAVT2	600	560	5.5	50	0.473	0.385	19
average							16

Table 3. Comparison of Measured and Predicted d/h values for Longwall Case Studies and Relative Error (for $h < 1500$ ft and d positive).

Case Study	h (ft)	W (ft)	m (ft)	HR (%)	d/h measured	d/h predicted	Error (%)
LPAL1	500	600	4.5	26	0.21	0.21	1
LPOH1bc	450	485	6.5	--	0.21	0.19	10
LPPA2	550	610	4.5	41	0.23	0.19	15
LPPA4c	455	460	4	39	0.1	0.18	76
LPPA6	400	430	5.5	25	0.35	0.19	46
LPPA7	400	470	3.7	21	0.26	0.20	21
LPWV1a	600	450	5.5	37	0.25	0.06	77
LPWV1b	600	450	5.5	37	0.18	0.06	68
LPWV1c	660	600	5.5	23	0.21	0.15	29
LPWV1d	550	600	5.5	23	0.22	0.19	13
LPWV1e	600	605	5.5	37	0.21	0.18	16
LUWV4a	300	500	4.9	32	0.25	0.24	4
LUWV4b	250	500	4.9	32	0.36	0.25	30
LPWV7a	750	600	5.5	25	0.19	0.10	49
LPWV9ab	275	500	5	58	0.43	0.25	43
LPWV9c	520	500	6.2	--	0.12	0.16	37
LUVAVT1	700	520	7.25	46	0.13	0.05	63
LUVAVT2	600	560	5.5	50	0.33	0.16	53
average							37

Table 2. Comparison of Measured and Predicted Smax/m values for Room and Pillar Case Studies and Relative Error

Case Study	h (ft)	W (ft)	m (ft)	HR (%)	R (%)	Smax/m measured	Smax/m predicted	Error (%)
RPPA1	325.00	360.00	6.50	22.00	75.00	0.29	0.42	45
RPPA2a	375.00	430.00	6.50	15.00	80.00	0.50	0.48	3
RPPA3	375.00	840.00	4.70	39.00	90.00	0.62	0.39	36
RPPA4	650.00	910.00	6.20	31.00	88.00	0.20	0.44	120
RPPA5a	725.00	720.00	6.00	33.00	85.00	0.52	0.40	23
RPPA5b	750.00	1460.00	6.10	29.00	88.00	0.46	0.46	0
RPPA6	745.00	1955.00	5.70	30.00	88.00	0.46	0.46	1
RPPA7	535.00	500.00	6.50	31.00	88.00	0.54	0.42	21
RPPA9a	350.00	470.00	7.20	41.00	85.00	0.48	0.35	27
RPPA9b	425.00	570.00	7.20	41.00	85.00	0.40	0.35	12
RPPA10	532.00	1100.00	6.80	41.00	85.00	0.52	0.36	31
RUPA34	354.00	300.00	5.40	20.00	90.00	0.49	0.50	2
RPWV1	600.00	600.00	5.50	50.00	90.00	0.32	0.30	8
RPWV2	555.00	325.00	6.00	47.00	90.00	0.30	0.26	12
RUWV4	800.00	830.00	5.00	64.00	78.00	0.29	0.18	37
RUWV4	690.00	400.00	6.50	70.00	100.00	0.26	0.16	37
RUVAVT1	540.00	525.00	5.00	32.00	85.00	0.43	0.41	6
RUVAVT2	365.00	980.00	5.00	80.00	85.00	0.42	0.14	66
RUVAVT3	258.00	680.00	5.50	80.00	85.00	0.53	0.14	73
RUVAVT5	590.00	2000.00	3.70	50.00	85.00	0.57	0.29	49
RUVAVT6	490.00	1200.00	3.90	50.00	85.00	0.42	0.29	30
average								31

Table 4. Comparison of Measured and Predicted d/h values for Room and Pillar Case Studies and Relative Error

Case Studies	h (ft)	W (ft)	m (ft)	HR (%)	R (%)	d/h measured	d/h predicted	Error (%)
RPPA2a	375.00	430.00	6.50	15.00	80.00	0.33	0.20	39
RPPA3	375.00	840.00	4.70	39.00	90.00	0.35	0.26	27
RPPA5a	725.00	720.00	6.00	33.00	85.00	0.36	0.17	52
RPPA7	535.00	500.00	6.50	31.00	88.00	0.32	0.16	51
RPWV2	800.00	830.00	5.00	64.00	78.00	0.23	0.18	21
RUVAVT1	540.00	525.00	5.00	32.00	85.00	0.12	0.17	43
RUVAVT2	365.00	980.00	5.00	80.00	85.00	0.21	0.26	23
RUVAVT3	258.00	680.00	5.50	80.00	85.00	0.25	0.26	4
RUVAVT5	590.00	2000.00	3.70	50.00	85.00	0.20	0.27	33
RUVAVT6	490.00	1200.00	3.90	50.00	85.00	0.31	0.26	15
average								31

MECHANISMS OF CHIMNEY SUBSIDENCE OVER ABANDONED COAL MINES

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INTRODUCTION

Mining related subsidence is a major concern over abandoned shallow coal mines. Many of the subsidence prone areas are presently used or will be used in the future for residential housing development. Concern over mine subsidence is inhibiting the development of surface land uses on previously mined area. This concern prompted public officials to realize the need to develop guidelines which can be used to assess and reduce the risk of mine subsidence damage to building and other improvements. Subsidence and the associated ground movements must be defined before one can relate them to building damage.

Surface movements resulting from underground coal mining occurs in two phases termed as: active subsidence and residual subsidence. Active subsidence refers to all surface movements occurring simultaneously with the mining operations, while residual subsidence is that part of the surface deformation which occurs following the cessation of mining.

Residual subsidence is the major concern over abandoned mine lands. The duration of residual subsidence is of particular importance from the standpoint of damage at the surface to structures built after the mine has been abandoned.

Time spans during which surface subsidence may occur, vary markedly with the mining method being used. In total extraction methods, such as longwalls, subsidence is induced rapidly, beginning almost immediately after mining. Subsidence concurrent with coal extraction is desirable for a safe longwall mining operation. Usually subsidence is complete within a relatively short time. The remaining settlement occurs as residual subsidence due to the gradual compaction of the subsided ground. The relative proportion of the residual subsidence movements is dependent upon the presence

of remnant pillars and roadside packs, rate of face advance and depth of workings. In partial extraction methods, such as with room-and-pillar systems, major occurrences of surface subsidence may be delayed for decades until the support pillars and/or the roof have substantially deteriorated and collapsed. The actual time involved depends on a number of factors such as the strength of coal, roof, and floor, extent of fracturing, presence of water, depth of workings, pillar size and percentage extraction. Consequently residual subsidence is more likely to occur over abandoned room-and-pillar mines.

The following sections define subsidence over abandoned mines, review the proposed mechanisms, identify the controlling factors and examine the prediction methods.

SUBSIDENCE OVER ABANDONED MINES

Generally, there are two types of surface affecting subsidence associated with abandoned underground mining. These are: (1) troughing, which is a gentle, large area depression of the terrain, and (2) sink-hole, conical depressions or chimney subsidence. The first type of subsidence can be readily detected in the very early stages of the process, and surface damages caused by it are relatively small. The second type of subsidence is due to underground mine roof failure in localized areas where caving is propagated to the surface to form a chimney or sink-hole. This type of subsidence can be quite hazardous to life and property due to the absence of any surface indications until the chimney is formed, and to the inability to predict where or when it will occur, especially if the geometry of the underground workings is unknown. Although such phenomena as trough subsidence and caving are common occurrences, the basic mechanisms associated with the phenomena are not well known.

The three basic mechanisms responsible for residual subsidence over room-and-pillar mines include:

- pillar failure,
- squeezes or punching,
- roof failure

Pillar failure and squeezes or punching, depending on factors such as depth, mined thickness, unsupported roof span, mechanical properties of the overlying strata and others, may result in the formation of a through subsidence at the surface.

Roof collapses depending on factors, such as mined thickness, depth and bulking, may form a chimney subsidence.

Trough subsidence is a gentle, continuous, shallow dish-shaped, large area depression in the ground surface formed by sagging or downwarping of the overlying rock formations into a mined-out area. Trough subsidence will cause vertical and horizontal displacements, slope or tilt, and in smaller occurrences curvature at the center.

Troughs above room-and-pillar mines, whatever their origin, exhibit geometries similar to those above active longwall mines. They are, however, almost always elliptical in plan rather than rectangular and usually measure only a few tens of feet to a few hundreds of feet in diameter.

Documented instances of troughs are far fewer than sinkholes, and it is probable that unless a trough extends beneath a structure and causes damage, it generally goes unnoticed.

Pillar failure and punching of pillars into the mine floor are thought to be the predominant mechanisms of trough subsidence over room-and-pillar mines.

Pillar failure is associated with (Figure 1) narrow pillars or fenders. Narrow pillars may be created after robbing or from long term deterioration by spalling, due to stress concentration, and/or by weathering. The smaller pillars, if unable to carry the loads, will fail. Pillars may also fail by strength reduction resulting from ground water fluctuations when the mine spaces are alternately flooded and dry.

In some instances [1] dewatering of a flooded mine may trigger pillar failure. In these cases it can be speculated that draining the mine may have withdrawn the buoyant support provided to the overburden by the water, causing the weight of the overburden to be transferred back onto the pillars. If the pillars had weakened

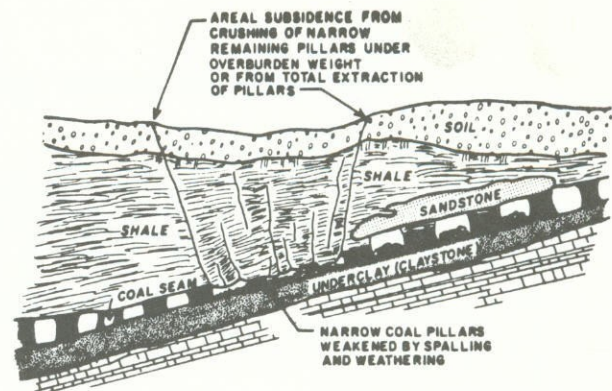


Figure 1. Trough Subsidence Associated with Pillar Failure [1].

since mining this increased load may be sufficient to cause one or more of them to fail.

Whatever the triggering mechanisms, the subsidence process is initiated by the failure of a single pillar, the first pillar whose strength drops below the level of the imposed stress. Failure may come either by slow yielding or by outright collapse, but in either case a portion of the load the pillar has been carrying is transferred to adjoining pillars. If the magnitude of the transferred load is sufficiently high, the adjoining pillars will also fail, causing still another redistribution of load. Successive pillar failure and stress redistribution will continue until the overburden is no longer able to span and fails by shear or by flexure.

Pillar punching occurs (Figure 2) when the bearing capacity of the mine floor material is exceeded [2,3]. The presence of weak coal, claystones or mudstones in

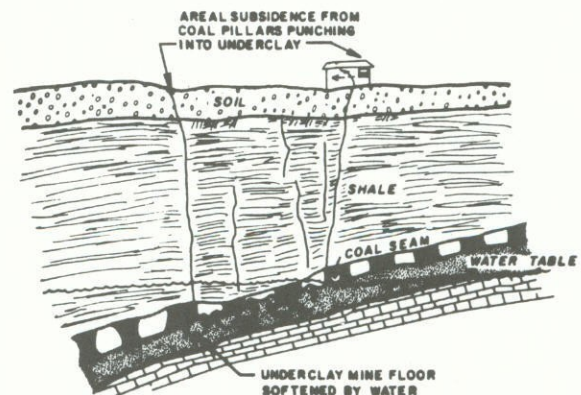


Figure 2. Trough Subsidence Associated with Pillar Punching [1].

the mine floor, high pillar stresses and flooded mine conditions are factors that favor bearing capacity failure as a subsidence mechanism. Weak claystones may deform pastically and move outward from beneath the pillar and upward into the mine openings when bearing stresses are greater than their yield strength, especially when these rocks are saturated. Pore water will soften the claystone and reduce its bearing capacity. Punching of the pillars into the mine floor will cause a downward displacement of the overlying rock formations and may result in trough subsidence at the surface.

It is unlikely that any large scale subsidence will take place above a mine closed and flooded for several decades if signs of such movement have not been noticed earlier. It is also likely that subsidence due to punching of pillars into the mine floor, once initiated, will take place over as wide as an area of the mine floor as is flooded and softened, in contrast to the relatively limited extent of subsidence due to pillar failure, which is governed by the thickness of overburden and which seems to initiate by the failure of a single pillar.

Through subsidence and associated ground movements over abandoned and active room and pillar mines have been studied [4-8] more extensively than chimney subsidence.

In the next section, chimney subsidence is defined, and the factors controlling the development of subsidence sinkholes are identified. The mechanisms involved in the development of chimney and means of early identification of impending chimney subsidence are discussed.

CHIMNEY SUBSIDENCE

Chimney subsidence results from the intermittent, sequential collapse or unravelling of underground mine roofs in localized areas whereby caving migrates through the overlying material, until the failure zone intercepts the unconsolidated overburden. The surface expression can either be a through, dish-like depression with a continuous and symmetric surface profile or a sinkhole with an abrupt surface expression, usually in the form of conical depression with the apex upward. In plan view, they are circular or oblong reflecting the geometry of the mine workings. In all reported studies on chimney subsidence, chimney cavings occur in overburden of less than 300 feet (Table I) and in zones of rock weakness or areas of extensive vertical rock fracturing [1,9-15].

Chimney subsidence results from the intermittent, sequential collapse or unravelling of underground mine roofs in localized areas, whereby caving migrates through the overlying material, until the failure zone intercepts the unconsolidated overburden.

Chimney subsidence surface features would appear to occur in random manner over a mining area, and it is this randomness or unpredictability that causes concern. Before the basis of predictive methods can be formulated, the cause of chimney caving, its mode of progression through to the surface, and the controlling factors must be discussed.

Mechanisms

Chimney subsidence is a roof stability or instability problem. Roof fall can be initiated by the change in the state of stress of the roof rock caused by the removal of the coal. The vertical overburden load, which was originally supported by a continuous seam of coal, must be transferred to the pillars by arching (Figure 3a). The rock immediately above the opening will be destressed.

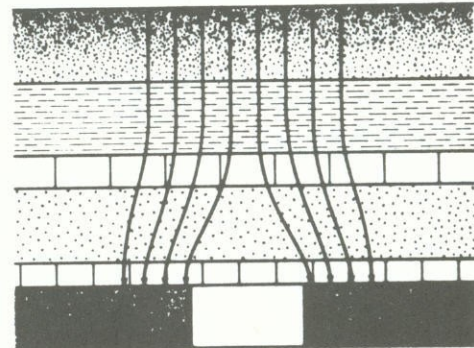


Figure 3a. Overburden Loads Not Carried by Immediate Roof

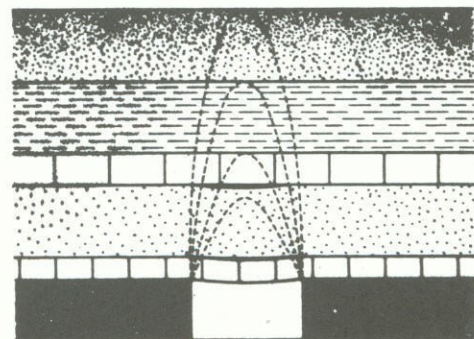


Figure 3b. Deflection of the Immediate Roof, Bed Separation and Roof Stress Variation.

TABLE I

Chimney Occurrence - Overburden Thickness Relationship			
Western Pennsylvania [1]	-	Overburden less than	150 ft.
Sheridan Area, Wyoming [9]	-	Overburden less than	200 ft.
Beulan Area, North Dakota [9]	-	Overburden less than	80 ft.
St. David Area, Illinois [10]	-	Overburden less than	165 ft.
Hanna Area, Wyoming [11]	-	Overburden less than	240 ft.
Colorado Springs Area, Colorado [12]	-	Overburden less than	150 ft.
Glenrock, Wyoming [13]	-	Overburden less than	100 ft.
Superior, Wyoming [14]	-	Overburden less than	100 ft.
Rock Springs, Wyoming [15]	-	Overburden less than	300 ft.

Depending upon the initial state of stress, tensile stresses on the immediate roof and/or high compressive stresses at the upper corners of the opening may develop. If the roof contains vertical fractures or joints, tensile stresses will not develop. Instead, opening of fractures may occur. In either case there will be a small downward deflection of the roof, and a self-supported linear arch, or Voussoir arch, will be formed (Figure 3b). Compressive stresses will develop on the upper part of the arch-forming layer and at the abutments.

The opening will stay stable as long as the stresses do not exceed the strengths of the roof rock. Over a period of time, roof stability may be jeopardized by the changes in the stresses and the strength of the roof rock.

The changes in stresses and strength may be due in part to ground waters, creep, or other factors. Fluctuations in ground water will affect the effective stresses by changing the hydrostatic head and the pore pressure within the rock mass, thus reducing the apparent strength. In Rock Springs, Wyoming, 80% of the chimney subsidence occur along a strip delineated by the fluctuation of the mine water pool level [15]. Slaking and slabbing, resulting from the variation in relative humidity and weathering of unstable minerals, will affect the stress distribution by modifying the boundary conditions of the mine opening. The presence of water may also deteriorate the strength of the rock [16]. The relationship between precipitation and sinkhole development has been shown by various authors [1,17]. Deformations due to creep could modify the stresses by stress relief. Furthermore, creep deformations may also exceed the plastic limit of the rock and cause failure.

The roof may fail in two basic modes, flexural failure and shear failure (Figure 4). If the roof is assumed to behave as a continuous beam, it will fail by flexure (Figure 4a): a) due to loading in a vertical plane; b) if the ratio of horizontal to vertical stress is low; c) if the roof is formed by thinly bedded layers or layers that have separated and have low stiffness; and d) if the tensile stresses generated by bending exceed the modulus of rupture of the rock. A continuous beam may also fail in shear when the horizontal and vertical stresses are high, the pillars are stiffer than the roof, softer layers overlay the immediate roof and when the shear stresses generated at the pillarside abutments exceed the shear strength of the roof rock (Figure 4b). If the roof is categorized as a Voussoir beam formed by blocks bounded by joints or cracks, it will fail in shear sliding along weakness planes when the vertical stress exceeds the shear resistance along the discontinuity plane. A Voussoir beam may also fail by rotation of blocks. Rotation may result when the upper, or abutment, portion of the arch is crushed due to compressive stresses exceeding the compressive strength of the material, or when deflections reduce the linear arch line beyond the limiting length [18,19].

After the initial failure of the immediate roof has begun, caving will continue to propagate upward to the surface, unless it is arrested by the formation of a new, stable, self-supported linear arch, by a more competent roof layer, by a vault that has achieved a stable geometry with respect to the stress conditions, or by bulking of roof debris. The failure of the roof is intermittent and progressive until the failure zone intercepts the unconsolidated overburden. The expression on the surface

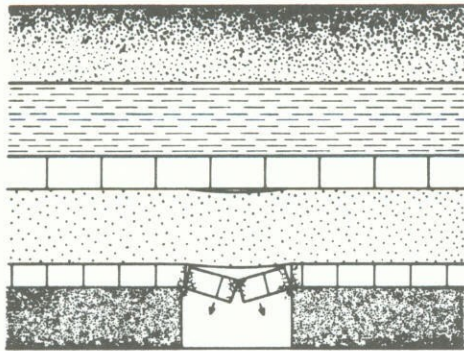


Figure 4a. Flexural Failure of Roof.

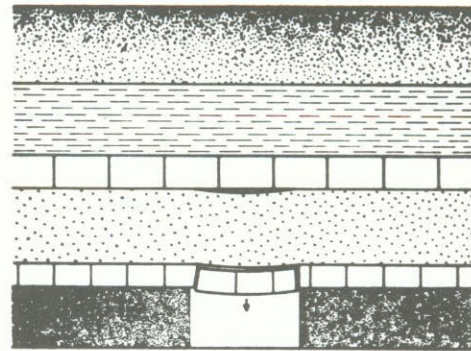


Figure 4b. Shear Failure of Roof.

will depend on the physical properties of the soil-like overburden. If the unconsolidated overburden is thick with high clay and water content it will behave as an elastic-plastic material [20,21]. Thus, a through will form with a continuous and symmetric surface profile as the overburden flows into the mine opening with a large angle of draw [22]. If the unconsolidated overburden is thin and dry with low clay content a pit or sinkhole will form with a discontinuous surface profile (Figure 5).

Prediction

As a result of public concern over the adverse effects of past mining activities, much attention has been focused on subsidence and its attendant problems. Questions commonly center around the possibility of predicting if, where, when and to what extent subsidence will occur. While several approaches to subsidence prediction exist, they only address the problem over active total extraction mining systems, and can not be

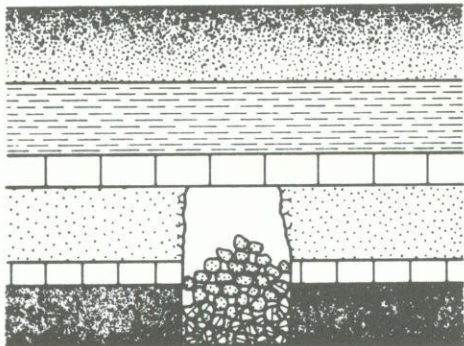


Figure 5a. Caving Arrested by a More Competent.

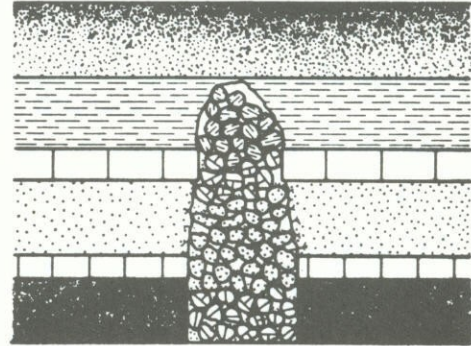


Figure 5b. Caving Arrested by Bulking of Roof Debris.

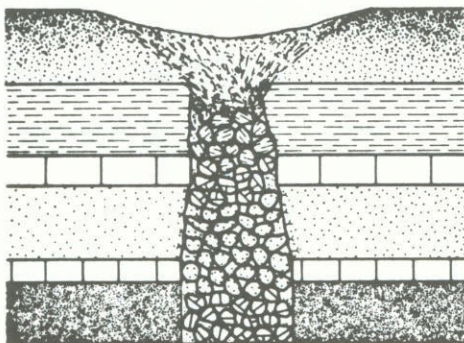


Figure 5c. Formation of a Trough Subsidence of the Surface.

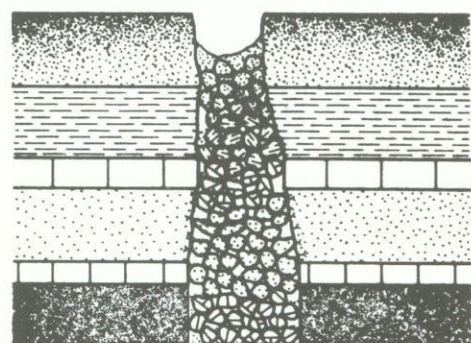


Figure 5d. Formation of a Sinkhole at the Surface.

used for subsidence over abandoned mines. The following briefly discusses the different approaches that can be taken in attempting to predict chimney subsidence over abandoned mines.

Subsidence prediction can be made at three levels: prediction of the probability that surface subsidence will occur at a specific location; prediction of the likelihood that a given degree of ground movement will occur in the event of subsidence; and, prediction of the severity of damage due to ground movement. While the third level of prediction is most desirable, only the first level of prediction is presently attainable for chimney subsidence. Based on subsidence mechanism modeling or analysis of historical data, the probability of subsidence may be determined.

Since chimney subsidence is a roof stability problem, roof failure modeling can be used in predicting its occurrence and describing its development.

Roof stability may be analyzed using the elastic beam and plates theories. The rock above the mine opening is assumed to act as a series of elastic beams or plates with assumed constraints and load distributions. The analysis relies on parameters difficult to estimate accurately, parameters such as roof layer thicknesses, continuity, and load distributions [20,23,24].

Voussoir beam theory can be used to predict the deformation behavior of the immediate roof. The theory assumes that the beam consists of a no-tension material and carries its weight by arching. The stability is evaluated in terms of the self-supported span, the thickness of the roof layer, and the elastic modulus and uniaxial compressive strength of the rock [23-25].

The Voussoir beam theory may be extended to accommodate the more general geometry of plates by applying the concept of yield lines as used in the design of reinforced concrete slabs [25-26]. The Voussoir theory makes a number of mechanically reasonable, but unsubstantiated assumptions. When the jointing in the roof layer is not vertical, as assumed in Voussoir theory, wedge analysis or block theory may be used for stability analysis [27,28].

Numerical analysis such as finite element, boundary element, and the distinct element method of analysis may also be used in the roof stability assessment. This approach would require a much better definition of material properties, initial stress, and boundary conditions than is

usually available when trying to predict chimney subsidence over abandoned mine land [34].

In working underground mines where information is more readily available, roof stability may be predicted. But, in abandoned mines at shallow depths, where geometry of opening, structure, and the properties of rocks are unknown or subsurface information is very limited, using the above mentioned models for predicting chimney subsidence may be unrealistic. Precise information would be required to accurately evaluate the rock properties needed for model application and to demonstrate that the model is appropriate to the mining features under study. Furthermore, the effects of vertical and horizontal discontinuities in the overburden must be evaluated. Thus, the extent of the required additional subsurface investigation may be substantial. This will be especially true with overburden that is as variable and discontinuous lithologically as most rock formations are over shallow abandoned mines.

All of the above, except for the numerical analysis approach, will only evaluate the stability of the roof without giving any indication whether, in the case of instability, the caving may propagate to the surface to form a sinkhole.

In a caving prone area an alternative approach to chimney subsidence prediction has been proposed by various authors [29-32]. In their approach, simple relationships were developed between the height of collapse, the thickness of extracted mineral, and the bulking characteristics of the roof material, to predict if a caving with a given geometry will reach ground level before being choked by rubble (Figure 6). It was

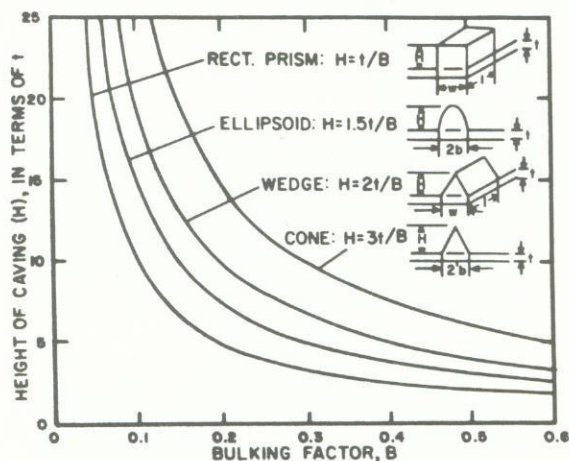


Figure 6. Maximum Height of Roof Caving [30].

stated that the greater the volume of the underground opening, the higher the process of collapse is likely to develop. Further, the smaller the change in volume of the caved material from the intact to loose state, the higher the height of the caving. Figure 7 shows the expected height of collapse for a given mining height and for an overburden with a 20% bulking factor. The relationship used for maximum stopping height determination assumes idealized geometries, constant bulking, and no lateral movement of the rubble. It is impossible to predict, a priori, the exact geometry of an impending chimney cave. Furthermore, the bulking factor will vary as the chimney cave propagates upward and intersects different rock formations. Lateral movements of rubble will occur in dipping seams and where ground water movements in and above the mined interval are present, thus giving rise to greater caving height predicted by the models based on bulking.

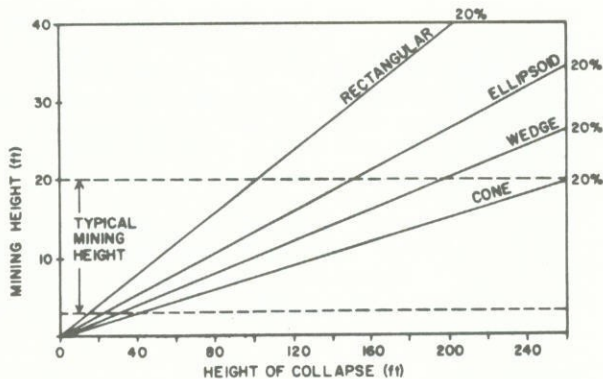


Figure 7. Collapse Height versus Mining Height for Bulking Factor of 20%.

Considering the assumptions and uncertainties in predicting chimney subsidence developments and the seemingly random nature of their occurrence, a more realistic approach would be to use the available historical data to make probabilistic prediction for the occurrence of subsidence; i.e., examining the spatial characteristics of existing chimney cavings.

As an example chimney subsidence and overburden thickness relationship for the Hanna 2 and Hanna 3 mines (Figure 8), is analysed using a probabilistic approach [33].

It was determined that the histogram follows a log-normal distribution, with a median thickness of 57 feet and a covariance of 56%. Based on this distribution, a cumulative probability curve of chimney subsidence for overburden thickness was developed (Figure 9). The cumulative probability plot can be used to

determine the probability of chimney occurrence for a given overburden depth range. From the curve, it can be seen that the probability of a chimney occurrence at overburden depth of more than 160 feet is 2%, and the probability for the range of 50 to 80 feet is 62%. The cumulative probability curve can give a measure of risk of sinkhole development within a given overburden depth interval.

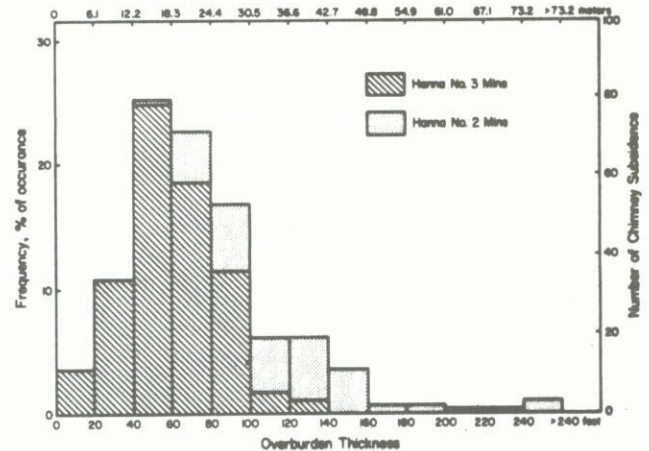


Figure 8. Frequency of Chimney Subsidence for Intervals of Overburden thickness at the Hanna Mine District - Wyoming [33].

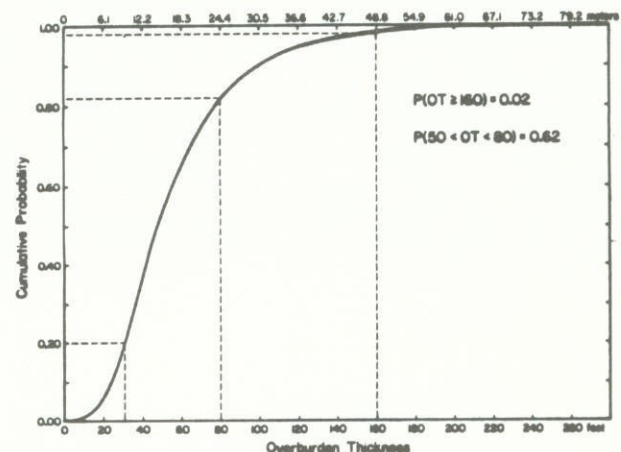


Figure 9. Cumulative Probability of Chimney Subsidence for Overburden Thickness [33].

The above discussion on prediction of subsidence features, if all required informations are available, will only give an idea on the likelihood of occurrence.

Predicting the magnitude and extent of ground movement and associated damage over abandoned coal mines is not currently

possible for several reasons, the principal are being that nothing or very little is known about mining-related ground movement above partial extraction systems. Moreover, even if reliable ground movement information did exist, there remains the problem of relating ground movements to building damage. Soil-structure interaction is a complex, three-dimensional phenomenon that has yet to be characterized in simple form. More study is needed to correlate the magnitude of ground movement with observed levels of damage for different categories of structures with various site conditions.

CONCLUSIONS

Since the fundamental objective of subsidence prediction is to evaluate potential damage to buildings rather than to estimate ground movements themselves, prediction and analysis of related ground movements is of little value unless the consequences of those movements can be determined and used for damage assessments. From a practical point of view, then, what is needed is a statistical approach to damage prediction which by-passes an explicit statement of soil-structure interaction, but incorporates in the analysis the primary parameters that govern ground movements, soil-structure interaction, and building deformation.

Complete and consistent documentation of subsidence damage, including data on structure, overburden characteristics, and mining, must be achieved if a realistic statistical predictive model is to be developed. Through the collection and analysis of this data, models can be generated which will satisfactorily determine the probability of damage severity for specific type structures, sites, and geologic and mining conditions.

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TIME-DEPENDENT BEHAVIOR OF IMMEDIATE WEAK
FLOOR STRATA FROM AN ILLINOIS COAL MINE

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ABSTRACT

The paper discusses laboratory and field studies to analyze time-dependent deformation behavior of immediate weak floor strata in an Illinois coal mine. In the laboratory, core samples of the immediate floor strata were subjected to incremental creep tests under unconfined compressive stress. Both axial and lateral creep deformations were recorded. Time-dependent behavior was reasonably characterized by a Burger model with one Kelvin-Voigt unit. The elastic and viscous parameters of the model were found to be stress-dependent, but in the range of 40% to 80% of the failure stress, the effect of stress was relatively small. Therefore, a linear visco-elastic model was developed to represent the laboratory time-dependent behavior of immediate floor strata. Such a model may be acceptable for the design of partial extraction mining systems having extraction ratios of 40% to 60%.

Field observations of roof-floor convergence (floor heave) at the mine also indicated a time-dependent behavior similar to that observed in the laboratory. Field time-dependent behavior was also characterized in terms of a Burger model with one Kelvin-Voigt unit to obtain scaling factors between the field and laboratory parameters. This approach should help in predicting likely floor heave in the mine based on laboratory creep tests. It was concluded that additional studies in the field are required to arrive at more reliable correlations between the two sets of parameters.

INTRODUCTION

Stability of a mining system in stratified deposits requires consideration of the interaction of roof, pillar and floor elements. Unless effective control

is obtained for all these elements, it becomes difficult to extract the mineral with the necessary degree of safety, efficiency and economy. Although the roof and pillar elements have been well-studied in the past, the floor element did not receive much attention until recently. It generally becomes the critical element in pillar design if coal seams are underlain by weak floor strata. The problems of floor heave in access roadways or pillar punching may also result in bad roof conditions impairing the stability of the overall mining structure.

Actively mined Illinois coal seams are usually underlain by moderately thick (2 ft to 7 ft) layer of weak underclays (300 to 1000 psi compressive strength). Several mines experience problems of floor heave, pillar punching and surface subsidence due to presence of weak floor. Limited data are available on the strength-deformation properties of weak underclay floor strata in Illinois. Data on time-dependent properties of underclay are totally lacking, even though it is known that the convergence and floor heave in the field are time-dependent.

Afrouz and Harvey (1) conducted laboratory creep experiments on air-dried and water-saturated underclay samples from a coal mine in South Wales. They found that on doubling the stress level, while the instantaneous creep increased by 50% to 80%, the secondary creep rate increased by about 90%. Saturation with water resulted in almost tenfold increase in instantaneous strains but only a twofold increase in secondary creep rate. Field observations of time-dependent deformation of weak floor strata were carried out by Johnson (2), Afrouz (3) and Sheorey and Durham (4) but all these studies pertain to floor heave problems in advancing longwall gate roadways in the United Kingdom. Published literature on time-dependent behavior of weak floor strata in room-and-pillar mining system in U.S. coal

mines are sparse. However, a few studies about the general geotechnical behavior of weak floor strata have been conducted by White (5, 6), Ganow (7), Rockaway and Stephenson (8) and Chugh (9, 10). The present study relates to the geotechnical properties including the time-dependent behavior of weak floor strata from an Illinois coal mine. The study was initiated under a grant from the Mine Systmes and Ground Control Generic Center at Virginia Polytechnic and State University.

DESCRIPTION AND ENGINEERING PROPERTIES OF IMMEDIATE FLOOR STRATA

The mine extracts Springfield No. 5 coal seam at depths varying from 250 to 400 ft. Core samples of immediated floor strata to a depth of about 20 ft were obtained at several sites in two panels at the mine; one on the west side and one on the east side. The immediate floor strata (Figure 1) at the mine consists of 4 to 6 ft of underclay which is highly sensitive to water. It is highly slickensided with poor to very poor Rock Quality Designation (RQD) ratings (0-20). Limestone nodules 0.25-2.0 in. in diameter are observed 2 ft or more below the coal seam and their frequency generally increases with increasing depth. The underclay is commonly underlain by relatively thin (0-12 in.) layers of shale, limestone, mudstone or claystone to a depth varying from 10 to 14 ft. Below these strata, weak moist underclay 12 to 24 in. thick is present. Water at a pressure of 10 psi is generally encountered at a depth varying from 14 to 16 ft.

The immediate floor strata can be divided into two zones depending on its natural moisture content (Figure 2). The upper 55 in. has a relatively uniform and much higher (7-9%) natural moisture content than the strata 55 in. or more below the coal seam (4-5%). Particle size analysis of the immediate floor strata samples (Table 1) show that most of the particles are in the size range of either clay (< .005 mm) or silt (.005 - 0.074 mm). While the upper 55 in. of immediate floor strata can be classified as clay or silty clay, the strata below this depth may be classified as silty clay, clay silty loam or silty loam (Figure 3) indicating a decrease in clay size particle content with increasing depth.

Illite and kaolinite were found to be the primary clay minerals aggregating to more than 50% in most cases; the remainder consisted of mixed-layers of illite and smectite. The ratio of smectite to illite in the mixed-layer clays ranged from 0.67

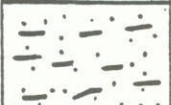
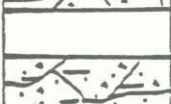
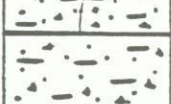
to 1.56. A decrease in mixed-layer content and a corresponding increase in illite content was observed with increasing depth below the coal seam. The mixed-layer content was higher (55.6%) in floor strata in the 0-55 in. depth range in samples obtained from the west side of the mine as compared to those from the east side of the mine (48%). Based on these data, a more plastic behavior of floor strata may be expected on the west side of the mine.

Atterberg limits data show that the mean values for plastic limit (P.L), liquid limit (L.L), and plasticity index (P.I) (Table 2) for 0-55 in. depth floor strata in the east side (24.34%, 42.8% and 18.40% respectively) are higher than those for strata below 55 in. (P.L = 14.85%, L.L = 26.61%, P.I = 12.61%). Similar observations hold good for the west side strata also.

Time-independent strength-deformation properties of the immediate floor strata are presented in Table 3. Unconfined compressive strength (C_o) and moduli of deformation were found to vary with depth below the coal seam. It was very difficult to obtain cores of weak and friable underclay and the test specimens usually came from stronger claystone or mudstone. Test results therefore are not truly representative of the entire thickness of immediate floor strata and may be biased towards the more intact samples. The C_o values usually ranged between 2000 to 4000 psi; modulus of elasticity (E) was on the order of 1 to 4×10^5 psi and Poisson's ratio (μ) ranged between 0.2 to 0.4. The E values at 50% of the C_o value were generally larger than that at 100% of the C_o value indicating larger elastic deformations just before failure. Stress-strain curves were generally s-shaped with low E and high μ values at low and high stress levels.

LABORATORY CREEP STUDIES

Eight samples, prepared from cores obtained from the immediate floor strata (four from east side and four from west side of the mine), were subjected to incremental creep studies under unconfined compressive stress. Axial as well as lateral strain-time behavior at various stress levels was studied. The studies were carried out either in a spring-loaded creep-testing frame (Singh, 11) designed and fabricated in the Department of Mining Engineering or in a 50,000 lb. servo-controlled electro-hydraulic MTS loading machine. Axial and lateral deformations were measured with linear variable differential transformers (LVDTs) with a

Core Log	Depth in.	Description	Natural M.C. %	RQD %
		Underclay; abundance of slickensides.	9.3	0
10				
		Underclay; limestone nodules present, slickensides rare.	9.4	100
22				
		Underclay; limestone nodules present, abundance of slickensides at top and bottom of core.	9.8	50
32				
37		Core loss		
		Underclay; abundance of limestone nodules; few slickensides.	8.7	0
45				
		Underclay; limestone nodules present; bedding plane at 55 in.	9.3	100
55				
		Underclay; abundance of limestone nodules and slickensides	5.7	0
63				
		Underclay; fractures present	4.6	0
72				
75		Core loss		
		Silty grey limestone	4.1	100
84				
		Hard claystone; limestone nodules present.	5.1	100
96				
		Hard claystone; no limestone nodules present	5.0	100
108				
		Underclay; hard at top with soft green base clay; No limestone nodules present. Slickensides at base.	10.0	65
116				

Scale 1" = 16"

Figure 1. Lithologic and geotechnical description for a typical core from the mine.

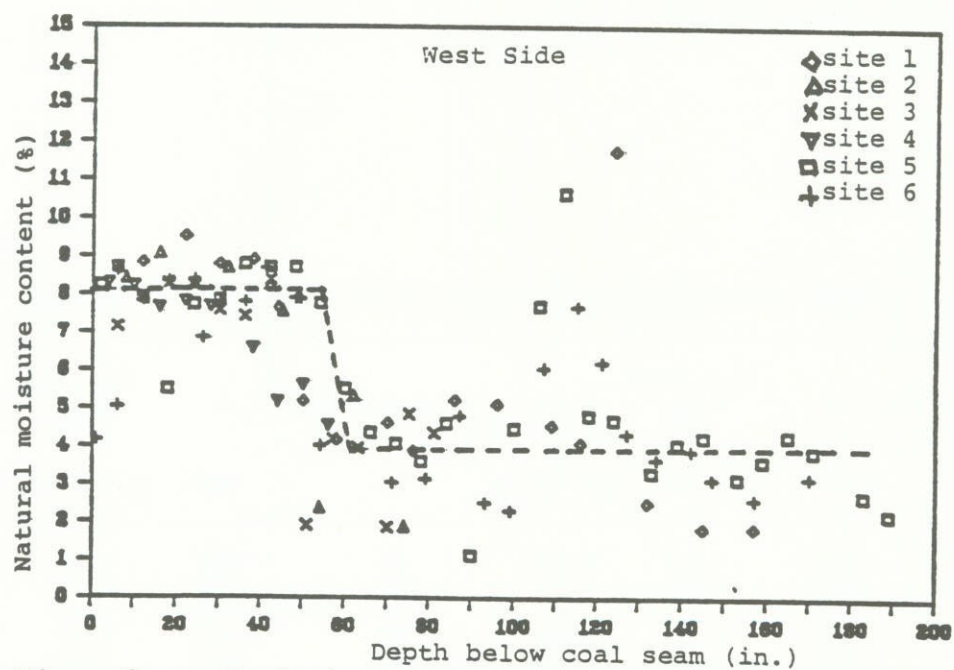
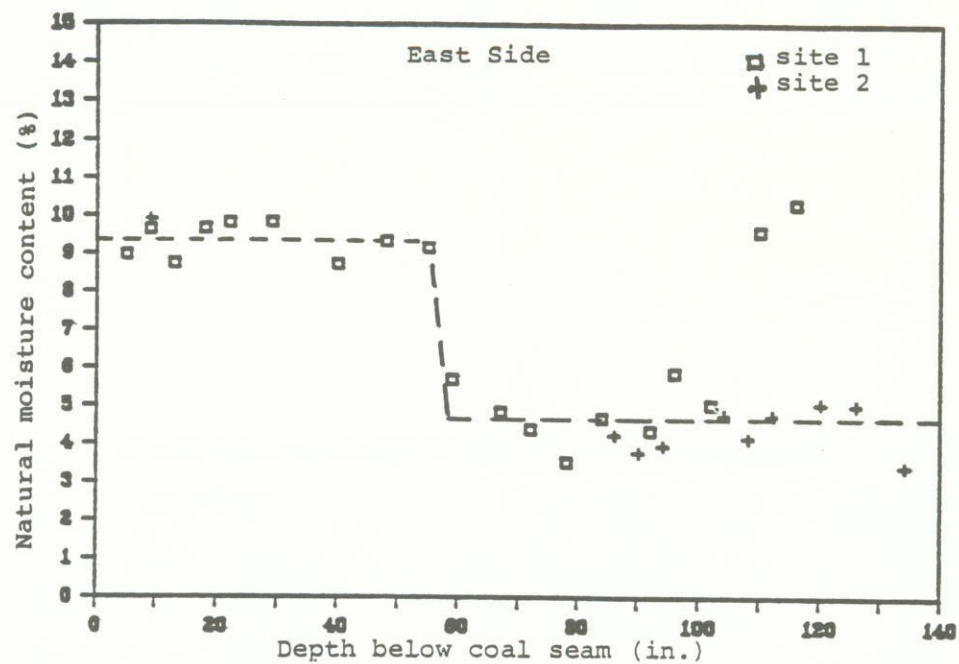


Figure 2. Variation of natural moisture content with depth below the coal seam.

Table 1. Particle Size Distribution Data for Immediate Floor Strata

Depth Below Coal Seam in.	Sand %	East Silt %	Side Clay Size %	Sand %	West Silt %	Side Clay Size %
0 - 55	2.7	40.7	56.6	2.7	47.3	50.0
> 55	6.0	57.8	36.2	3.2	53.4	43.4

Table 2. Engineering Index Properties of Immediate Floor Strata

Depth Below Coal Seam in.	M.C. %	East Side L.L. %	P.L. %	P.I. %	M.C. %	West Side L.L. %	P.L. %	P.I. %
0 - 55	9.38	42.80	24.34	18.46	7.66	46.94	19.55	26.22
> 55	4.57	26.61	14.45	12.16	3.97	37.71	17.51	20.79

L.L. = Liquid Limit, P.L. = Plastic Limit, P.I. = Plasticity Index,
M.C. = Moisture Content

Table 3. Static Strength-Deformation Properties of Immediate Floor Strata

Depth Below Coal Seam in.	Co (psi)	West Side E _{50%} (x10 ⁵ psi)	E _{100%} (x10 ⁵ psi)	Depth Below Coal Seam in.	Co (psi)	East Side E _{50%} (x10 ⁵ psi)	E _{100%} (x10 ⁵ psi)	μ
50	3487	2.92	0.88	90	2477	2.20	1.45	0.43
90	3033	3.73	3.76	100	2103	3.46	2.83	0.35
147	3279	4.62	1.27	106	2626	1.55	3.40	0.20
173-183	743	2.30	0.03	120	2336	1.29	1.65	0.30
208	2890	3.50	4.43	126	3716	1.77	1.94	0.11
				134	5030	6.20	9.40	0.38

Co = Unconfined Compressive Strength.

μ = Poisson's Ratio.

E_{50%} = Secant Modulus of Elasticity at 50% Co

E_{100%} = Secant Modulus of Elasticity at 100% Co

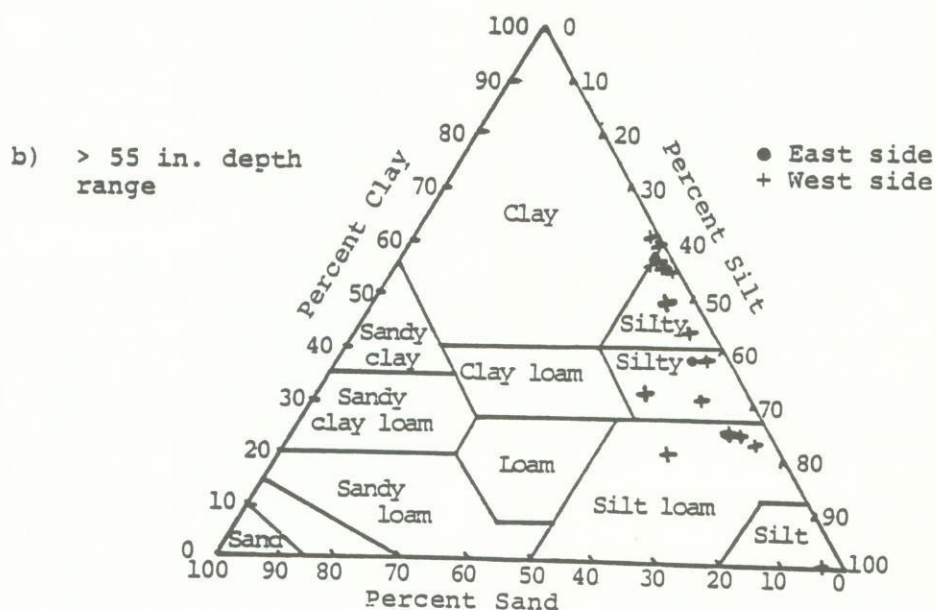
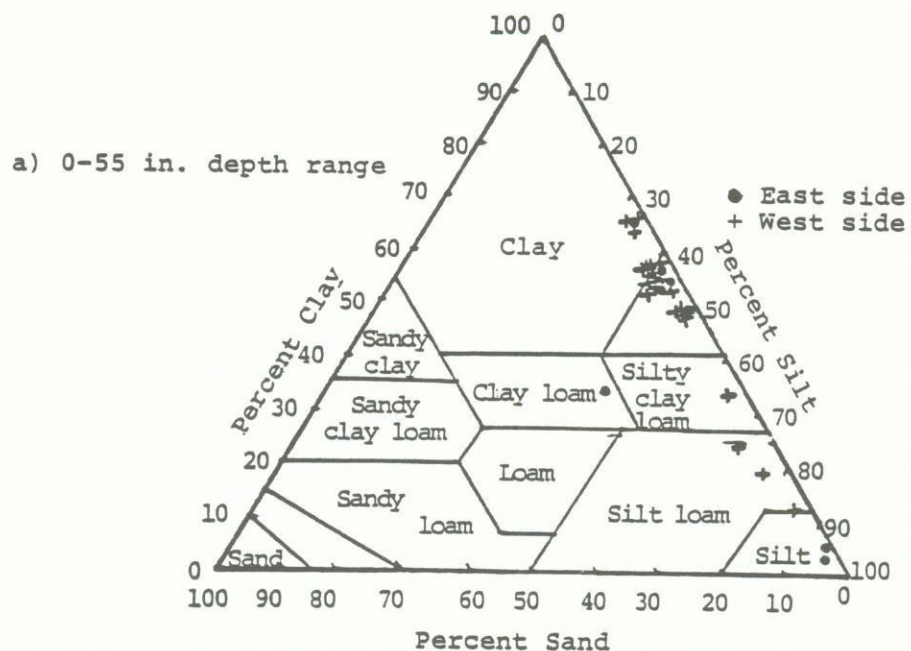


Figure 3. A plot of the particle size distribution data on the triangular soil classification chart.

resolution of 5 micro-inches. Load and deformation were monitored as a function of time on strip chart recorders as well as on a 32-channel data logger. An internal clock in the data logger was set to zero at the beginning of the test to establish a time coordinate. Data recording intervals could be varied from 1 sec. to 30 min in the data logger. A digital voltmeter was also used in parallel with the data acquisition system to obtain readings manually at regular intervals of time to provide a check on the recorded data. All experiments were carried out in a temperature (78 F) and humidity (65%) controlled environment. A maximum variation of 1 F in temperature and 5% in relative humidity was observed during the course of any test. Total duration of the test on individual samples varied from 18.3 to 404.5 hours. Experimental techniques are described in more detail elsewhere (12).

Strain-Time Behavior

Figures 4 and 5 show typical axial and lateral strain-time curves for a sample. The curves show instantaneous strain, a primary or transient creep indicating the rise in strain values with time at a decreasing rate, and a secondary or steady-state creep showing more or less constant increase in strain with time. Tertiary creep was also observed in a few specimens near the failure point.

The strain-time data were analysed in terms of Burger models with one ($n = 1$) and two ($n = 2$) Kelvin-Voigt units (Figure 6) for each increment of stress using a computer program developed by Hardy and Wang (13). The total strain for a stress increment, given by the generalized Burger model, can be expressed by the following equation.

$$\epsilon(t) = -\frac{\sigma_0}{E_{n+1}} + \sigma_0 \sum_{i=1}^n \frac{1}{E_i} \left(1 - e^{-\frac{E_i t}{N_i}}\right) + \frac{\sigma_0 t}{N_{n+1}} \quad (1)$$

where,

$\epsilon(t)$ = total strain due to applied stress increment after time 't'

σ_0 = constant stress applied

t = time for which σ_0 has been applied

E_i, N_i = Elastic and Viscous parameters of the i th Kelvin-Voigt unit.

n = total number of Kelvin-Voigt units
 E_{n+1}, N_{n+1} = Elastic and viscous parameters of the Maxwell unit.

The first, second, and third terms on the righthand side of the equation represent instantaneous, primary, and secondary creep strains respectively.

Some pertinent comments based on data analysis are:

(1) Significant time-dependent strains are observed only at stress levels over 80% of the σ_0 value.

(2) The $n = 1$ Burger model appears to fit the data well in most cases with an error of less than 10% for all predicted values. The $n = 2$ Burger model did not improve the predictions significantly. Burger model parameters ($n = 1$) for time-dependent axial strain, for samples in the 0-55 in. depth range, are presented in Table 4. Similar data for samples below 55 in. depth range were generally found to be higher indicating overall smaller creep strains.

(3) The lateral creep strain curves also follow a similar pattern but the strain values are much lower. Burger model parameters ($n = 1$) for time-dependent lateral strains along two (x- and y-) directions for a sample are presented in Table 5. The model predicted the strains with less than 10% error. A few sudden jumps were observed during exponential creep phase which are thought to be due to slippage along slicken sides. There was great similarity in time-independent and time-dependent strains along the two horizontal directions indicating transversely isotropic behavior.

(4) Burger model parameters are stress dependent. All parameters generally decrease just prior to failure of the sample indicating a significant increase in time-independent and time-dependent axial and lateral strains (Tables 4 and 5). Some samples show a relatively steady decrease in parameter values with increasing stress while others show an overall increasing trend until just prior to failure. There are significant random variations from the overall trend. These variations may be due to crushing of asperities, micro-level stick-slip phenomenon and/or movements along different slickensided surfaces. Slippage along slickensided surfaces may be the cause of sudden increase in lateral time-dependent primary creep strain exhibited by several samples.

(5) In spite of the overall stress-dependency of Burger model parameters, it is found that the effect of stress is

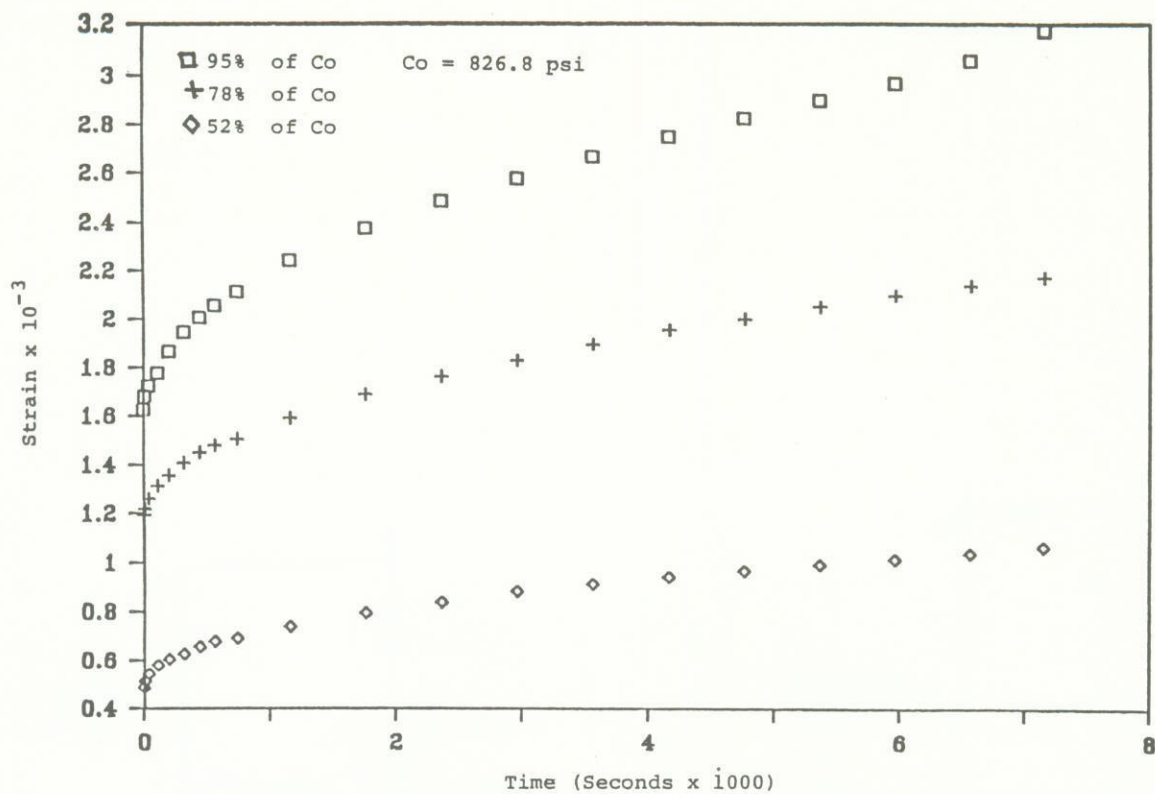


Figure 4. Time-dependent Axial Strain at Different Stress Levels

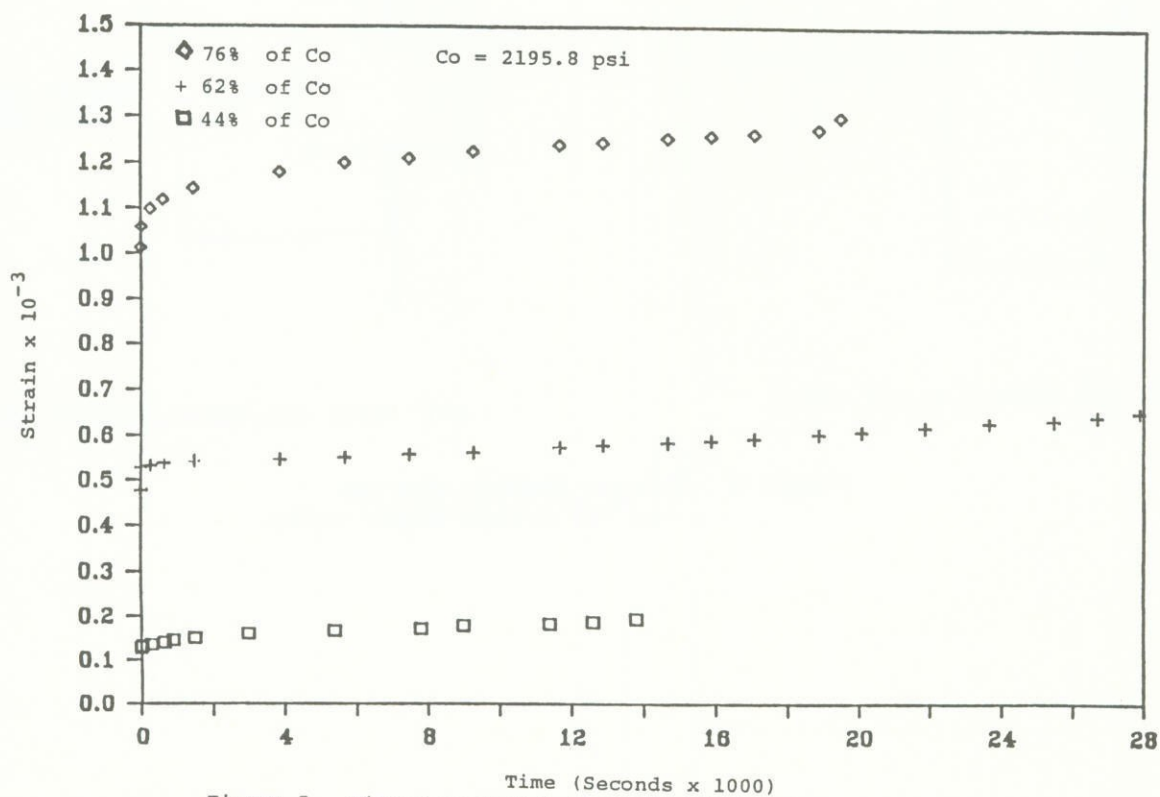
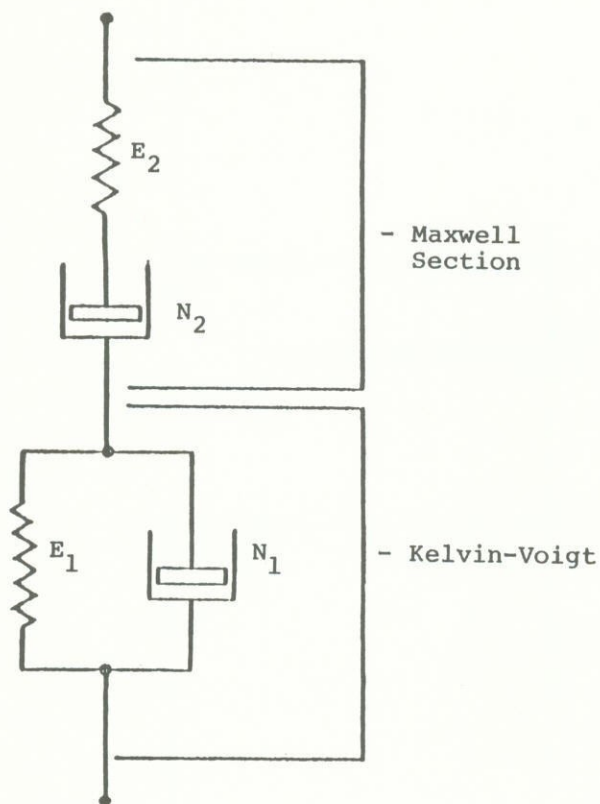
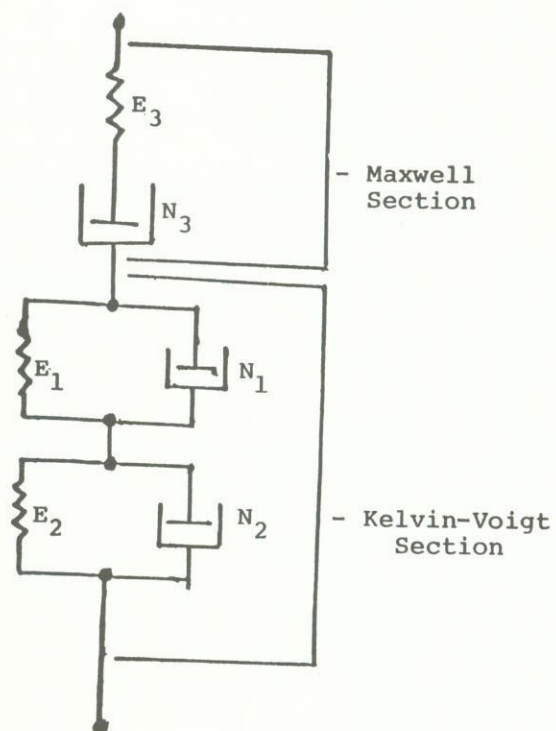


Figure 5. Time-dependent Lateral Strain at Different Stress Levels



(A) With one Kelvin-Voigt unit.



(B) With two Kelvin-Voigt units.

Figure 6 Burger Models With One and Two Kelvin-Voigt Units.

Table 4. Effect of Stress on Burger's Model Parameters For Axial Strain For Samples in the 0-55 in. Depth Range

Sample No.	% of Failure Stress	E1 $\times 10^6$ psi	N1 $\times 10^9$ psi-sec	E2 $\times 10^5$ psi	N2 $\times 10^{10}$ psi-sec	N1/E1 sec	Sample No.	E1 $\times 10^6$ psi	N1 $\times 10^9$ psi-sec	E2 $\times 10^5$ psi	N2 $\times 10^{10}$ psi-sec	-N1/E1 sec
1E1	40	0.55	0.24	1.77	2.82	441	6W1	1.18	0.43	4.11	2.54	360
	50	0.29	0.12	1.98	6.47	436		0.91	0.44	4.79	4.00	478
	60	9.14	3.00	1.50	6.25	328		1.35	0.48	4.77	3.18	357
	75	0.95	0.47	2.00	10.97	495		1.02	0.54	5.65	3.51	528
	90	0.36	0.14	1.65	11.66	433		0.56	0.124	6.45	2.21	423
	100	0.18	0.05	0.86	18.72	278		9.37	0.07	4.86	0.49	8
3E1	40	0.50	0.01	7.23	4.33	12	5W1	1.14	0.63	5.03	3.43	552
	50	1.08	0.92	3.22	9.97	849		1.05	0.84	6.15	4.99	798
	60	0.32	0.34	2.19	15.77	1085		1.39	1.04	5.56	6.18	751
	70	0.47	0.27	1.28	19.67	578		1.34	0.80	6.34	8.60	594
	90	0.74	0.37	5.17	50.59	500		0.70	1.94	4.21	16.56	2793
	100	0.53	1.15	6.95	23.09	2163		0.65	0.04	7.54	5.83	656

Table 5. Burger model parameters for time-dependent lateral strain.

X-direction

Increment No.	Total Stress psi	E1 $\times 10^6$ psi	N1 $\times 10^9$ psi-sec	E2 $\times 10^5$ psi	N2 $\times 10^{10}$ psi-sec	N1/E1 sec
1	353.1	1.38	0.61	10.63	8.20	4481
2	396.2	1.40	0.57	15.95	15.05	4082
3	499.6	1.21	0.88	7.97	10.13	7273
4	628.7	28.53	15.01	12.75	32.17	5264
5	749.3	2.35	1.02	9.81	98.16	4335
6	26.8	3.68	1.52	4.73	27.47	4126

Y-direction

Increment No.	Total Stress psi	E1 $\times 10^6$ psi	N1 $\times 10^9$ psi-sec	E2 $\times 10^5$ psi	N2 $\times 10^{10}$ psi-sec	N1/E1 sec
1	353.1	1.24	0.54	9.58	9.90	437
2	396.2	1.11	0.40	16.11	21.66	365
3	499.6	1.25	0.88	18.27	12.19	706
4	628.7	12.15	6.58	13.52	26.08	542
5	749.3	26.71	12.12	10.00	45.38	453
6	826.8	10.00	4.21	9.30	18.02	421

relatively small in the range 40-80% of the failure stress, and a linear visco-elastic model can be used to represent the behavior of immediate floor strata in this range. Such a model should be acceptable for the design of partial extraction mining systems having extraction ratios in the range of 40 to 60%. Table 6 gives mean values of parameters E_1 , N_1 , E_2 , and N_2 for such a prediction model for east and west panels of the mine with 65% confidence level (one standard deviation). The equation of the model is given by:

$$\epsilon(t) = -\frac{\sigma_0}{E_2} + -\frac{\sigma_0}{E_1} \left(1 - e^{-\frac{E_1 t}{N_1}}\right) + \frac{\sigma_0 t}{N_2} \quad (2)$$

(6) In the east side of the mine, the model parameters for over 55 in. depth range samples are significantly higher as compared to those in the 0-55 in. range (Table 6). Thus, most of the time-dependent strain in this side of the mine would be contributed by the upper 55 in. of the floor strata. The model parameters for both the depth ranges are very similar in the west side (Table 6). Thus, one may expect the immediate floor strata to a much larger depth involved in the floor heave process in the west side. This is actually observed in the field from inclinometer and Sondex measurements (14). Floor strata upto a depth of about 10 ft is involved in the heave process on the west side as compared to only about 5 ft on the east side.

(7) Table 7 presents the mean values of a Burger Model Parameters ($n = 1$) for lateral time-dependent strains in x- and y- directions for 40 to 80% of the C_0 value.

Parameter values for lateral strains are considerably larger than those for axial strain. Thus considerably smaller amount of time-dependent lateral strain may be expected. The ratios of E_2 values in the axial direction to the E_2 values in the lateral direction are approximately in the range of 0.4 to 0.5 which represents the Poisson's ratio.

The parameter values for sample 3E₁ are significantly different than sample 1E₁ except for the parameter N_1 . Additional data on lateral strains are required to develop the time-dependent lateral strain

model with confidence.

(8) Table 8 presents the constant creep strain rates in the axial direction. These rates are in the range of 10^{-9} /s in the east side and 10^{-8} /s in the west side. These compare reasonably well with those found by Afrouz and Harvey (1) for air-dried underclay samples from a South Wales mine (1.5×10^{-9} /s and 2.7×10^{-9} /s at stress levels of 372.6 psi and 745.3 psi respectively). The constant creep rate for samples from the west side of the mine is about 10 times higher than for samples from the east side. This would be expected based on a high liquid limit and plasticity index values for the west side. The constant creep rate generally becomes higher as the specimen approaches failure stress. The unusually high values for the first load increment (40% of failure stress) may be due to closing of cracks and discontinuities inherently present in the sample. Steady creep strain rate generally increases with increasing stress level which is typically observed in rocks. Significant variations from the above general trend are observed and may be due to stick-slip behavior of slickensided claystone.

FIELD STUDIES OF TIME-DEPENDENT DEFORMATION OF IMMEDIATE FLOOR STRATA

Roof to floor movement (convergence) studies along the center of roadways have been conducted at the mine by the coal company and Chugh et al. (14). Data indicate very little roof movements; thus all movements are related to movement of immediate floor strata. Typically, convergence rates of 3.5 in. per month are recorded immediately after a mine opening is created. These rates decrease exponentially to a steady state rate of about 0.2 in./month over a 300-day period. Thus, a viscoelastic model may be appropriate for representing the floor heave process.

Analysis of floor heave data in terms of a Burger model ($n = 1$) presented some problems. Since measurements were not initiated immediately after openings were created, the entire instantaneous elastic deformation and part of the primary creep deformation were lost. Therefore, the E_2 parameter could not be directly calculated from the floor heave data. This was, however, determined from the in-situ plate bearing tests carried out in the mine by Chugh (10). In this analysis, floor heave deformation data was reduced to strain data by assuming that the deformation was contributed only by the top 55 in. of the floor strata. This is a reasonable assumption based on the inclinometer and

Table 6. Mean Values of Burger Model Parameters for Axial Strain for 40% to 80% of Uniaxial Compressive Strength

Depth Range: 0-55 in. below the coal seam

Parameter	East Section	West Section
E1	$(0.59 \pm 0.28) \times 10^6$ psi	$(1.17 \pm 0.16) \times 10^6$ psi
N1	$(0.39 \pm 0.26) \times 10^9$ psi-sec	$(0.65 \pm 0.21) \times 10^9$ psi-sec
E2	$(1.99 \pm 0.58) \times 10^5$ psi	$(5.30 \pm 0.71) \times 10^5$ psi
N2	$(9.53 \pm 5.43) \times 10^{10}$ psi-sec	$(4.55 \pm 1.86) \times 10^{10}$ psi-sec

Depth Range: Over 55 in. below the coal seam

Parameter	East Section	West Section
E1	$(3.35 \pm 1.67) \times 10^6$ psi	$(1.83 \pm 0.84) \times 10^6$ psi
N1	$(4.03 \pm 3.37) \times 10^9$ psi-sec	$(1.05 \pm 0.27) \times 10^9$ psi-sec
E2	$(6.95 \pm 3.94) \times 10^5$ psi	$(7.60 \pm 2.25) \times 10^5$ psi
N2	$(62.3 \pm 26.2) \times 10^{10}$ psi-sec	$(2.35 \pm 1.50) \times 10^{10}$ psi-sec

Table 7. Mean Values of Burger Model Parameters for Lateral Strain for 40-80% of Unconfined Compressive Strength

Sample 1E1

Parameter	X-Direction	Y-Direction
E1	$(1.33 \pm 0.09) \times 10^6$ psi	$(1.19 \pm 0.06) \times 10^6$ psi
N1	$(0.69 \pm 0.14) \times 10^9$ psi-sec	$(0.61 \pm 0.20) \times 10^9$ psi-sec
E2	$(11.8 \pm 2.92) \times 10^5$ psi	$(14.4 \pm 3.24) \times 10^5$ psi
N2	$(16.4 \pm 9.45) \times 10^{10}$ psi-sec	$(17.5 \pm 6.65) \times 10^{10}$ psi-sec

Sample 3E1

Parameter	X-Direction	Y-Direction
E1	$(3.76 \pm 2.29) \times 10^6$ psi	Could not be obtained
N1	$(0.44 \pm 0.58) \times 10^9$ psi-sec	
E2	$(5.86 \pm 3.01) \times 10^5$ psi	
N2	$(31.5 \pm 11.9) \times 10^{10}$ psi-sec	

Table 8. Constant Creep Rates at Different Percentages of Failure Stress

<u>East Side</u>				
Sample No.	Failure Stress (psi)	Stress Level (psi)	Percentage of Failure Stress (%)	Constant Creep Rate (x10 ⁻⁹ /s)
1E1	826.8	330.7	40	11.70
		413.4	50	6.38
		496.1	60	7.93
		620.1	75	5.65
		744.1	90	6.38
		826.8	100	4.41
3E1	2195.8	878.3	40	20.20
		1097.9	50	11.00
		1317.5	60	8.35
		1646.8	75	8.37
		1976.2	90	3.90
		2195.2	100	9.51
<u>West Side</u>				
Sample No.	Failure Stress (psi)	Stress Level (psi)	Percentage of Failure Stress (%)	Constant Creep Rate (x10 ⁻⁸ /s)
5W1	3532.1	1412.8	40	4.12
		1766.1	50	3.54
		2119.3	60	3.43
		2649.1	75	3.08
		3178.9	90	1.92
		3532.1	100	6.05
6W1	3359.2	1343.6	40	5.29
		1679.6	50	4.20
		2015.5	60	6.33
		2519.4	75	7.17
		3023.2	90	13.68
		3359.2	100	68.55

Table 9. Field Parameters of Burger Model

<u>East Side</u>	
E2	$= (3.99+2.18) \times 10^4$ psi
N2	$= (2.55+1.26) \times 10^{11}$ psi
E1	= could not be determined.
N1	= could not be determined.
<u>West Side</u>	
E2	$= (3.99+2.18) \times 10^4$ psi
N2	$= (5.00+1.52) \times 10^{11}$ psi-sec
E1	$= (1.03+0.68) \times 10^4$ psi
N1	$= (3.53+1.47) \times 10^{11}$ psi-sec

sondex studies (14). The field time-dependent deformation data was analyzed for $n = 1$ Burger model. Table 9 gives the parameters of this model. These parameters are considered representative for a stress level of 80-90% of the ultimate bearing capacity (65% confidence level). A comparison of laboratory and field parameters shows that N_2 values based on the field data are 2 to 10 times higher than those based on laboratory data. Based on these results, it may be concluded that steady-state creep may not have been achieved in many cases in laboratory studies. E_1 values for the laboratory constitutive model are 50 to 100 times larger than those for the field data while N_1 values are about 5 times lower. Since convergence measurements were not initiated immediately after the openings were made, part of the exponential creep was not recorded and therefore the values of parameters E_1 and N_1 may not be truly representative.

CONCLUDING REMARKS

Engineering index properties and laboratory and field time-dependent deformation behavior of immediate weak floor strata from an Illinois coal mine is presented in the paper. An attempt is also made to correlate Burger model parameters determined from the time-dependent deformations observed in the laboratory and in the field. This should permit estimation of floor heave as a function of time from laboratory creep tests.

The results show promise in this direction but indicate need for additional similar studies before confidence can be developed in determined parameters. Large scatter in data obtained from field is primarily due to variability in thickness and engineering properties of immediate floor strata and inability to determine stresses acting on pillars. Similar studies are presently underway for another mine located about 70 miles away. The developed viscoelastic model parameters will be used in a finite element model to estimate pillar punching, floor heave, and surface subsidence as a function of time.

ACKNOWLEDGEMENTS

This research has been primarily supported by the Department of the Interior's Mineral Institute's program administered by the Bureau of Mines under allottment grant number G1125151.

Additional support was provided by the coal company. Financial support from the above two groups is highly appreciated.

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THE SIGNIFICANCE OF SPECIMEN STIFFNESS AND POST YIELD CHARACTERISTICS ON PASSIVE ROOF SUPPORT DESIGN

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ABSTRACT

The intent of this paper is to make mine operators and engineers aware of parameters which affect the loading behavior of passive roof support systems. The strength of a passive roof support element alone is not a meaningful measure of support performance.

Passive roof support elements and the surrounding strata act as a system, responding to changes in the physical mine environment due to the extraction of coal and associated redistribution of stresses. The loading of a passive roof support element is a function of the stiffness of the roof support structure; a stiffer structure will react a higher load to a converging mine roof than will a more flexible structure. If the stiffness of the support structure is not properly designed, excessive loading will result due to the displacement of the mine roof, causing premature failure of the support. Ideally, the yield strength of a passive support needs to be no greater than that required to support the dead weight of the immediate mine roof, providing the post yield characteristics of the support are compatible with the irresistible convergence of the overburden strata and the support system continues to provide this resistance after reaching its yield load. Idealized design considerations for passive roof support systems in terms of load-displacement characteristics are discussed. The stiffness and post yield characteristics of wood and concrete cribbing, as determined from controlled tests in the Bureau's Mine Roof Simulator, are compared. Some preliminary recommendations for passive roof support utilization are also provided.

INTRODUCTION

A passive roof support device is a structure which is incapable of exerting a force, other than to react to an externally applied load or displacement. Passive roof supports, such as posts and cribbing, are used extensively in underground mining applications, typically as permanent and expendable roof support systems, and often in areas of poor roof conditions. Since passive roof supports are relatively simplistic structures, they have received less attention in terms of engineering design than the more sophisticated active mechanical and hydraulic roof support systems. Most research which has been conducted on passive roof support elements has been limited to evaluation of ultimate load strength.

It is important to understand in the evaluation of roof support systems, both passive and active support structures, that the roof support elements and the surrounding strata act as a system, responding to changes in the physical mine environment due to the extraction of coal and associated redistribution of stresses. The strength (load capacity) of a passive roof support element alone is not a meaningful measure of support performance. The loading of a passive roof support element can be significantly dependent upon the stiffness of the support element, as a stiffer structure will react a higher load to a converging mine roof than will more flexible structure. A high yield strength does not necessarily insure optimum or even adequate roof support in all applications. A stiff support structure with a high yield load can fail prematurely due to excessive loading, if utilized in areas of inevitable strata convergence from

subsidence (displacement) of the overburden.

Equally important to the loading characteristics of a passive roof support system is its behavior after reaching yield load. Passive supports which lose their load carrying capability quickly after reaching yield load, fail to provide adequate roof control, creating an unsafe working condition.

The physical properties of wood and concrete have been well researched and documented. Stiffness and post yield characteristics of wood and concrete have been determined from tests with laboratory-sized specimens. However, it is unlikely that these data can be extrapolated to crib structures. In addition to the size effects, crib supports are constructed of several unit blocks and the effect of these discontinuities is not known. These circumstances necessitate full scale testing of crib supports. Such testing requires a large, relatively stiff, load frame capable of controlled displacement loading. Tests of full scale passive roof support elements are conducted in the Bureau's Mine Roof Simulator.

These issues are discussed in detail in the paper and design considerations are evaluated in terms of stiffness, yield load, and post yield characteristics of passive roof support systems.

STIFFNESS OF A STRUCTURE

Stiffness is a structural engineering term defined as the force required to produce a unit deflection in a structure. Mathematically, it can be expressed in the familiar Hooke's Law form of equation 1, where the resulting force is a linear function of the applied displacement.

$$F = K \cdot x \quad (1)$$

where F = Force (Load)
 x = Displacement
 K = Stiffness

Figure 1 illustrates the significance of stiffness in terms of loading of a structure. Shown are load versus displacement plots for two hypothetical structures of different

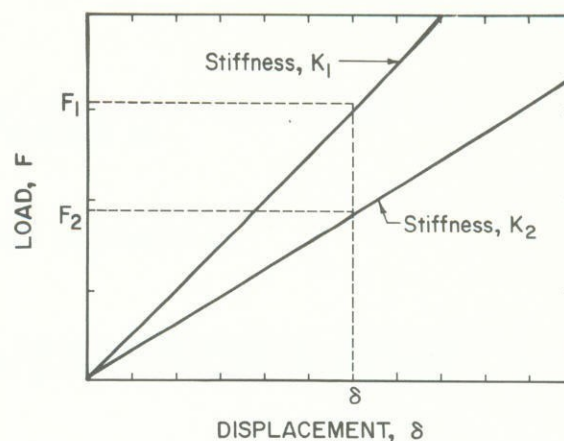


FIGURE 1. - Significance of specimen stiffness on load reaction.

stiffnesses, K_1 , and K_2 , where $K_2 < K_1$, as determined from the slope of the load versus displacement plot. As can be seen from the figure, if both structures were displaced an equal amount δ , the resultant load on the stiffer structure would be 1.5 times that of the softer structure. Therefore, the degree to which a passive roof support system is loaded by a converging mine roof is dependent upon the stiffness of the roof support structure.

THE CONCEPT OF DISPLACEMENT CONTROL

The previous discussion, which defined loading of a roof support element to be a function of the stiffness of the structure, implies displacement control roof behavior. The concept of displacement control is mechanically illustrated in figure 2. Figure 2A shows a spring of stiffness K_1 forced to undergo a displacement δ_1 by force F_1 . From Hooke's Law (equation 1), this force is calculated as $F_1 = K_1\delta_1$. If the stiffness of the spring is changed to K_2 , as illustrated in figure 2B such that $K_2 < K_1$, the force required to displace the spring the same amount ($\delta_2 = \delta_1$) is less ($F_2 < F_1$), since $\delta_2 = \delta_1$ and $K_2 < K_1$.

The important thing to note about displacement control is that it is displacement which is the control (independent) variable and that force is the dependent variable. In the mechanical model illustrated in figure 2, the spring is analogous to the passive roof support element, and the spring load (F_1) is analogous to

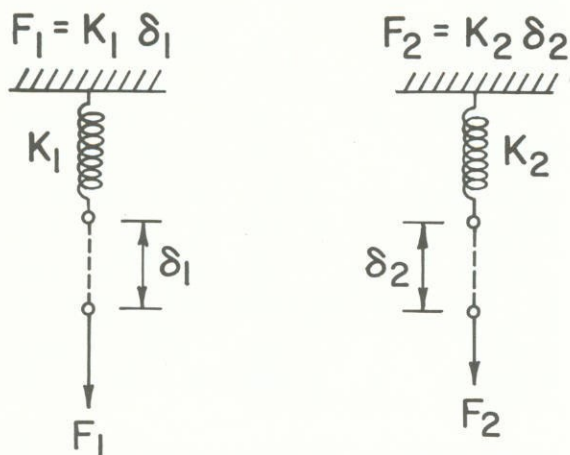


FIGURE 2. - Displacement controlled loading.

support loading or support resistance. As illustrated, in displacement loading the structure is forced to undergo a displacement independent of the support resistance, and the loading of the support element is dependent upon the stiffness of the structure and linearly proportional to the applied displacement.

In contrast to displacement control is force control, which is illustrated in figure 3. A weight (W) is suspended from two springs, one of stiffness K_1 and the other of stiffness K_2 with $K_2 < K_1$. In displacement control, the displacements remain constant regardless of the force with the force being a function of the stiffness of the structure. Whereas, in force control, the force remains constant being equal to the weight (W) and the displacement is a function of the stiffness of the structure.

It is generally accepted that the primary function of roof support systems in underground mining applications is to support the immediate roof which is left unstable by the mining operation. In the worst case, this unstable roof will behave as a dead weight, which must be supported by the roof support system. This would be an example of force control roof behavior. In reality, the immediate roof, despite being fractured and unstable, retains

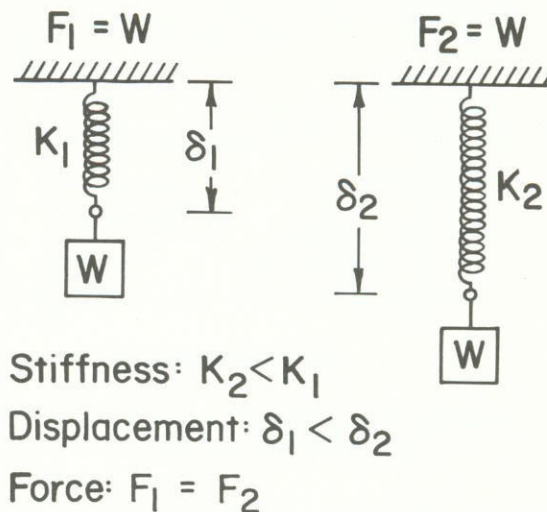


FIGURE 3. - Force controlled loading.

sufficient integrity to act as a beam subject to displacements due to bending, and therefore has properties of displacement control. In addition to the action of the immediate (unstable) strata, the subsidence of the overburden is critical to roof behavior. The subsidence action of the overburden is an example of displacement control roof behavior, since convergence (displacement) of this large mass of strata is virtually inevitable and unaffected by the resistance of roof support systems in the mine. The extent to which the displacement behavior of the overburden translates to localized convergence in mine openings is unknown, but it is safe to assume that there is some displacement-type roof behavior occurring.

IDEALIZED PASSIVE ROOF SUPPORT

The idealized passive roof support should be initially stiff or rapid load bearing, but ultimately yielding so as to accommodate the irresistible convergence forced upon it by the weight of the overburden. This behavior can be characterized by analysis of the load-displacement curve of an idealized passive roof support shown in figure 4. The desirable characteristics are interpreted from the figure as follows:

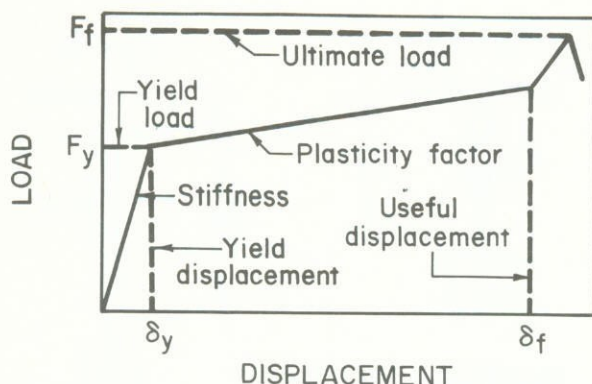


FIGURE 4. - Idealized passive roof support design.

1. Stiff Specimen During Loading - Generally, it is desirable to have the roof support accept load as fast as possible to prevent bed separation of the immediate strata, which could lead to deteriorating roof conditions. However, as indicated earlier, the stiffness must be compatible with the displacement behavior of the roof strata and post yield characteristics of the support to prevent unnecessary and excessive loading.
2. Predictable and Compatible Yield Load - Structurally, yield load is defined as the load at which the structure no longer behaves elastically and any further deflection will cause permanent deformation. While yield load can be construed as a point of failure, it is not necessarily ultimate failure in that, as in the case of the idealized support, the support will continue to provide resistance beyond its yield load. The desired magnitude of the yield load is dependent upon the conditions in which the support is employed and the post yield load characteristics of the support. Ideally, the yield strength of a passive roof support needs to be no greater than that required to support the dead weight (in force control) of the immediate mine roof, providing the post yield characteristics of the support are compatible with the irresistible convergence of the overburden strata (in displacement control). Having a predictable yield load displacement is probably as important as the magnitude of the yield load.

3. Plastic Behavior After Yield - The idealized passive support would exhibit plastic behavior after reaching yield load, indicating the support will continue to provide resistance despite being permanently deformed under continued deflection. This behavior is critical to displacement control roof behavior, since convergence of the mine roof is likely to continue after the support reaches its ultimate load. Ideally, the slope of the load/displacement curve in this portion should be positive or zero, as a negative slope would indicate a reduction in load carrying capability.

4. Large Useful Convergence Range - An idealized roof support would continue to provide support resistance indefinitely with post yield displacement. A more practical displacement range might be 5-10 in, but field studies are needed to substantiate this value.

Realistically, there are tradeoffs to be made among the desirable characteristics of passive roof supports previously described. Generally, the stiffer a structure is the less energy it can absorb, resulting in brittle failures (poor yield characteristics) for stiffer specimens. While a mine operator may not be able to design a support meeting these ideal criteria, he should recognize their impact on support performance and select a support that is most compatible with the conditions in his mine. To assist mine operators in this selection, the Bureau is determining the behavior of various passive roof support materials and designs. The Bureau is also conducting research to develop passive supports which more closely behave like the ideal support described above.

COMPARISON OF WOOD AND CONCRETE CRIB SUPPORT BEHAVIOR

The behavior of wood, nonreinforced concrete, and steel fiber reinforced (SFR) concrete cribs to applied displacement tests conducted in the Bureau's Mine Roof Simulator (fig. 5) are shown in figure 6. The results clearly indicate that wood and concrete are not structurally alike as they provide significantly different support behavior.

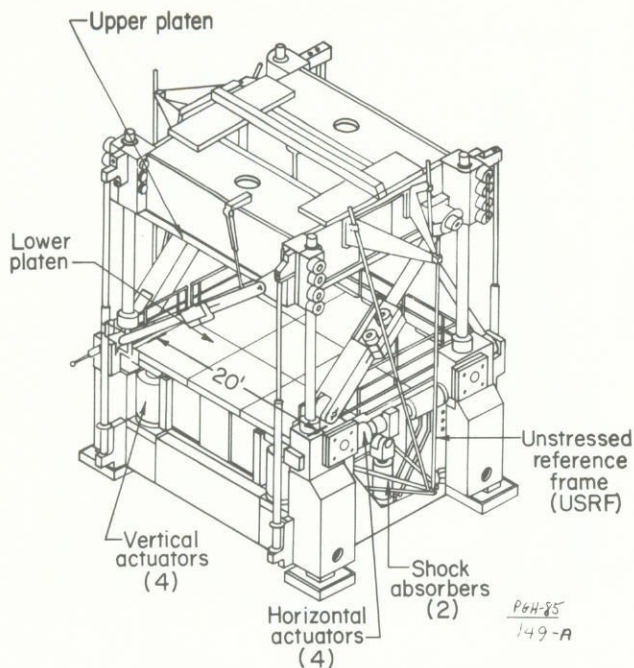


FIGURE 5. - Mine roof simulator.

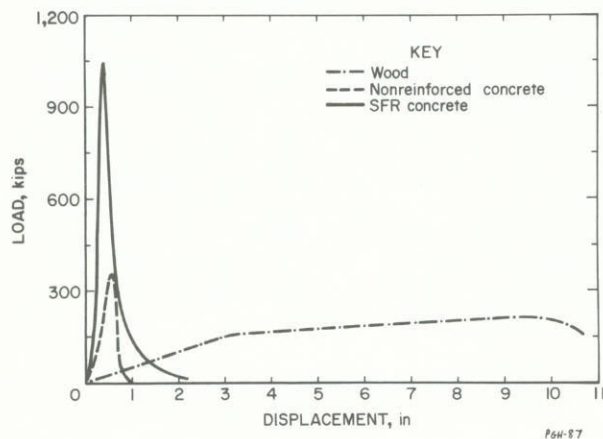


FIGURE 6. - Load-displacement characteristics.

One of the most striking differences between wood and concrete cribs is the significantly higher yield load of concrete compared to wood. The compressive strength of concrete, approximately 4,000 psi, is about eight times that of wood, resulting in a higher yield load. While there is considerable variability in wood samples, open wood cribs with a contact area between crib blocks of 144 in² can be expected to yield at

about 150-175 kips, whereas SFR concrete cribs of open configuration with a contact area of 256 in² exhibited a yield strength of about 1,050 kips. Solid SFR concrete cribs were shown to exhibit an ultimate strength of nearly 2,000 kips. The nonreinforced concrete exhibited less strength, despite similar compressive strength to the SFR concrete crib, since the failure is due to the lower tensile strength of the nonreinforced concrete crib.

Another significant difference between wood and concrete is their stiffnesses, which as previously indicated, determines the loading rate of support structures operating under displacement control. As illustrated in figure 7, SFR concrete is much stiffer than wood. Typical stiffness values for these materials in open and solid crib configurations are indicated in table 1. These results indicate that a solid SFR concrete crib would experience a load of approximately two times that of open SFR concrete crib and 18 times that of wood crib for the same displacement prior to reaching the yield strength of the crib. The displacement at which the yield strength of the support is reached is also a critical measure of performance. The stiff concrete supports, despite having a much greater yield strength, reach yield load at much smaller displacement than crib supports constructed of wood. As seen from table 2, concrete cribs, both reinforced and nonreinforced, fail at less than 0.5 in. of displacement, whereas wood can displace nearly 3 in. before reaching yield load. Therefore, if the convergence of the roof is irresistible (displacement control behavior), the high yield

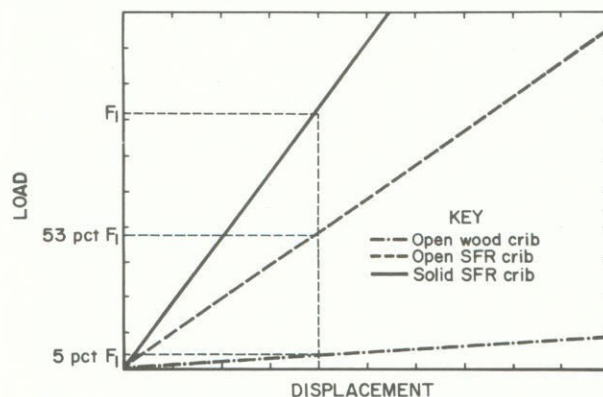


FIGURE 7. - Stiffness comparison.

TABLE 1. - Stiffness comparison.

Crib structure	Stiffness (kips/in)
Open wood.....	500
Open SFR concrete...	4,800
Solid SFR concrete..	9,000

TABLE 2. - Support deflection at yield load.

Crib structure	Yield strength (kips)	Displacement (in)
Wood.....	150	3.0
Nonreinforced concrete.....	320	.45
SFR concrete...	1,050	.35

strength of the concrete crib will be of no advantage if the convergence of the roof exceeds 0.5 in. (It should be noted that the concrete cribs were tested with 3/4-in of plywood on the top of the crib. A higher wood to concrete ratio, as recommended by some manufacturers, will change the wood displacement characteristics of the concrete.)

Figure 6 compares the load-displacement characteristics of wood, nonreinforced concrete, and SFR concrete illustrating the differences in stiffness, yield strength, and yield displacement parameters previously discussed. Figure 6 also vividly illustrates the differences in post yield characteristics of these passive roof supports showing the advantages of wood structures as a yieldable support compared to concrete supports. As can be seen from the figure, wood cribbing exhibits a plastic behavior after reaching yield load, providing continued support resistance for over 10 in of displacement. In contrast, concrete structures exhibit more of a brittle failure, losing virtually all load carrying capacity soon after reaching yield load. While some improvement has been made in the post yield characteristics of concrete through the use of steel fiber reinforcement by allowing more of a ductile failure (fig. 8), concrete supports, including SFR concrete, remain stiff structures with relatively poor post yield characteristics for roof support applications where convergence is inevitable.

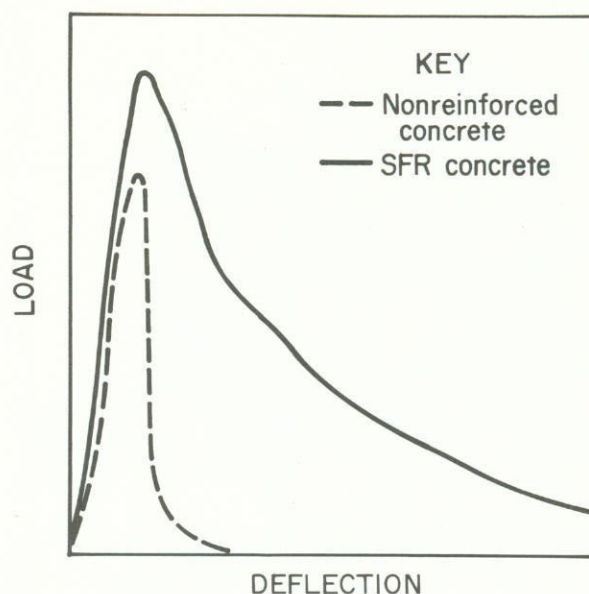


FIGURE 8. - Behavior of fiber-reinforced concrete.

A final measure of passive roof support performance which encompasses stiffness, yield strength, and post yield characteristics is the energy absorption capability of the support, computed as the area under the load/displacement curve by mathematical integration of the load function. As seen from table 3, wood supports exhibit an energy absorption capability of over twice that of SFR supports despite the much lower yield strength, due to the ability of wood to provide support resistance over a wide displacement range.

TABLE 3. - Energy absorption comparison (kip/in)

Crib structure	Pre yield	Post yield	Total
Open wood.....	1,642	500+	2,000+
Open SFR concrete...	140	383	522
Solid SFR concrete..	314	744	1,047

CONCLUSIONS

The intent of this paper is to make the reader aware of parameters which affect the loading behavior of passive roof support systems. The concepts of force control and displacement control were introduced to demonstrate the

need to evaluate the loading mechanism in roof behavior as well as the response of the roof support structure. It was established that evaluation criteria, other than yield strength, need to be examined when evaluating or selecting a passive roof support system. The greater load capacity of a high strength support, such as SFR concrete cribbing, does not necessarily make it the most effective roof support system for all applications. The response characteristics of wood and concrete are completely different. Wood being a visco-elastic material, exhibits low yield strength but excellent yield characteristics; whereas, concrete is a stiff, brittle material with high strength and poor yield characteristics. Despite the improvements made in concrete due to steel fiber reinforcement, concrete supports still are not considered to be yieldable supports and should not be used in applications of irresistible roof convergence. The idealized passive roof support should be initially stiff to be rapid load bearing with a yield strength sufficient to support the dead weight response of the immediate fractured strata and have sufficient post yield characteristics to be compatible with the displacement behavior of the overburden.

In conclusion, considerations must be given to parameters, other than strength, in the design and selection of a passive roof support system. Tradeoffs often exist between the desirable characteristics of a passive roof support system, and a design should be selected which is most compatible with the conditions in which the support is to be employed.

FRictional PROPERTIES OF ROCK APPLIED TO MINING EXCAVATIONS

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ABSTRACT

Results of direct shear tests on rock discontinuities and triaxial compression tests on intact rocks are presented and discussed.

The main parameters considered are: stress functions, angle of internal friction and cohesion variations.

Primary objective of this work is to determine relationships that adequately describe the rock behaviour for design of mining excavations, while still maintaining a certain degree of simplicity.

INTRODUCTION

The essence of stability analysis, applied to mining excavations, lies with the accurate estimation of the shear strength, internal friction and cohesion of the rock mass.

In surface and shallow underground excavations the stability of the mining structure tend to be mostly influenced by the presence of structural discontinuities, weight of the overburden, groundwater regime and the degree of weathering. Excavations at greater depths, however, tend to primarily be controlled by the rock mass response to the in situ stress variation in the vicinity of the openings, the tectonic stresses and state of fractured-zones surrounding the openings.

Under high stresses and in the absence of major weakness planes, the intact strength of rock in the immediate vicinity of an opening is much higher than the broken or heavily jointed rock masses. However, the fissured, fractured or jointed rock is capable of supporting considerable loads, especially in the presence of a lateral restraint. Therefore, an understanding

of the complete stress-strain characteristics of a confined rock mass have application in the mining field, especially where deep underground structures are developed in rocks which fail due to the induced ground stresses.

The purpose of this investigation is to present the laboratory results performed on discontinuities and intact sedimentary rocks. Use of a stiff-testing machine allowed the determination of complete stress-strain curves. It is believed that the above program allows for the determination of the frictional properties of the investigated rocks, for mining excavation design considerations.

SHEAR STRENGTH OF ROCK DISCONTINUITIES

Variety of models have been suggested to adequately predict the frictional behaviour of discontinuities in rock (1-15). The mere proposal of so many different models is a testimony to the underlying complexity in the determination of rock mass frictional behaviour. Patton's (2,3) pioneering work laid the foundations for most of the recent studies. The proposed bi-linear criterion (2) recognized the effects of surface roughness and dimensions on the frictional properties of rock discontinuities. While mechanically correct, his model was limited due to the assumption of a single asperity inclination. Ladanyi and Archambault (4) have suggested a model that provides a physically appropriate simulation of the shearing process. Its main restriction, however, lies in the accurate collection of the required input parameters, which is greatly hindered by currently available laboratory and in situ methodology. Heuze and Barbour (10) have proposed a model that has the capacity for strain softening and dilatancy effects. Swan (15) has analyzed the contact mechanics

of a number of rock joints by obtaining information on the detailed topography of joint surfaces. Barton (6,7) and his co-workers have (8,11,12) provided valuable contributions that improved our understanding of the behaviour of rock joints. The Barton-Bandis model (11) is a most comprehensive design tool, backed by an abundance of experimental data. Its major premise is the use of simple index tests in order to determine the input parameters. As such the overall reliability of the model can not be any greater than the quality of the derived input parameters.

In recent years the use of a Power Curve has been applied as a means of representing failure envelopes of discontinuities. Hassani (13) and Udd et al. (14) have used direct shear test results and obtained Power Law relationships that adequately described the frictional behaviour of the rock joints.

ANALYSIS OF DIRECT SHEAR RESULTS ON ROCK DISCONTINUITIES

A large direct shear testing machine was used to determine the shear strength of rock joints. The testing procedure complied with the suggested ISRM guidelines (16). Table 1 provides a summary of the peak failure envelopes for the tested samples. A minimum of five points were used to determine the failure envelope. Table 2 indicates the ultimate failure envelope of the tested samples. The term "ultimate" was chosen over "residual" in that the authors tend to agree with Krahn and Morgenstern (17) view that rock joints do not have a fixed residual value as the case for soils. It is quite probable that the ultimate frictional resistance of hard unweathered rock depends on the initial surface roughness and the type of alteration occurring during shearing.

The power law equation is

$$\tau = A \cdot \sigma_n^B \quad (1)$$

where τ = shear stress, MPa
 σ_n = normal stress, MPa
 A, B = unit dependent coefficients, MPa, given in tables 1 and 2.

The relationships presented in Tables 1 and 2 were obtained by a statistical analysis of previously conducted tests (18,19). For comparative purposes the linear Mohr-Coulomb relationships were also derived.

The immediate temptation is to derive

a normalized equation of the power law. It is felt, however, that since in this particular case normalization will have to be attained by incorporating σ_c values obtained from other tests than the direct shear. Nevertheless the advantages due to normalization do not necessarily outnumber the loss of accuracy caused by introducing another test such as the point load or the Schmidt-hammer.

While equation (1) adequately describes the failure envelope, it is sometimes necessary to determine the angle of internal friction (ϕ) and cohesion (c) so that they can be used as input parameters for mine design excavations. An additional requirement being that these parameters should be easily determined. The traditional design approach is to take unique c and ϕ values and substitute for them in the design equations. This however is in direct contradiction with the curvilinear nature of the failure envelope i.e. their dependence on the applied normal stress. A way to overcome this difficulty has been proposed by Hadjigeorgiou (18).

Equation (1) can also be represented by:

$$\tau = \sigma_n \cdot \tan(\phi_T) \quad (2)$$

where ϕ_T is the total angle of internal friction. Barton (7) has suggested that ϕ_T comprises of the basic friction angle (ϕ_b), the peak dilation angle (d_n) and the shear component (S_n), i.e.

$$\phi_T = \phi_b + d_n + S_n \quad (3)$$

Hassani and Scoble (20) have examined the relationship of the ϕ_T (which they referred to as ϕ_p) with the residual angle of friction (ϕ_r) and the inclination of the surface asperities (i). In their work they found that

$$\phi_p > \phi_r + i$$

The reason for this discrepancy was attributed by the authors to a deformation of the asperities in the direction of shear. This reduces the effective inclination of the asperities, consequently resulting in lower frictional resistance.

To overcome these difficulties and in order to determine a simple criterion suitable for design considerations the

Table 1. Peak Failure Envelopes for Rock Joints.

Specimen	Peak Failure Envelopes					
	Power Law			Mohr-Coulomb		
	A	B	r	c	ϕ	r
Sandstone No. 1	0.586	0.928	0.9991	0.015	29.41	0.9972
No. 2	0.647	0.929	0.9998	0.020	31.75	0.9999
No. 3	0.763	0.905	0.9999	0.043	34.99	0.9995
No. 4	0.759	0.876	0.9997	0.063	33.64	0.9876
Mudstone No. 1	0.912	0.796	0.9897	0.127	35.61	0.9976
No. 2	0.682	0.786	0.9992	0.226	24.40	0.9985
No. 3	0.864	0.840	0.9850	0.132	34.51	0.9949
No. 4	0.844	0.819	0.9904	0.367	26.69	0.9985
Siltstone No. 1	0.668	0.852	0.9987	0.102	28.18	0.9999
No. 2	0.847	0.872	0.9977	0.144	34.47	0.9884
No. 3	1.000	0.827	0.9977	0.167	38.14	0.9998

Table 2. Ultimate Failure Envelopes for Rock Joints.

Specimen	Ultimate Residual Failure Envelopes					
	Power Law			Mohr-Coulomb		
	A	B	r	c	ϕ	r
Sandstone No. 1	0.491	0.938	0.9956	0.000	27.33	0.9919
No. 2	0.530	0.943	0.9971	0.003	27.76	0.9929
No. 3	0.549	0.921	0.9988	0.007	28.50	0.9985
No. 4	0.608	0.897	0.9997	0.042	28.61	0.9981
Sandstone saw-cut	0.723	0.967	0.9986	0.054	32.80	0.9974
Mudstone saw-cut	0.527	0.989	0.9894	0.000	27.87	0.9952
Siltstone saw-cut	0.575	0.994	0.9945	0.009	29.53	0.9998

following modified, curvilinear form of the Mohr Coulomb criterion could be more appropriate.

$$\tau_i = c_i + \sigma_n \cdot \tan \phi_i \quad (4)$$

where: τ_i = instantaneous shear strength, MPa
 ϕ_i = instantaneous angle of friction, degrees
 c_i = instantaneous apparent cohesion, MPa

The difference between the total friction angle (ϕ_T) and the instantaneous angle of friction being that ϕ_T is secant to the failure envelope while ϕ_i is a tangential value (Fig. 1).

Assuming the validity of the Power Law relationship then by differentiating eqn. (1):

$$\phi_i = \tan^{-1} A B \sigma_n^{(B-1)} \quad (5)$$

The instantaneous angle of internal friction can then be equated to:

$$\phi_i = (\phi_b + d_n) \quad (6)$$

The instantaneous apparent cohesion which has been introduced in equation (4) is equivalent to the shear component S_n . This value is highly dependent on the surface roughness of the discontinuities in that surfaces with more asperities produce more gouge (Hassani and Scoble (20)) and on the rigidity of the asperities. For design purposes it can be shown that a good estimate of the instantaneous apparent cohesion is obtained by:

$$c_i = A \sigma_n^B (1-B) \quad (7)$$

Figure 2 demonstrates the relationship of ϕ_T , ϕ_i , and c_i with respect to the applied stress for mudstone specimen No. 2. It can be seen that ϕ_i decreases as the normal stress increases; similarly the c_i is increasing with increased normal stress. This is attributed to the collected grout created by the shearing of the asperities.

It is hence possible to determine the appropriate c and ϕ values at a range of normal stresses based on the A and B coefficients of the power curve. The basis of this theorem is that the strength envelope can be predicted by means of a Power Law relationship.

PEAK AND POST-PEAK TRIAXIAL COMPRESSIVE STRENGTH OF ROCK

Since the early work of Coulomb (21), several criteria have been developed in order to predict the behaviour of rock in compression. Theoretical criteria are generally an inadequate fit to data obtained by experimentation on rock over a wide range of applied compressive stress conditions. In modifying these theoretical criteria to better fit experimental data, some loss of simplicity is incurred. Alternatively an empirical criterion can be adopted, having little or no theoretical foundation, selected only to fulfill a specific practical requirement for accurate strength prediction and simplicity of use. This approach has been followed by Franklin (22), Hoek (23), Bieniawski (24), and Hoek and Brown (25) among others.

Hudson et al. (26) have described the development of mechanical testing of materials and testing machines. The two major advances they describe are the development of stiff frames and the use of feedback control systems. Rummel and Fairhurst (27), Wawersik and Brace (28) have used servo-controlled testing machines to determine the post-failure behaviour of brittle rock.

In this work, the experiments were performed in a "conventional" triaxial cell. This is characterized in that $\sigma_1 > \sigma_2 = \sigma_3$. The principal stress, σ_1 , was applied by means of an electro-hydraulic servo-controlled testing machine and the confining stress was maintained constant by means of a hydraulic pump unit. The rock specimens of 50 mm diameter and 100 mm height were accommodated in a Hoek and Franklin cell (29). The confining fluid pressure was kept constant and specimen dilation was recorded. In order to minimize the influence of the testing procedure on the material stress-strain characteristics, the ISRM standardized procedure was followed (16).

GEOTECHNICAL DESCRIPTIONS OF INVESTIGATED ROCK SPECIMENS

The major classification of sedimentary rocks in their division by origin into respective chemical and/or physical derivatives, i.e., sandstone, siltstone, mudstone, seatearth and coal. Major subdivisions of such rock types is on the basis of the average specimen grain size (Blyth and Feritas (30)). In this study, the investigated rock specimens were:

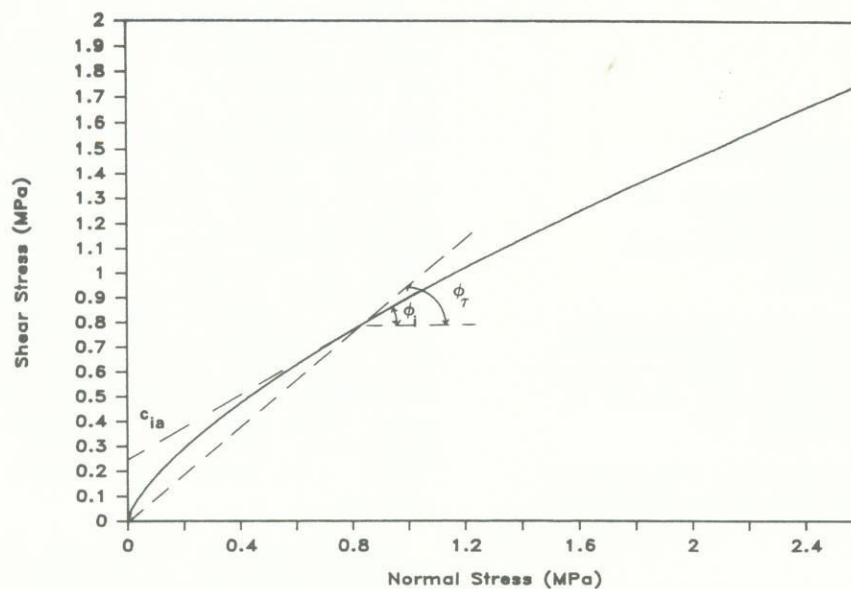


Figure 1. Modified Failure Criterion for Rock Joints

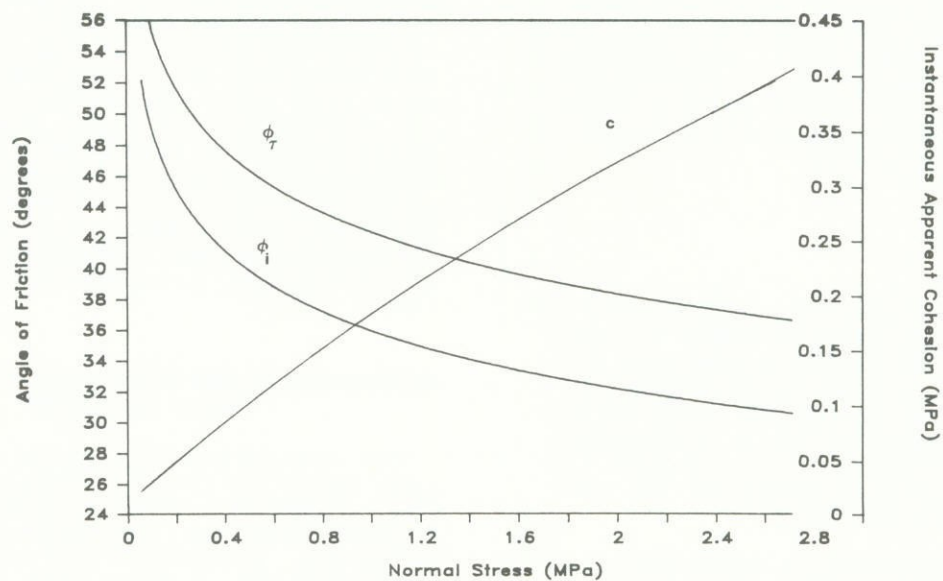


Figure 2. Variation of the Frictional Characteristics of Rock Joints with respect to Applied Normal Stresses (Mudstone Sample).

(a) Fine-grained sandstone:

This sandstone has a quartz content of greater than 60% by volume and a median grain size of between 0.2 and 0.06 mm. Individual grains were just distinguishable with a X10 hand lens. The unit weight, γ , was equal to 25.1 KN/m³, the void ratio $e = 6.5\%$ and the porosity $n = 6.10\%$.

(b) Coarse-grained sandstone:

Defined as having a quartz content of greater than 60% and a median grain size of between 0.6 and 2.0 mm. Individual grains are distinguished by the naked eye. Unit weight, $\gamma = 219$ KN/m³, $e = 15.07\%$ and $n = 13.10\%$.

The quartz content for siltstones is between 30 and 60%. Siltstones are commonly grey and the reflectance of the surface is not increased by the self-polishing test and remains matt or granular. Rock has a gritty feel when scraped against a knife. $\gamma = 26.7$ KN/m³, $e = 7.87\%$ and $n = 7.30\%$.

(d) Mudstone:

The quartz content of mudstone is less than 30%. Mudstone has a graying-black color and when the self-polish test is applied, sticky or soapy patches are found and the rock surface increases in reflectance. When scratched against a knife, mudstone exhibits a greasy, smooth feel. $\gamma = 24.5$ KN/m³, $e = 16.14\%$ and $n = 13.90\%$.

(e) Seatearth:

This is a genetic term which refers to the fossil soil that underlies coal seams. Seatearths are identified by their "earthy" texture and highly organic content, frequently present as fossil rootlets. In the strict sense the term includes siltstones and sandstones with high organic content. $\gamma = 23.5$ KN/m³.

ANALYSIS OF TRIAXIAL TEST RESULTS

Tables 3 and 4 provide a summary of the statistical analyses performed on the results of the compressive triaxial test program. It was evident that the investigated four criteria all gave acceptable descriptions of the σ_1 vs. σ_3 relationships. The four criteria for peak stress conditions were:

$$\sigma_1 = A \sigma_3^B + C \quad (8)$$

$$\sigma_1 = A \sigma_3 + C \quad (9)$$

$$\sigma_1/\sigma_c = A(\sigma_3/\sigma_c)^B + C \quad (10)$$

$$\sigma_1/\sigma_c = \sigma_3/\sigma_c + (m(\sigma_3/\sigma_c) + s)^{1/2} \quad (11)$$

Criterion (8) is a curvilinear criterion while criterion (9) is the traditional linear criterion commonly used for design purposes. While they both gave satisfactory results over the range of stresses applied, they were subsequently rejected. The reasons being in, that both equations rely on unit dependent coefficients A, B and C, and to the more serious underlying consideration the fact that these equations were not normalized. The advantages in normalizing the experimental results by dividing each measured value by the uniaxial compressive strength of that particular material is that it permits the direct comparison of a number of tests on the same plot in a dimensionless form. An additional case for normalizing is that the influence of testing techniques are minimized since they are common to both numerator and denominator of the dimensionless ratios.

The normalized criteria (10) and (11) which were selected are the Bieniawski (15) and Hoek and Brown (16) stress functions.

Criterion (10) which gave the best results, appears to be the more satisfactory of the four, for the investigated rocks. It differs, however, from the original Bieniawski criterion in that coefficient B is allowed to vary for rocks of the same type. This is similar to the approach followed by Brook (31). Bieniawski had opted to keep B as constant for each rock type, i.e. 0.75 for sandstone, not allowing for different size grains of the same material. The coefficients m and s for the Hoek and Brown criterion were derived by means of their proposed procedure (32). It should be noted that Hoek and Brown assume a value of $s = 1$ for intact rock material and a value of $s = 0$ for completely broken down rock. It was interesting to observe that the Bieniawski criterion consistently provided a better fit to the experimental data for intact rock than Hoek and Brown's more widely used criterion (Table 3).

The following criteria were used to describe the residual triaxial stress functions:

$$\sigma_1 = A \sigma_3^B \quad (12)$$

Table 3. Regression Analyses of Peak Triaxial Stress Functions.

Equation	F.G. Sandstone	C.G. Sandstone	Mudstone	Siltstone	Seatearth
1. $\sigma_1 = A \sigma_3^B + C$	A. 7.1575 B. 0.8893 C. 108.09 r. 0.9843	16.5981 0.6869 46.62 0.9965	12.3383 0.6953 20.69 0.9440	13.0050 0.6008 20.69 0.9440	10.1708 0.5600 20.17 0.986
2. $\sigma_1 = A \sigma_3 + C$	A. 3.8295 C. 118.0947 r. 0.9766	5.0307 62.7890 0.9874	3.1071 38.4757 0.9549	3.1274 60.3203 0.9840	2.4386 28.8195 0.9867
3. $\frac{\sigma_1}{\sigma_c} = A \left(\frac{\sigma_3}{\sigma_c} \right)^B + 1$	A. 4.1475 B. 0.8766 r. 0.9839	4.9381 0.6819 0.9974	4.8983 0.6952 0.9425	2.7565 0.5987 0.9894	2.6967 0.5591 0.9890
4. $\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \left(m \cdot \frac{\sigma_3}{\sigma_c} + s \right)^{0.5}$	m. 7.650 S. 1.000 r. 0.9559	17.253 1.000 0.9950	7.6215 1.000 0.9343	6.6998 1.000 0.9617	3.3449 1.000 0.9598

Table 4. Regression Analyses of Residual Triaxial Stress Functions.

Equation	F.G. Sandstone	C.G. Sandstone	Mudstone	Siltstone	Seatearth
1. $\sigma_1 = A \sigma_3^B$	A. 21.1039 B. 0.4931 r. 0.9907	18.9684 0.4876 0.9941	11.7299 0.6144 0.9884	14.0414 0.5762 0.9865	11.9293 0.5474 0.9986
2. $\sigma_1 = A \sigma_3 + C$	A. 3.9768 C. 17.7174 r. 0.9919	2.9224 18.6942 0.9777	3.3333 9.4407 0.9987	3.8314 9.0112 0.9987	2.5199 11.2129 0.9861
3. $\frac{\sigma_1}{\sigma_c} = A \left(\frac{\sigma_3}{\sigma_c} \right)^B$	A. 1.9255 B. 0.4842 r. 0.9897	2.6266 0.4825 0.9945	3.6488 0.6150 0.9877	2.7043 0.5738 0.9864	3.0614 0.5464 0.9985
4. $\frac{\sigma_1}{\sigma_c} = \frac{\sigma_3}{\sigma_c} + \left(m \cdot \frac{\sigma_3}{\sigma_c} + s \right)^{0.5}$	m. 2.727 S. 0.	3.050 0.0668	4.460 0.	3.940 0.	2.1650 0.3884

$$\sigma_1 = A \sigma_3 + C \quad (13)$$

$$\sigma_1 / \sigma_c = A (\sigma_3 / \sigma_c)^B \quad (14)$$

$$\sigma_1 / \sigma_c = \sigma_3 / \sigma_c + (m(\sigma_3 / \sigma_c) + s)^{1/2} \quad (15)$$

All criteria provide an adequate description of the residual stress functions for the tested rock specimens (Table 4). For the same reasons described above, the non-normalized criteria (12) and (13) were subsequently rejected in favor of criteria (14) and (15). It should be noted that when equation (15) results in a negative value of coefficient s , s is assumed to be equal to zero and coefficient m is recalculated.

Figure 3 illustrates the predicted peak behaviour of all rock types as represented by the Bieniawski criterion. It is interesting to observe the deviation in peak stress for the two sandstone samples. The Hoek and Brown representation of the peak stresses for the same specimen is shown in Figure 4, while the range of predicted peak stresses for the sandstone specimens, as predicted by the two criteria, is shown in Figure 5. It is evident that application of the Bieniawski criterion leads to higher predicted values.

Figure 6 and Figure 7 show the predicted residual stress functions for all rock types by the Power Law and Hoek and Brown equations, respectively. The closer cluster of the Hoek and Brown curves may tend to suggest that this criterion is not as stress-sensitive as the Bieniawski criterion. This is again exemplified in Figure 8, where the predicted range for residual stress behaviour of sandstone specimens is contrasted with the Power Law functions.

DETERMINATION OF THE MOHR FAILURE ENVELOPE FOR THE INVESTIGATED ROCKS

The principal stress functions that have been investigated are more appropriate for underground excavation design. However, for surface design purposes, it is traditional to express the failure criteria in τ , σ coordinates. If the direct shear test is used on the jointed rock specimens, then the experimental data are readily represented in τ , σ space. The use of triaxial testing apparatus leading to σ_1 and σ_3 values calls for further manipulation of the experimental data. By far the easiest approach is to plot the σ_1 , σ_3 data points and graphically derive the failure envelope. This method, however, suffers in the somewhat arbitrary selection of the failure envelope, which is

inherent in most graphical techniques. Furthermore, this process has to be repeated for all tested rock specimens. Alternatively, several different criteria have been developed as a means of representation of the principal stress results for the triaxial test in τ , σ space.

(a) Bieniawski's Criterion

This is based on a particularly useful criterion proposed by Hoek (18) which stated that:

$$\frac{(\tau_m - \tau_0)}{\sigma_c} = A \left(\frac{\sigma_m}{\sigma_c} \right)^B \quad (16)$$

where $m = 1/2(\sigma_1 - \sigma_3)$ and B , C and τ_0 are constants. For practical purposes $\tau_0 = \sigma_t$ (uniaxial compressive strength).

Bieniawski (24) found that results could be obtained within an accuracy of $\pm 1\%$ if the uniaxial tensile strength was assumed to be one tenth of the uniaxial compressive strength. Therefore equation (16) can be written as:

$$\frac{\tau_m}{\sigma_c} = A \left(\frac{\sigma_m}{\sigma_c} \right)^B + 0.1 \quad (17)$$

(b) Hoek and Brown Criteria

Hoek and Brown (32) have shown that their empirical principal stress criterion could be expressed in terms of shear strength, τ , and normal stress, σ_n , as:

$$\left(\frac{\tau}{\sigma_c} \right) = \left(A \left(\frac{\sigma_n}{\sigma_c} \right) - \left(\frac{\sigma_t}{\sigma_c} \right) \right)^B \quad (18)$$

where

$$\frac{\sigma_t}{\sigma_c} = 0.5m - (m^2 + 4s)^{1/2} \quad (19)$$

Furthermore, Hoek and Brown have suggested that A and B are functions of the m and s coefficients.

(c) Power Law

Power Law has also been applied to triaxial test data in order to derive the failure envelopes. Brook (17) and Hassani (12) have both reached the conclusion that Power Law between normalized τ_m and σ_m is always, if only marginally, a more accu-

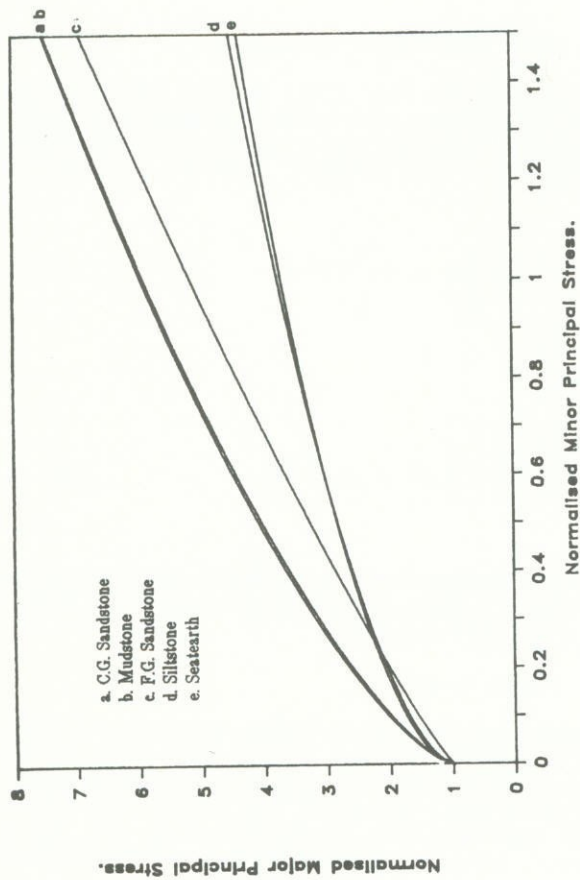


Figure 3. Bieniawski Peak Failure Criteria for all Rock Types

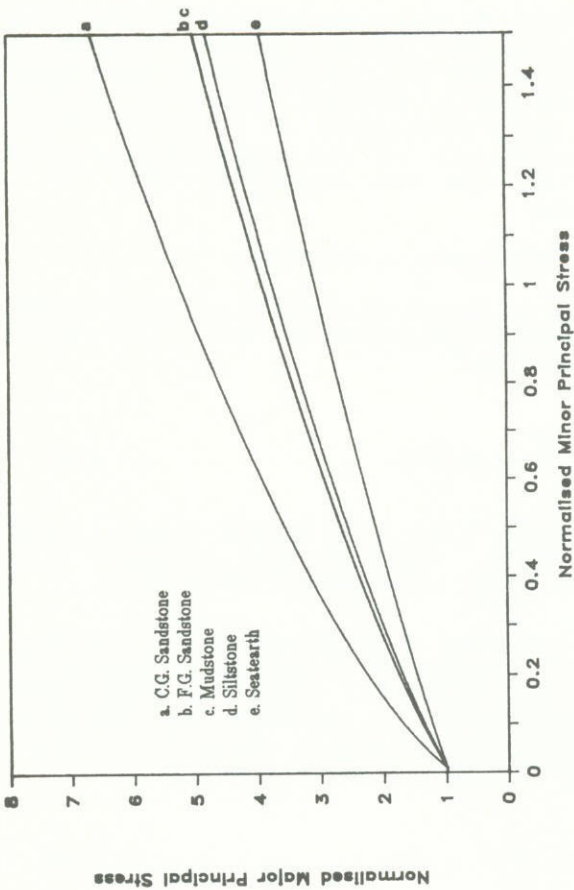


Figure 4. Hoek and Brown Peak Failure Criteria for all Rock Types

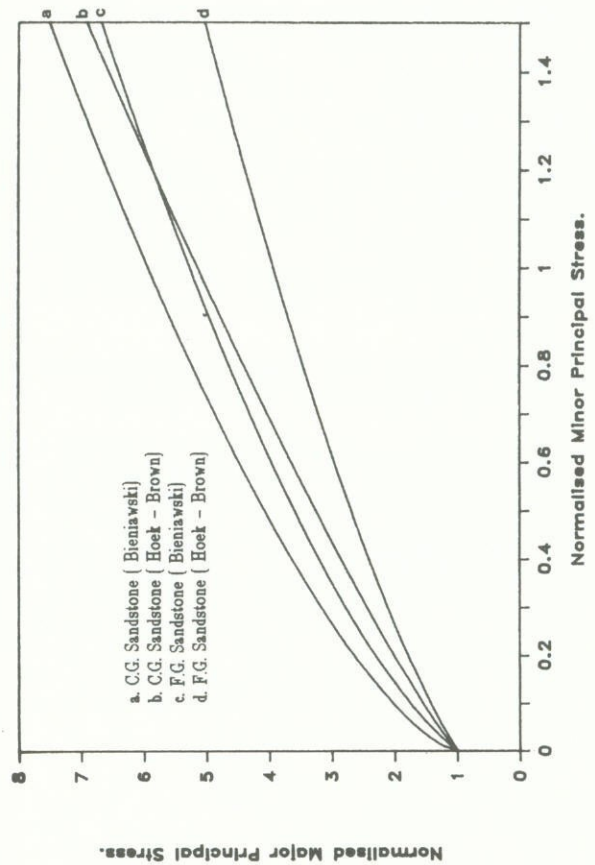


Figure 5. Comparison of Predicted Peak Values for the Sandstone Specimens

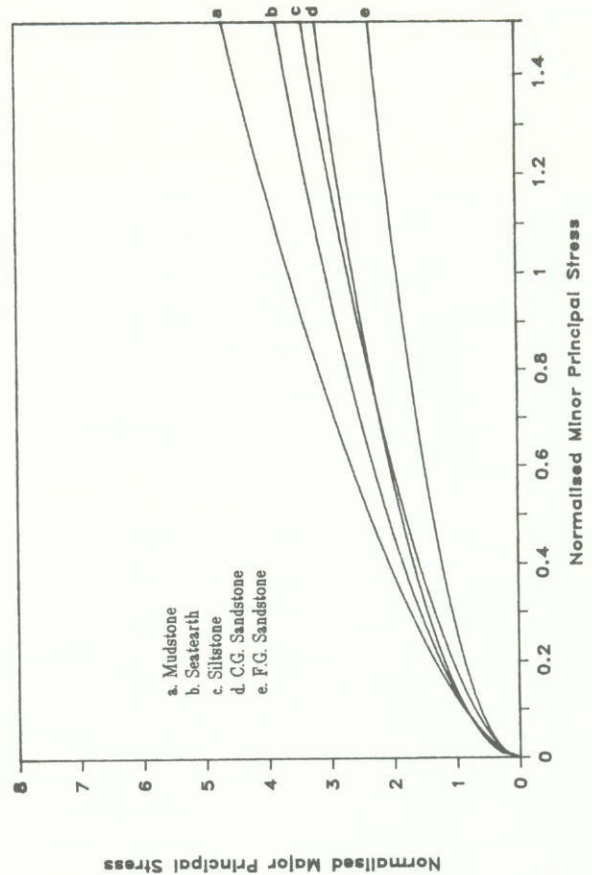


Figure 6. Power-Law Residual Failure Criteria for all Rock Types

rate relationship. They recommend that:

$$\left(\frac{\tau_m}{\sigma_c}\right) = A\left(\frac{\sigma_m}{\sigma_c}\right)^B \quad (20)$$

should be employed.

(d) Analytical Solutions

Ucar (33) has taken the empirical m and s coefficients derived by the Hoek and Brown method and has proceeded to derive the failure envelope by the analytical rather than the empirical solution proposed by Hoek and Brown.

The final form of his proposed equation is:

$$\tau = \frac{m \cdot \sigma_c}{8} \left\{ \frac{(1 - \sin \phi)(1 - \sin^2 \phi)^{0.5}}{\sin \phi} \right\} \quad (21)$$

where

$$\phi_i = \sin^{-1} \left[\frac{k}{3} 2 \cos\left(\frac{1}{3}\right) \cos^{-1} \left(1 - \frac{27}{4k^3}\right) + \frac{4\pi}{3} + 1 \right] \quad (22)$$

and

$$k = \frac{8}{m^2} m\left(\frac{\sigma}{\sigma_c}\right) + S + 1.5 \quad (23)$$

In an earlier work, Bray (34) has suggested that the Mohr failure envelope for the Hoek and Brown non-linear failure criterion can be described by:

$$\tau = (\cot \phi_i - \cos \phi_i) \cdot m \cdot \sigma_c / 8 \quad (24)$$

where

$$\phi_i = \tan^{-1} \left\{ \frac{4h \cos^2 30 + \left(\frac{1}{3}\right) \sin^{-1} h^{-3/2}}{-1} \right\} - 1/2 \quad (25)$$

where

$$h = 1 + \frac{16(m + s \sigma_c)}{3m^2 \sigma_c} \quad (26)$$

Under closer scrutiny, it is observed that both methods lead to the same analytical solution. It can, in fact, be shown that:

$$h = -\frac{2}{3} k \quad (27)$$

REPRESENTATION OF THE MOHR FAILURE ENVELOPE FOR THE INVESTIGATED ROCK TYPES

In table 5 the derived peak failure functions for the investigated rocks are presented while table 6 shows the residual failure functions. The difference between the definition of the τ_m/σ_c peak failure envelope and the actual normalized failure envelope, as obtained by using analytical techniques has been shown by Bieniawski (24). The τ_m and σ_m quantities are given physical representations as the maximum shear stress acting in the specimen at failure and the normal stress acting on the plane of the maximum shear stress. Bieniawski (24) and the Power Law failure criteria define the locus of points at the top of Mohr failure envelopes and are referred to as the maximum shear stress locus. An argument in favor of the shear stress locus representation is that the Mohr envelope can be constructed by a simple and reliable graphical representation. On the other hand, Hoek and Brown's solution as well as the analytical techniques provide a more accurate representation of the "true" Mohr failure envelope.

Table 7 compares the derived normalized peak failure envelopes for the intact fine-grained sandstone samples. For the range of normalized stresses investigated, it can be seen that the difference in prediction of the normalized shear stress between the semi-empirical Hoek and Brown relation (equation 18) and the analytical solutions of Bray and Ucar (equations 21 and 24) is never more than 2%. In contrast, the linearized Mohr-Coulomb relation could differ from the analytical solution by as much as 12%, this percentage number increasing for higher normal stress. Similar results were obtained from the other rock types.

Figures 9-13 illustrate the peak and residual failure envelopes for the investigated rock types. These were derived based on the analytical solutions and are believed to be representative of the behaviour of the rocks.

Table 5. Peak Failure Envelopes for Rock.

Equation	F.G. Sandstone	C.G. Sandstone	Mudstone	Siltstone	Seatearth
1. $\frac{\tau_n}{\sigma_c} = A\left(\frac{\sigma_n}{\sigma_c}\right)^B + 0.1$	A. 0.7245 B. 0.8349 r. 0.9967	0.7914 0.9268 0.9979	0.7895 0.8656 0.9905	0.7233 0.7949 0.9931	0.7236 0.7304 0.9919
2. $\frac{\tau_n}{\sigma_c} = A\left(\frac{\sigma_n}{\sigma_c}\right)^B$	A. 0.8233 B. 0.7048 r. 0.9976	0.9016 0.8231 0.9992	0.9027 0.7743 0.9931	0.8287 0.6899 0.9945	0.8306 0.6475 0.9937
3. $\frac{\tau}{\sigma_c} = A\left(\frac{\sigma}{\sigma_c} - \frac{\sigma_t}{\sigma_c}\right)^B$	A. 0.832 B. 0.661	1.096 0.696	0.802 0.654	0.839 0.663	0.641 0.593

Table 6. Residual Failure Envelopes for Rock.

Equation	F.G. Sandstone	C.G. Sandstone	Mudstone	Siltstone	Seatearth
1. $\frac{\tau_r}{\sigma_c} = A\left(\frac{\sigma}{\sigma_c}\right)^B$	A. 0.6007 B. 0.8295 r. 0.9988	0.6366 0.7827 0.9957	0.6774 0.8303 0.9976	0.6257 0.8384 0.9991	0.6326 0.7597 0.9983
2. $\frac{\tau}{\sigma_c} = A\left(\frac{\sigma}{\sigma_c} - \frac{\sigma_t}{\sigma_c}\right)^B$	A. 0.649 B. 0.714	0.630 0.681	0.631 0.703	0.710 0.709	0.546 0.619

Table 7. Comparison of Derived Normalized Failure Envelopes.

F.G. Sandstone σ/σ_c	Normalized Shear Stress		
	Analytical	Hoek & Brown	Mohr-Coulomb
0.1	0.312	0.316	0.327
0.2	3.401	0.401	0.401
0.3	0.482	0.479	0.476
0.4	0.556	0.550	0.550
0.5	0.625	0.617	0.624
0.6	0.691	0.680	0.698
0.7	0.753	0.741	0.773
0.8	0.813	0.799	0.847
0.9	0.870	0.855	0.921
1.0	0.925	0.909	0.995
1.1	0.978	0.962	1.070
1.2	1.029	1.013	1.144

Table 8. Instantaneous Angle of Friction and Cohesion Values for Fine-Grained Sandstone.

σ/σ_c	Analytical		Hoek & Brown		Mohr-Coulomb
	ϕ_i	c_i/σ_c	ϕ_i	c_i/σ_c	
0.1	43.59	0.22	42.21		
0.2	40.19	0.23	38.73		
0.3	37.66	0.25	36.24		
0.4	35.65	0.27	34.32		
0.5	33.98	0.29	32.77		$\phi = 36.59$
0.6	32.56	0.31	31.48		$c/\sigma_c = 0.25$
0.7	31.34	0.33	30.38		
0.8	30.26	0.35	29.42		
0.9	29.29	0.37	28.60		
1.0	28.43	0.38	27.83		
1.1	27.64	0.40	27.15		
1.2	26.92	0.42	26.54		

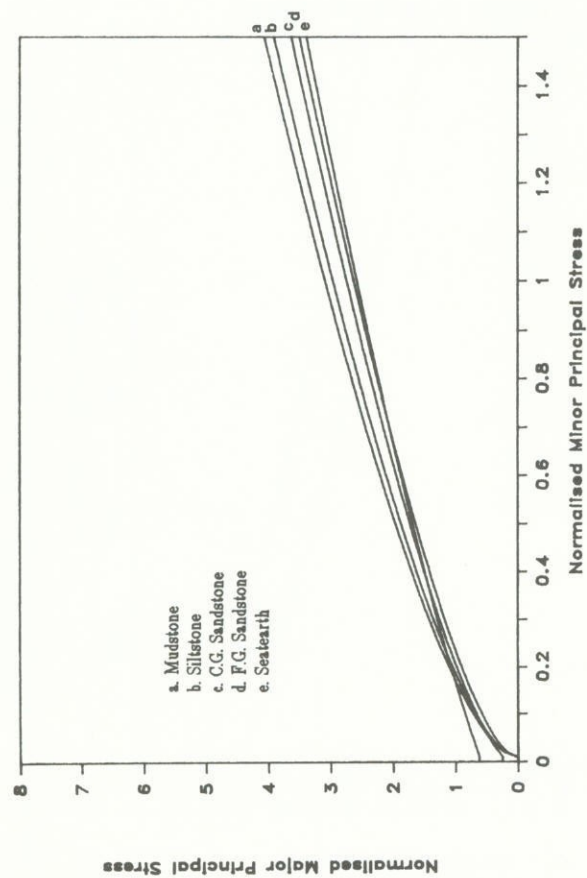


Figure 7. Hoek and Brown Residual Failure Criteria for all Rock Types

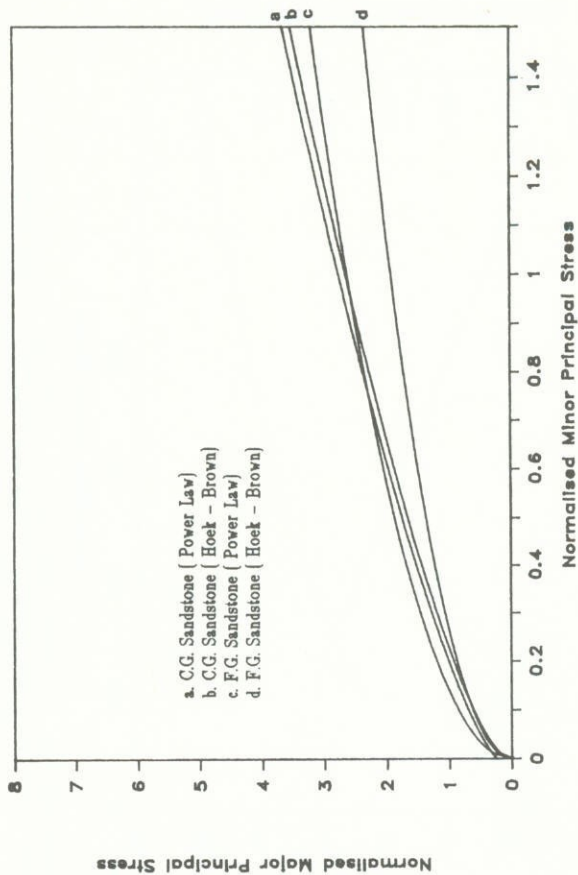


Figure 8. Comparison of Predicted Residual Values for the Sandstone Specimens

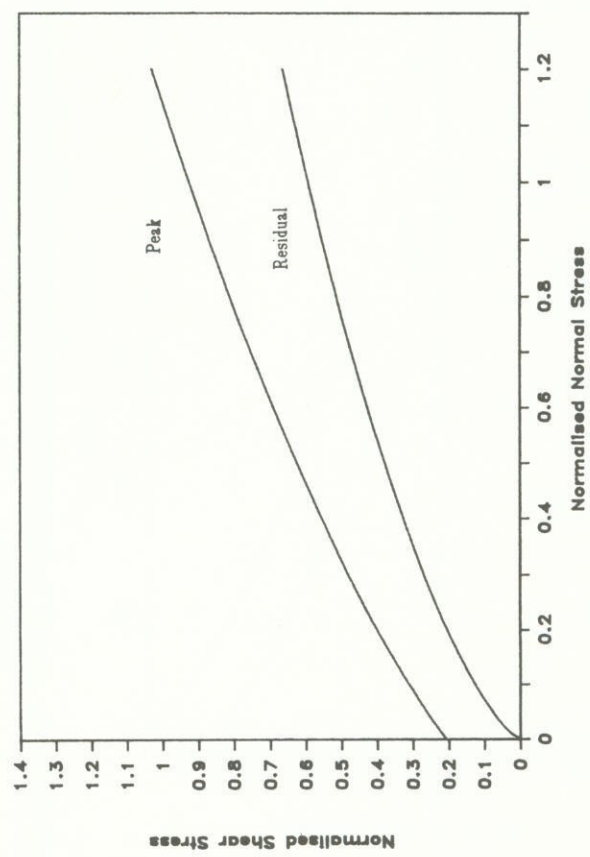


Figure 9. Peak and Residual Failure Envelopes for Fine Grained Sandstone

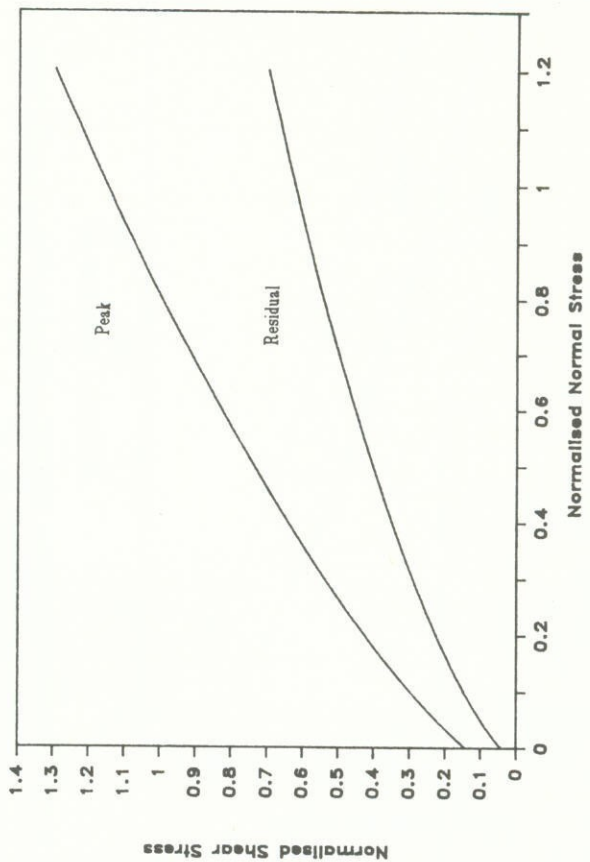


Figure 10. Peak and Residual Failure Envelopes for Coarse Grained Sandstone

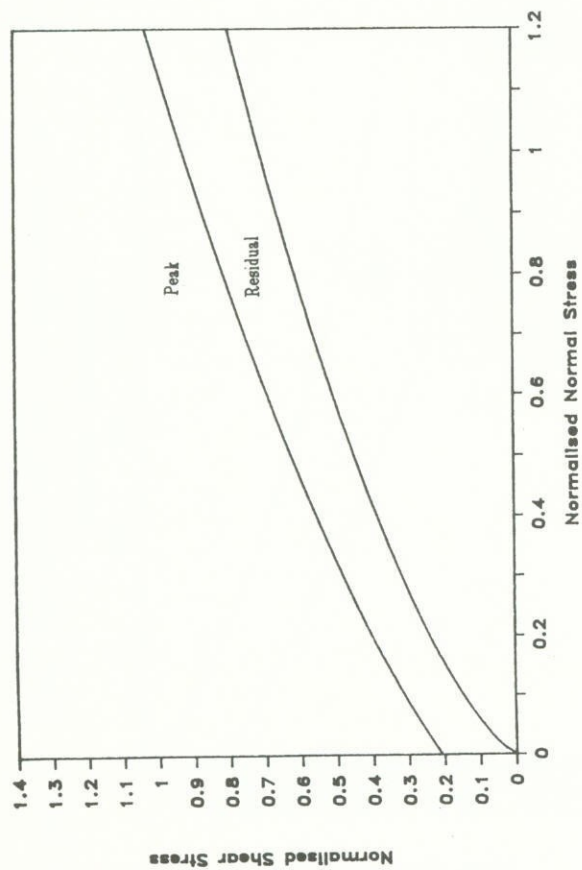


Figure 11. Peak and Residual Failure Envelopes for Mudstone

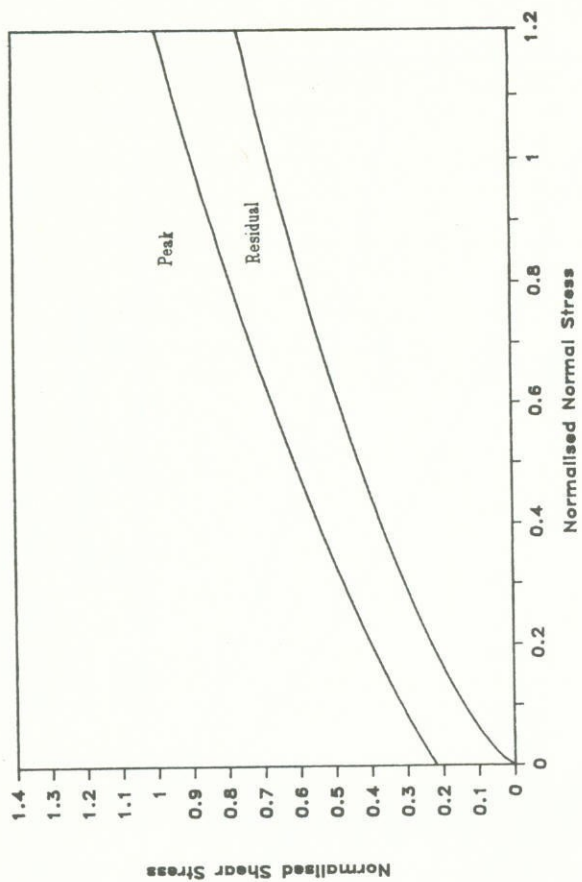


Figure 12. Peak and Residual Failure Envelopes for Siltstone

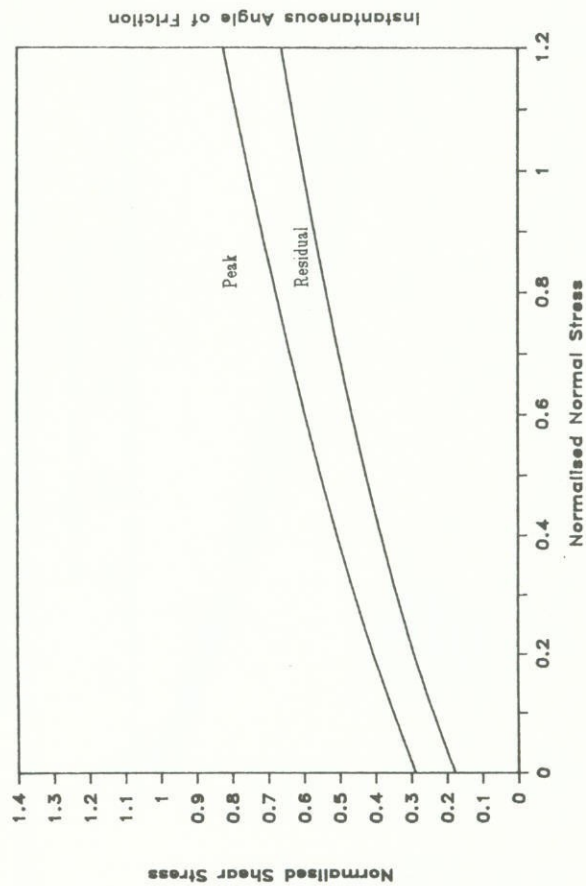


Figure 13. Peak and Residual Failure Envelopes for Seatearth

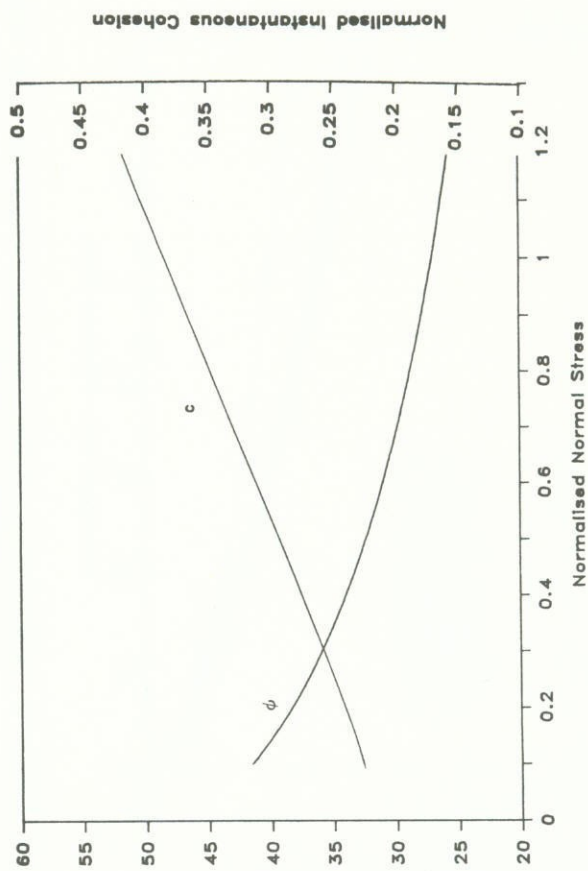


Figure 14. Variation of ϕ_i and c_i with respect to Normalized Normal Stress (Mudstone)

FRictional and Cohesive Properties of Rock

Based on the shear failure envelopes, it is possible to quantify the angle of internal friction and cohesion of the investigated rocks. The angle of internal friction, ϕ , describes the rate of increase of peak strength with normal stress and cohesion, c , is the shear strength intercept. Since the shear failure envelope, however, is not linear, then it is evident that the parameters ϕ and c are not constant but vary with respect to the applied stress. Therefore, it is inappropriate to quote singular ϕ and c values for rock unless the specific stress for which they were derived is also defined.

Determination of the instantaneous angle of friction, ϕ_i , and instantaneous cohesion, c_i , is possible and is dependent on the equation used to describe the failure envelope. Hoek and Brown (32) suggest the following relationship:

$$\phi_i = \tan^{-1} AB (\sigma/\sigma_c - \sigma_t/\sigma_c)^{B-1} \quad (28)$$

$$c_i = \tau - \sigma \tan \phi_i \quad (29)$$

where all parameters are based on their defined shear envelope.

Ucar (33) recommends the use of equation (22) while Bray (34) has suggested equation (25) for the calculation of the instantaneous angle of friction, while the instantaneous cohesion is defined by equation (29).

Table 8 provides a summary of the c_i and ϕ_i values for the fine grained sandstone under varying stresses. Similar results were obtained for all investigated rock type. It is thus demonstrated that use of unique values of angle of friction and cohesion, based on the linear Mohr-Coulomb failure criterion, can lead to misleading results. Another observation is the close agreement between analytically derived c_i and ϕ_i values and values derived by Hoek and Brown's approximation.

Figure 14 illustrate the behaviour of the ϕ_i and c_i under a range of normal stress values for the mudstone specimen. It can be seen that the angle of internal friction decreases as the normal stress increases, while the instantaneous cohesion increases as the normal stress increases.

CONCLUSIONS

The design of structures in rock requires a solid knowledge of the mechanical properties of the rock mass. In this work a modified criterion is introduced which is hoped to aid in the prediction of the frictional characteristics of rock discontinuities. This is based on the validity of the Power Law as a suitable description of the failure envelope. Its inherent simplicity makes it an attractive and more appropriate alternative to the linear Mohr-Coulomb criterion.

Based on the analysis of triaxial compressive tests on intact rock specimens the validity of the Hoek and Brown, Bieniawski and Power Law relationships was investigated. The results would indicate that the above criteria are powerful design tools that adequately describe the behaviour of the rock in compression. Furthermore, due to the established non-linearity of the failure envelopes, instantaneous angle of friction and cohesion should be used in subsequent analysis.

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ICE PILLARS, PACKWALLS AND BRATTICES

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ABSTRACT

This paper describes a system for providing structural support in underground mines, employing water ice for pillars, packwalls and brattices. It consists of a refrigeration brine chiller, brine circulation system, and disposable plastic containers. The system provides for the timely freezing of uniform ice pillars, and for ice maintenance and/or melting on demand.

Unique, useful ice properties include: expansion upon freezing, resilience to dynamic loads, and plasticity. Ice will not dilute or contaminate coal or ore.

Economic advantages include: 1) lower labor and material costs (water can be piped to the ice pillar site through mine-water, spray-water, or drill-water lines); 2) replacement or supplementation of timber, concrete or other roof support equipment; and 3) potential use of ice brattices as ventilation control; and 4) conversion of ice brattices to ice pillars upon retreat. See Figure .

A safety advantage for personnel and equipment is that ice pillars can be placed in hazardous sites, by remote control, if necessary, without human entry. All filling, freezing and melting operations can be controlled from a safe distance.

INTRODUCTION

This system employs water ice -- in lieu of conventional building materials -- for cribs, packwalls and brattices. This ice is frozen in place in a container containing a circulation tube for freezing brine. The brine is supplied via hoses and pipelines from a brine storage tank. The brine is chilled by a refrigeration plant (and/or radiators, if the climate is cold enough, See Figures 1, 2, and 3). The system is essentially an old-fashioned ice plant.

The idea of using ice in mines is not new; the mining industry has long recognized the merits of a structural material that can be 1) transported to the site as a liquid, 2) made solid on demand, and 3) made to fail under controlled conditions. Indeed, U. S. Patents involving the use of ice in underground mine support, appear as early as 1914. Recently, basic research validating the structural properties of ice was presented by Noranda Mines at the 1985 annual meeting of the Canadian

Institute of Mining and Metallurgy (CIM).¹ Noranda concluded that ice is ideally suited for pillar-extraction mining because it provides good lateral wall support, and reduces dilution, thus allowing maximum recovery of mineable ore.

This system solves fundamental problems that have previously kept ice from competing on a significant scale with more conventional materials and methods. In what follows, we treat problems of placement, freezing, maintenance, abandonment and/or forced failure.

PROPERTIES OF ICE

Although certainly not as strong as concrete, ice compares favorably with other "masonry" materials such as cemented rock and cemented hydraulic fills.^{2,3} Noranda concluded that confined ice meets and even exceeds the short term strength of present backfill materials. Ice utilization is governed by melting rate and creep rate.¹ Ice is stronger than some coals. It has a compressive strength of 276 psi. and will support 19.87 stons per square foot.^{4,5} Due to the brine circulation lines distributed throughout the ice pillars, a strength of 17 stons per square foot will be used for the strength of ice pillars and packwalls.

A comparison of estimated load carrying capacities, and of installed costs (labor, materials, energy) between timber cribs and similarly sized ice pillars is presented in Table 1. The timber cribs are four-foot-square, open cribs, without fill. Ice pillars are cylindrical and solid (except for the enclosed brine-circulation lines).

Table 1. COMPARISON OF TIMBER CRIES AND ICE PILLARS

Type of crib.	Load in stons.	Installed cost (US\$)
Timber, 4 ft. sq:		
4" x 4" sq. timbers	32	277.00
5" x 5" sq. "	51	295.00
6" x 6" sq. "	72	313.00
8" x 8" sq. "	128	348.00
Ice Pillars, cylindrical		
5 ft. diameter	412	191.00
8 ft. "	1055	313.00

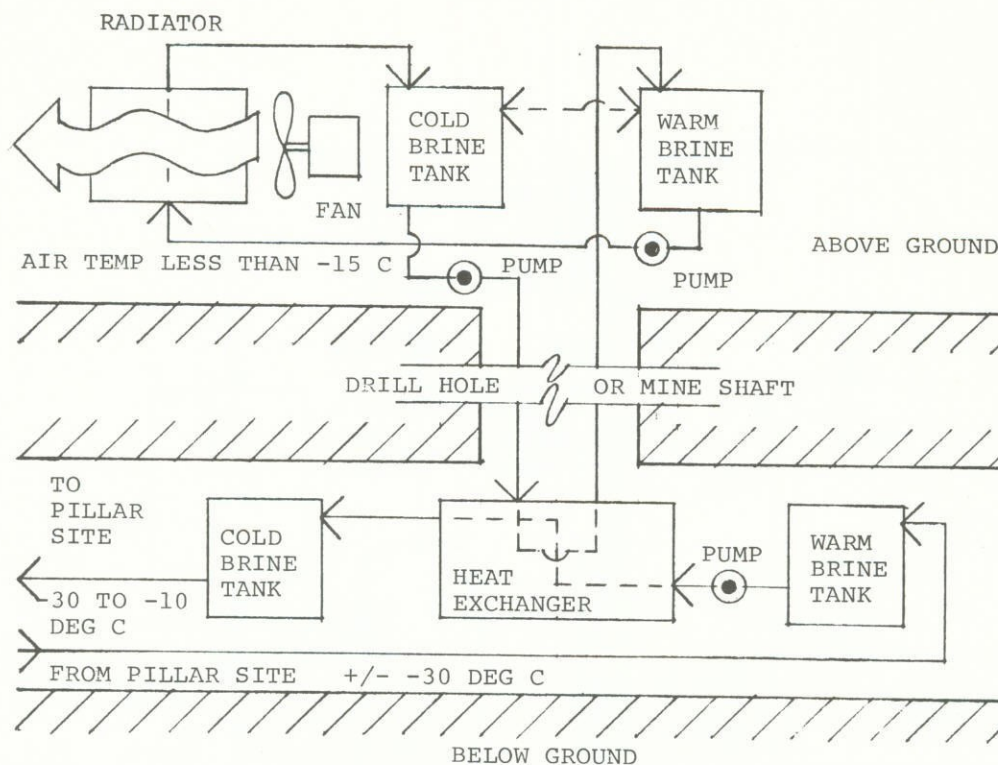


FIGURE 1. RADIATOR FOR COLD CLIMATE

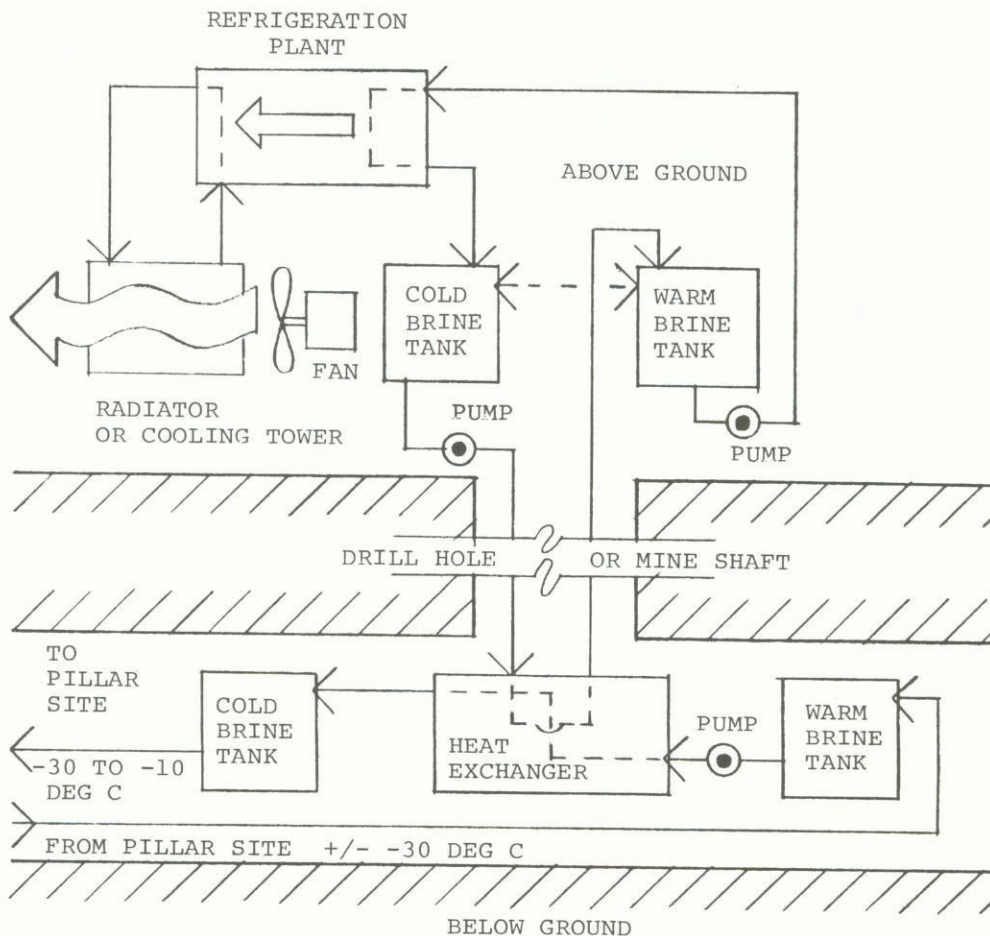


FIGURE 2. SURFACE BRINE CHILLER

Notes for preceeding table:

All cribs and pillars are six feet high (typical average height in room-and-pillar and longwall coal mines). Roof loads that timber cribs carry are assumed to be at 20% vertical compression. Roof loads that ice cribs carry are assumed to be at 15% vertical compression. Costs for the ice pillars include refrigeration energy, labor and capital costs. Timber crib costs include material, cutting, transportation underground, and erection.

The strength of ice pillars can be increased by filling the brine circulation lines with ice. With sufficient confinement, this can increase the strength of ice pillars from around 17 tons per square foot to as much as 200 tons per square foot (filling the entry with ice from wall to wall and floor to roof). This can permit mining room-and-pillar pillars; or, it can give greater strength to ice pillars and packwalls, for longwall tailgate support just prior to passage of the shearer. However, once this is done, the ice can no longer be maintained as the brine circulation lines will be plugged.

Filling brine tubes with ice is, therefore, a possible final act before abandoning ice pillars. At this point, the brine circulation pipelines and hoses can be salvaged. Ice pillar failure rates depend upon loading rate, pillar size, temperature of the surroundings, and time.

ICE PILLAR CONTAINERS

The system was originally designed primarily for temporary use during retreat room-and-pillar mining. The least amount of insulation was originally designed into the pillar containers, to reduce costs. Because mine operators have displayed interest in maintaining pillars for longer periods, and because of the possible use of ice for brattices, use of more insulation, for longer maintenance periods, has become a design option. See Figure 4. The amount of insulation required to maintain ice pillars will depend upon mine conditions and power costs.

As the pillars freeze, water expands, prestressing against the roof, ribs and floor. Ice expands 9% upon freezing. This tends to fill any voids between the ice container and the rock, thus preventing loosening and resultant rock failure.

Pure water makes the best ice. Relatively pure water can usually be obtained from the spray water lines. Mine water can be used even if impure. This is because the first ice to freeze is pure water ice, regardless of the salt content. Later ice will be impure and freeze at lower temperatures. In order to have pure-water ice, some of the water must be bled off of the container as it freezes to remove dissolved salts. Even with very impure water, nearly pure water ice can be frozen in the containers. After the container is full, and frozen, circulation of water through it will stop.

METHODS OF USE

Free-standing pillars will be frozen in place in lightweight sheet plastic containers containing a thin plastic tubing brine circulation system. The top of the

container can be attached to roof bolts or other anchorage in the roof, in order to secure the position of the container during filling and freezing.

Pillars should be proportioned such that column slenderness failure never becomes a problem. Where possible, the smallest horizontal dimension of the ice pillar should be at least one and one half times the height. For replacing shields on longwall shield line moves, ice pillars must be small enough so as not to interfere with the removal of the next shield. The upper limit to horizontal dimensions of pillars are governed by power costs. See Figure 5.

For very high pillars, containers will be laid up like bricks in a chimney, and frozen sequentially, until the pillar reaches the roof. Shapes and dimensions of such pillars are limited only by the needs and desires of the mine operators. See Figure 6.

Ice packwalls may be constructed using the above mentioned containers. They may be used to prevent excessive methane dispersion into operating areas of the mine. In longwall panel tailgate entries, packwalls may be used for roof support. See Figure 4.

The use of ice for brattices has several desirable features. As the ice freezes, it expands, making a tight seal between brattices and rocks. Ice deforms plastically to conform to rib, floor, and roof movement. These features produce and maintain seals where many other materials permit leaks.

Where brattices are to be used as pillars on retreat room-and-pillar mining, they may be made thicker than when used strictly for ventilation control. See Figure 7.

SAFETY ADVANTAGES OF ICE PILLARS

1. Ice pillars can be placed, frozen, and maintained by remote-controlled equipment, in areas too dangerous for personnel.
2. Roof subsidence can be partially controlled by selective failure of ice pillars through programmed melting. As an integral part of the system, pillars can be left to thaw naturally; or, they can be made to melt rapidly.
3. Failed ice pillars leave no stress concentrations in the gob. This minimizes potential for sudden and unexpected roof failures in underlying mine levels.
4. In longwall shield-line moves, ice pillars can replace the shields as they are moved. Ice pillar support can eliminate human entry of space where shield is removed.
5. Ice packwalls in tailgate entries can reduce the overall span of the roof beam, and also prevent rib failure where the ice contacts the wall.
6. In advancing longwall systems, ice does not require the long cure time of concrete or cemented fill materials.

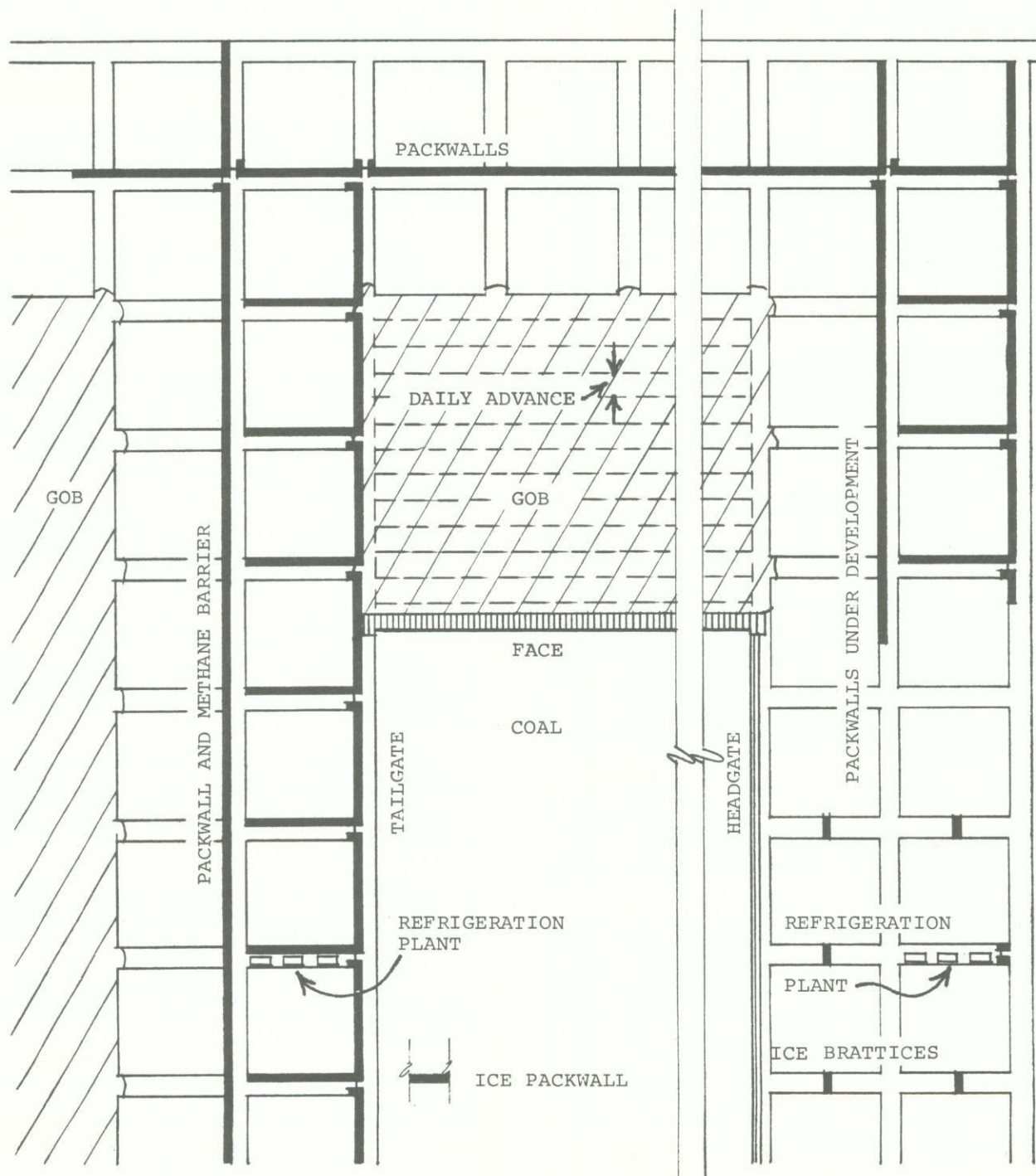


FIGURE 4. TYPICAL ICE PACKWALL LONGWALL APPLICATION

CONCLUSION

The above suggested uses of ice for underground mine roof support are as yet untested in the field. With this in mind, and the great variability of conditions in mines in the same district and mining the same ore deposit or coal seam, different mines will vary in their application of ice to roof support. The above suggested methods come from the author's experience and the suggestions of mine operators.

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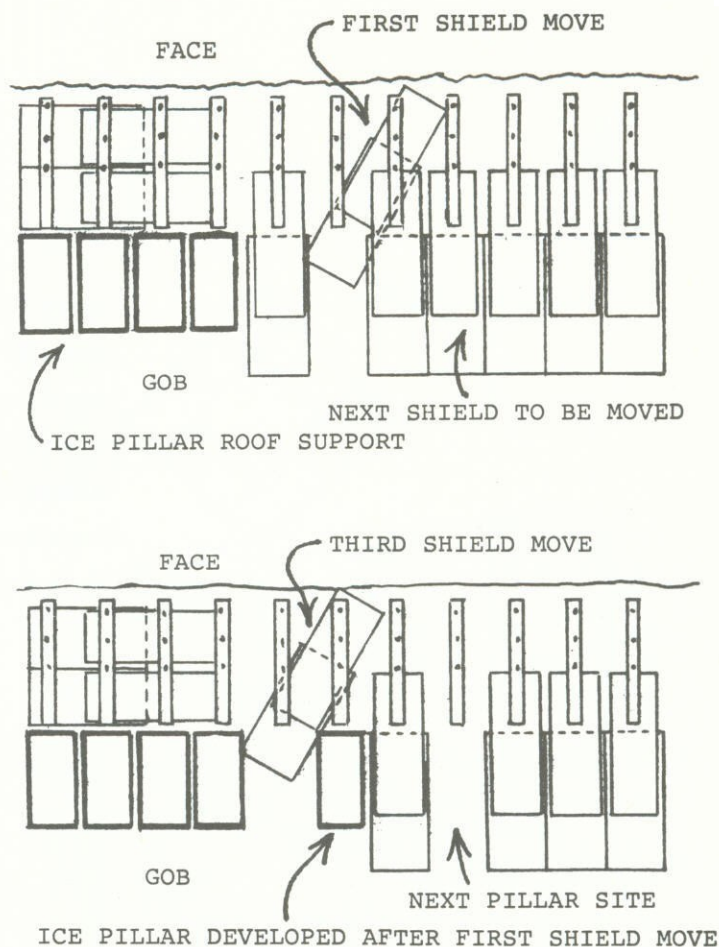


FIGURE 5. ICE PILLARS IN LONGWALL SHIELD MOVE

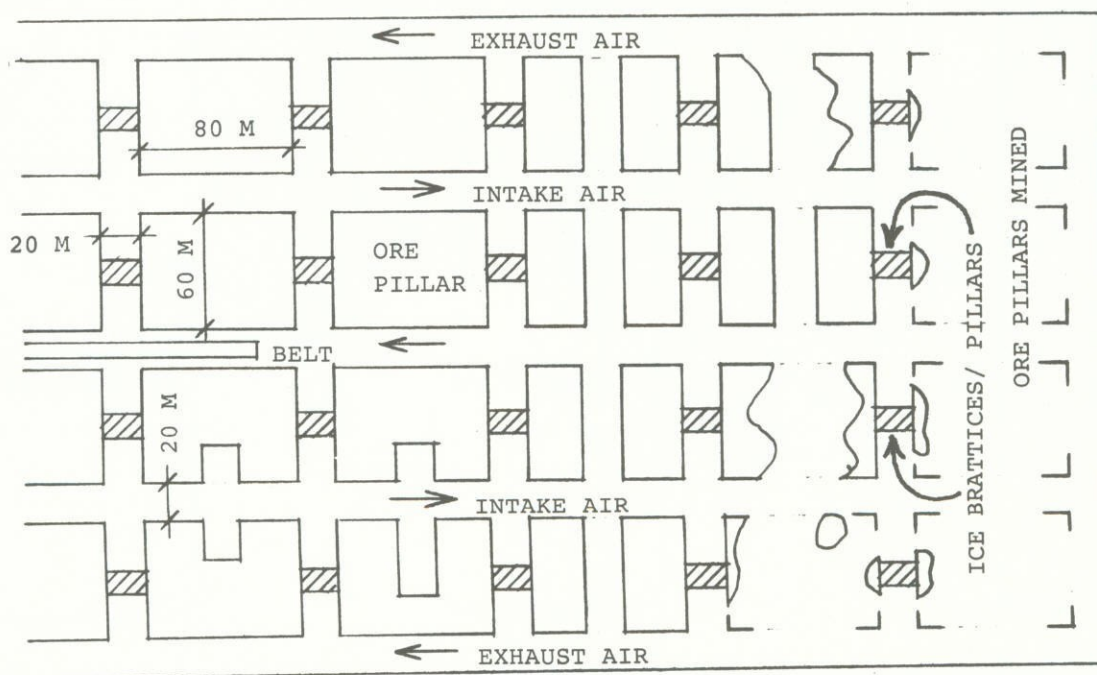


FIGURE 7. ICE BRATTICE/ PILLARS IN RETREAT MINING

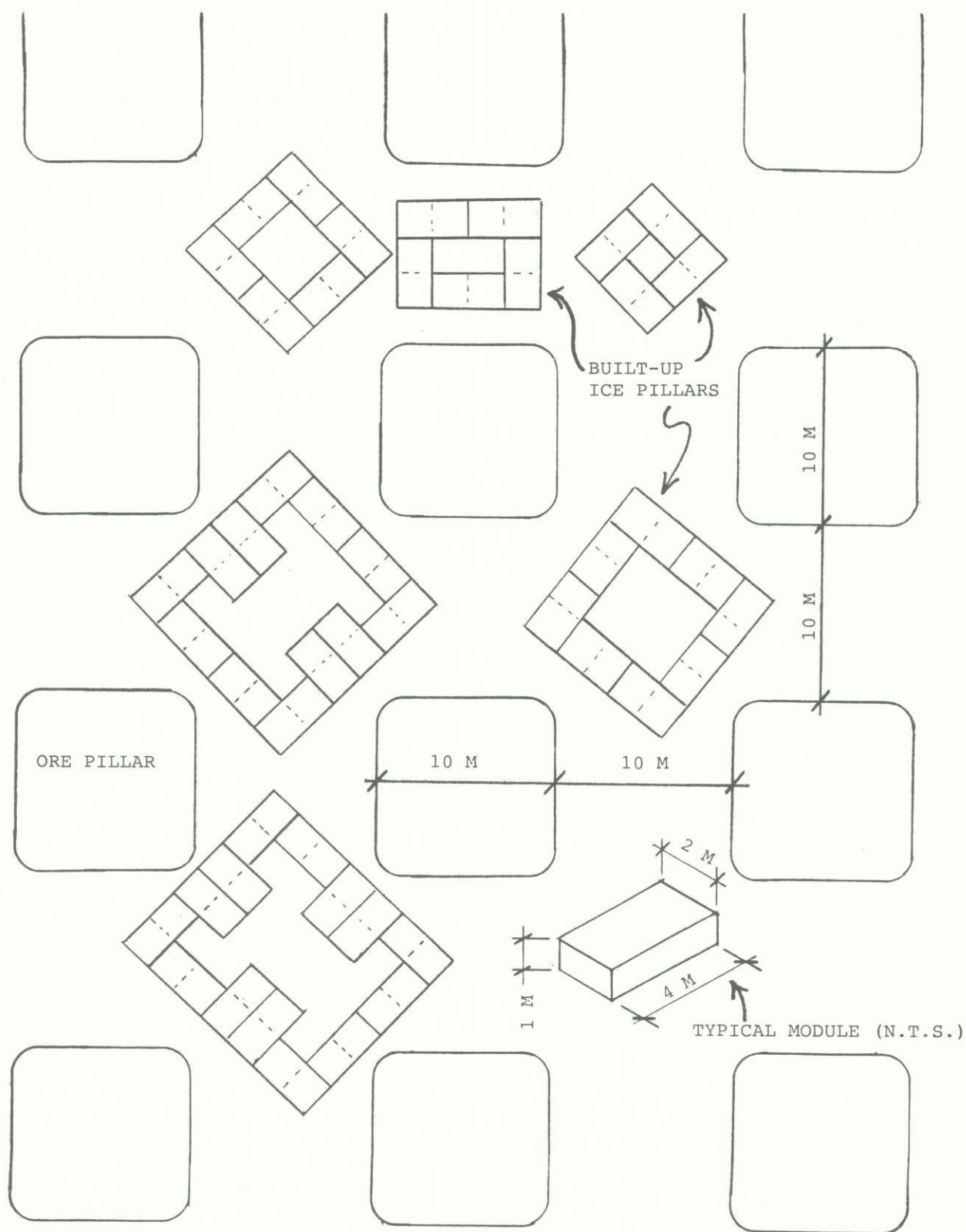


FIGURE 6. TYPICAL ICE PILLAR CONFIGURATIONS BUILT FROM 4 X 8 X 1 M MODULES

INVESTIGATION OF SUBSIDENCE OVER AML: A CASE STUDY

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INTRODUCTION

The problem of abandoned mine land (AML) subsidence is getting more severe because abandoned mine lands which used to be located in the remote areas have been gradually developed into suburbs or even urban areas. Abandoned mine subsidence is also becoming more common than before because the time-dependent effect of exposed underground materials. These materials will deteriorate gradually and cause subsidence as time passes. Subsurface conditions of abandoned mines are seldom known to the public. Mine maps are not always available. If the mine maps are available, their accuracy are very often questionable. In addition, the most common damages to surface structure and ground surface due to mine subsidence can also be created by other non-mining related geological hazards. These factors make the study of abandoned mine subsidence significantly complicated, tedious, and inconclusive. For instance, the cause, time, and amount of surface subsidence can be predicted with relative accuracy during active mining. However, it will be a considerable challenge to do so on abandoned mine subsidence. If abandoned mine subsidence can be predicted in advance, then the damages can be held to a minimum. However, this prediction technique is not yet well developed. Therefore, remedial measures require more attention. In order to have effective remedial measures, the real cause of the damage should be determined first.

This paper describes the approach which was employed for the determination of abandoned mine subsidence. A site investigation was made first. According to the information obtained from the site investigation, a hypothesis was set up. Then the instrumentation was used to prove the hypothesis. By going through the procedures for several cases, a guideline which can be used by the inspectors to inspect abandoned mine subsidence will be established.

CAUSES OF SUBSIDENCE

Many factors contribute to surface subsidence over abandoned mine land. From the mining

engineering point of view, the major causes of subsidence can be divided into mining-related and non-mining related. Mining-related subsidence is based on different failure mechanisms including roof, pillar, and floor failures. This type of subsidence results in two surface features: potholes and troughs.

Generally, roof failure in the abandoned mines will create pothole subsidence. A mine roof will sag right after its supports are removed. When roof sagging is too large to sustain its own weight and the partial weight from the overburden, the roof collapses. Failure will migrate through the overburden. As failure reaches the ground surface, subsidence occurs. The degree of roof sagging mainly depends on the span of the openings, roof material strength, and the geological discontinuities (Gray, et al., 1978; Aughenbaugh and Elifrits, 1983).

Generally, pillar and/or floor failure in the abandoned mines will create trough subsidence. If a coal pillar is too small or too weak to carry the weight from the overburden, in time the pillar will be crushed and trigger roof sagging or failure. Again, roof failure or severe roof sagging may induce subsidence. In the early days of coal mining, most of the pillars were not designed or retreated properly. In addition, the pillars in the abandoned mines were weakened by long periods of loading and the high humidity or a flooding in the mines. These are main factors which cause pillar failure. If the floor is too weak to support the load from the pillars and the overburden, the pillars will punch through the floor and cause roof failure. As mentioned previously, roof failure may induce subsidence. Floor problems generally are due to a low-bearing capacity of the floor material or improperly designed pillars.

Landslides, freeze-thaw effects, shrink-swell of soils, frost heaving, and limestone solution, etc. can induce surface subsidence, too. The damages due to these geological hazards are similar to those caused by abandoned mine subsidence and can be mistakenly attributed to mine-related subsidence. Generally, the damages due to non-mining related subsidence occur randomly, gradually, and periodically. In addition, some

special characteristics of each cause can most likely be observed in the damaged areas and their vicinity. Those characteristics will be discussed in another paper.

Site Description

The study site was located in southwestern Pennsylvania. The overall slope of the studied area dipped about 15% southwest. Figure 1 shows the general topography of the study area and the location of the exploratory boreholes. The ground surface around this area was covered with grass. The topsoil was residual materials which included silt, clay, and rock fragments and was damp, but not sticky. A rectangular, single-story structure with a full basement located on the hill was damaged. The damaged house was built in the 1940's and consisted of two portions: the northeast portion in brick siding and the southwest portion is wood siding. The ground level in the north and northeast of the house was about 5 feet higher than that of the south, southeast, and southwest. A relatively steep slope dipping about 25% toward the northeast side of the house was directly against the basement wall. No landslides or related phenomena were observed on the slope.

Geologic Conditions

This area was located in the main bituminous coal fields of Pennsylvania and belonged to the Monongahela Group of the Pennsylvania System. The Pittsburgh Coal Seam was located about 80 feet deep in this area. The thickness of the coal seam was about 6 feet. The strata above the coal seam were mainly shale and sandy shale. Two geologic columns of the boreholes are shown in Figure 2. According to the core loggings, the roof sections which consisted of claystone, interbedded coal, and shale were relatively weak and tended to collapse into the mined area; all the gob areas were about 10 to 13 feet high and about completely filled with roof materials.

Mining Activity

The Pittsburgh Coal Seam was mined by Oak Hill No. 4 Mine of Oak Hill Coal Co. in this area in the 1900's. According to the USGS 7 1/2" quadrangle topography map and a mine map, the damaged house was located at the south edge of the mine. However, the mine map only showed the mine property and boundary. No detailed mine layout was shown in the area beneath the damaged house.

Drainage System

The drainage system of this damaged house was good. There was no open downspout and no ground surface washout. This area was flooded by 2 to 3 inches of water two days before the

reported subsidence occurred, but water did not stay and drained to the south immediately.

Structural Damage

The damage to the house initially occurred on July 9, 1986. All the major damage developed within 2 to 3 days. Our first visit to the site was on August 14, 1986. The damaged zone, which was restricted to the portion with brick siding, was about 20 feet by 25 feet (Fig. 3). No damage was found outside the house. The portion with brick siding sank about 4 inches, and separated about 1/4 to 7/8 inch from the portion with wood siding (Fig. 4 and 5). The separation was developed on the mortar along the contact between the wood and brick sidings. Figure 6 shows the front view of the damaged wall which sank 4 inches at the right hand corner and the different levels of the ground surface. In order to avoid damage to the upper portion of the house, the brick siding portion had been jacked up about 4 inches by screw jacks inside the garage to keep the first floor leveled. Figure 7 shows a major horizontal crack developed on the walls along the mortar line and extended from the northwest wall through the northeast wall to the southeast wall. This crack, at the extreme left side of the northwest wall, was about 1/2 inch wide and reached about 4 inches in width at its northeast end, and located about 1 to 2 feet beneath the ground surface. It remained 4 inches wide and was uniformly distributed along the northeast wall (Fig. 7 and 8). A primary ground crack was observed inside the garage floor parallel to the northeast wall. The ground crack was about 1/2 inch wide and offset about 1 inch, and developed from the northwest end of the garage floor to the cement slab outside the garage door (Fig. 7). Several fine cracks about 1/16 inch wide and 2 to 3 feet long were observed at the corner of the walls inside the brick siding portion in the first floor of the house. No other subsidence damage was reported in the vicinity.

Hypothesis

According to the observed damage patterns as described in the previous section some conclusions can be made: (1) The maximum vertical movement of the house was 4 inches and the maximum horizontal movement was only 1/2 inch. Therefore, the house movements involved minor horizontal movement but major vertical movement; (2) The northeast wall was damaged the most severe and the damaged zone was only about 20 by 25 feet. Therefore, the epicenter was relatively small and located close to the northeast wall; (3) The damage to the house was not due to landslides because no landslide characteristics were observed, although house damage due to landslides was very common in this area (Briggs, et al., 1975). Evidence shows that the steep slope directly against the northeast wall was one of the major factors inducing house damage. Those factors will be discussed later; (4) Shrink-swell of soils would not be the cause of the house damage because neither the field characteristics of expansive soils (Jones, 1976) nor

a special cracking pattern or a puffy "popcorn" texture (Jochin, 1981) was observed. In addition, if expansive soil was the cause of the damage, the house should have been subjected to floor heave problems instead of settlement. Therefore, the possibility of the house damage being due to shrink-swell of the soil was slim; and (5) Obviously, freeze-thaw effects and frost heaving which always occur in the cold weather areas would not be the cause of the damage because the damages occurred in July's hot weather.

The damage to the house was due to the combination of mining subsidence and slope effects. The direction and intensity of the house movement can be illustrated by the geometric analysis. The existence of slope effects can be proven by the results of the instrumentation. Figure 9 shows schematically the movement due to subsidence and slope effects. A_0A_1 represents the movement due to subsidence and A_0A_2 represents the movement due to slope effects. β is the angle of the slope. The final movement A_0A_3 is $A_0A_1 + A_0A_2$. U_1 and S_1 are the horizontal and vertical movement components, respectively, due to subsidence. U_2 and S_2 are the horizontal and vertical movement components, respectively, due to slope effects. The directions of movement between U_1 which moves toward northeast and U_2 which moves toward southwest are opposite. The opposite movement directions minimize the final movement, i.e. $U_3 = U_1 - U_2$. S_1 and S_2 have the same direction. The same movement direction will superimpose to enhance the vertical movement, i.e. $S_3 = S_1 + S_2$. Without the slope effects, the damage patterns and the amount of movements might be different. For instance, the final horizontal movement (U_3) could have been larger than it was, because the opposite horizontal movement due to slope effect (U_2) prevented the brick portion of the house from moving further northeasterly. However, the final vertical movement (S_3) could have been less than it was, if the vertical movement due to slope effect (S_2) was not present to enhance the vertical movement.

The garage floor crack might indicate the location of the maximum tensile strain. The wall separation at the contact between wood and brick sidings might indicate the location of the critical deformation since all house damages were to the right of this point. For longwall mining or high extraction room and pillar mining, according to Peng and Geng (1982), the angle of critical deformation is about 10° less than the angle of draw. In the Appalachian areas, the angle of draw commonly ranges from 21° to 30° (Kohli et al., 1982). Accordingly, the angle of critical deformation will range from 11° to 20° . If the seam depth is 80 feet, then the edge of opening will be only 15 to 30 feet from the critical deformation point. But for the partial extraction, the angle of draw will depend on the failure configuration of the underground structures. Figure 10 shows schematically the relationship between the damaged house and the predicted underground layout. According to the previous assumptions, the epicenter would be very close to

the northeast wall. The nearest neighbor, about 100 feet from the damaged house, was beyond the influence zone. Therefore there has been no subsidence reported in the neighborhood.

Instrumentation and Results

In order to explore the subsurface, three 4-in.-diameter boreholes were drilled vertically from the surface northeast of the house (Fig. 3). Figure 3 also shows the relationship between the damages and the ground movements which will be explained later. The No. 1 and No. 2 boreholes were located close to the north and east corners of the house, respectively. The No. 3 hole was located between boreholes 1 and 2. A TV borehole camera was put into all the boreholes to examine subsurface conditions such as fracturing of the sidewall and gob. Three types of viewing attachments-downhole, radial close-up, and spotlight right-angle - were adopted to the camera head. The downhole viewing attachment provided a forward viewing - a 360° view of the borehole for general stratigraphy and void examination. The radial close-up viewing attachment was used for inspecting the sidewalls of the borehole. The spotlight right-angle viewing attachment was used to inspect the void and gob conditions. Along the sidewalls of the boreholes diagonal and horizontal fractures were observed in each formation. It showed that the overburden has been disturbed and most likely has been subjected to subsidence. At the mine level, a mixture of clay and rock fragments was observed in all the boreholes. In No. 1 and No. 3 boreholes, a void about 1 foot was observed at the top of the gob, but the extent of these areas could not be determined due to the limited openings.

An inclinometer and Sondex casings were installed in the No. 2 borehole to monitor ground movements. The inclinometer casing monitored horizontal movement and the Sondex casing monitored vertical movement. Figure 11 and 12 show the horizontal movement along the A and B axis, respectively. The results of the inclinometer measurements show that the upper portion of the hole moved southwest toward the house. It coincided with the surface downslope direction and proved the hypothesis, i.e. the existence of the slope effects. In the lower portion of the borehole, the strata moved northeast toward the epicenter. The intensity of the movement was not significant, probably because the movement was in the residual condition. An upper strata movement larger than the lower strata movement could be expected, but the opposite movement direction of the upper and lower strata was not common.

Figure 13 shows the Sondex measurement results. Vertical strata movement were small and random, probably a reflection of residual movement. The maximum settlement was about 1 inch.

CONCLUSION

The structural damage in this case was caused by a combination of abandoned mine subsidence and slope effects. However, the slope effects were triggered by the subsidence. The intensity of the subsidence was unknown, but the residual horizontal movement was about 0.5 inch on the upper strata and about 0.1 inch on the lower strata. The residual vertical movement was 1 inch. The vertical movement was larger than the horizontal movement. In the house, the vertical movement was also larger than the horizontal movement. Therefore, the house movement was consistent with the ground movement.

The true mechanisms leading to this abandoned mine subsidence was not sure, but the epicenter was estimated to have been located in the northeast of the house and was very close to the northeast wall. The influence zone of this subsidence was small because the mine depth was shallow and the epicenter was localized.

ACKNOWLEDGEMENT

This project was sponsored by West Virginia Department of Energy and U. S. Office of Surface Mining and Reclamation Enforcement. The statements and opinions expressed in this paper are strictly of the authors.

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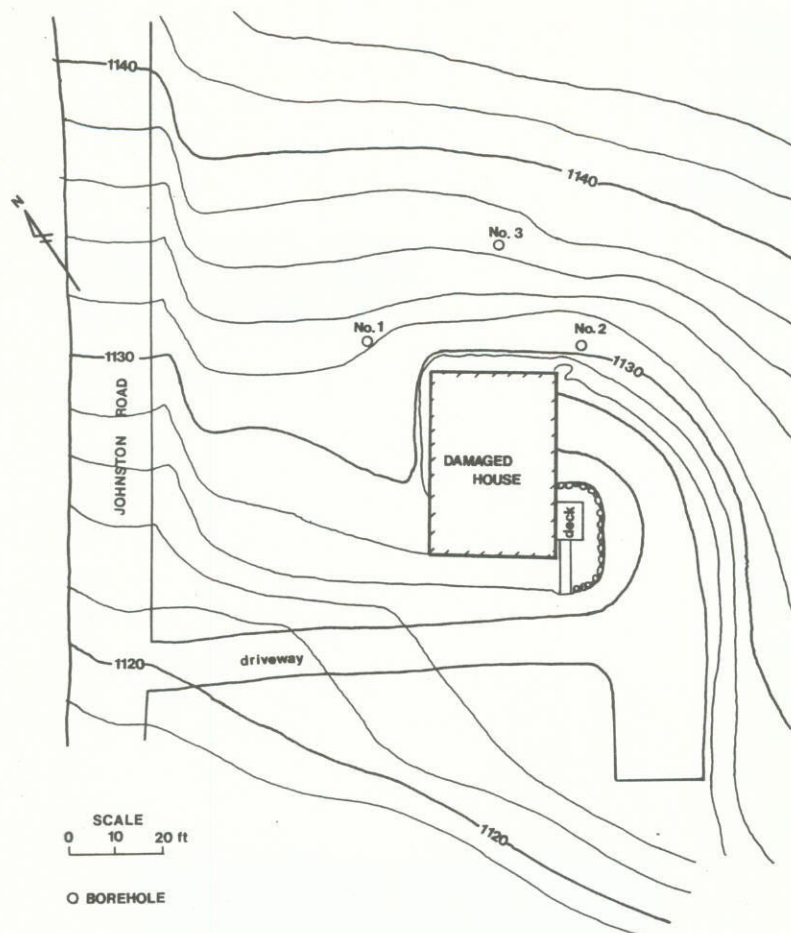


Fig. 1. The general topography of the studied area and the location of the boreholes. (after Peng and Hsiung, 1986)

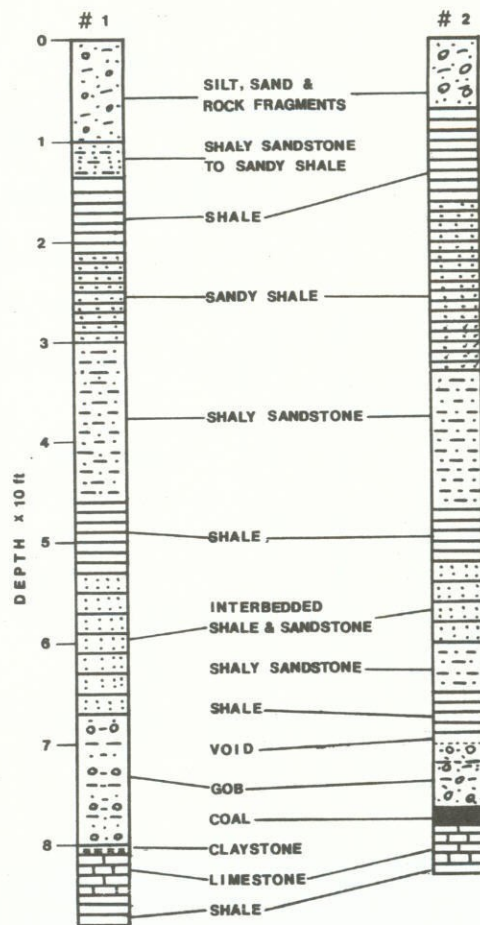


Fig. 2. Geological column of borehole #1 and borehole #2 shown in Fig. 1. (after Peng and Hsiung, 1986)

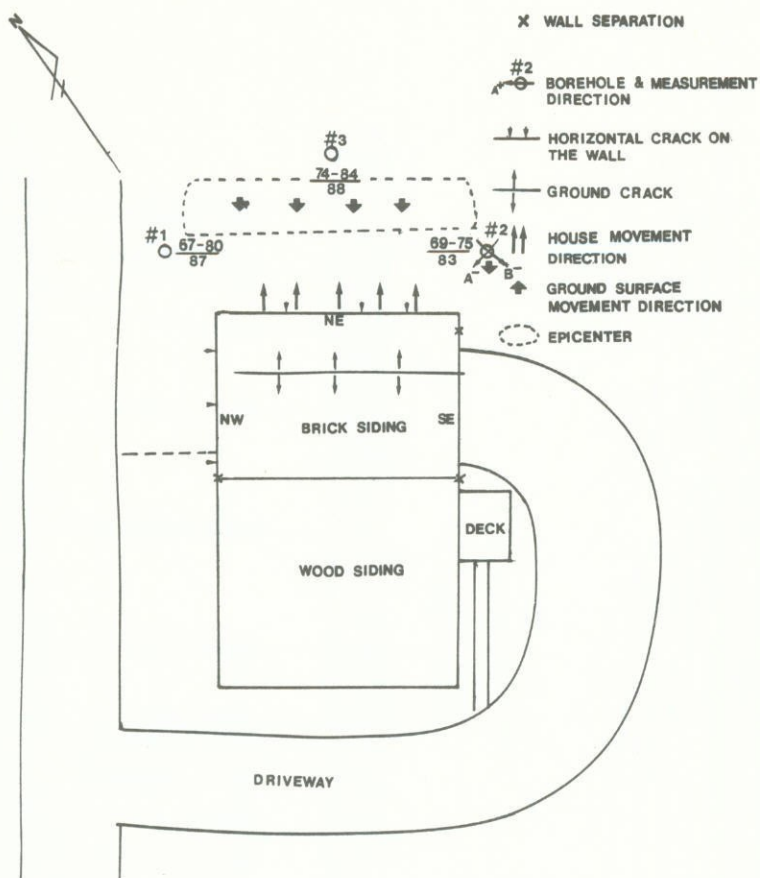


Fig. 3. Location of the boreholes and damages and the relationship between the damages and the ground movement.



Fig. 4. Separation on the southeast wall. (after Peng and Hsiung, 1986)



Fig. 5. Separation on the northwest wall. (after Peng and Hsiung, 1986)

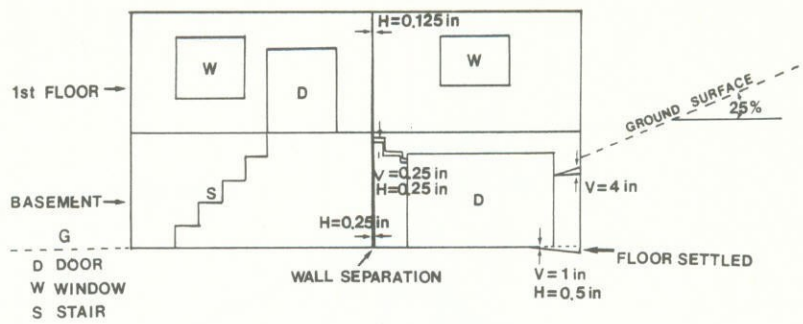


Fig. 6. Damage to the southeast wall (Front View) due to subsidence and slope effects.

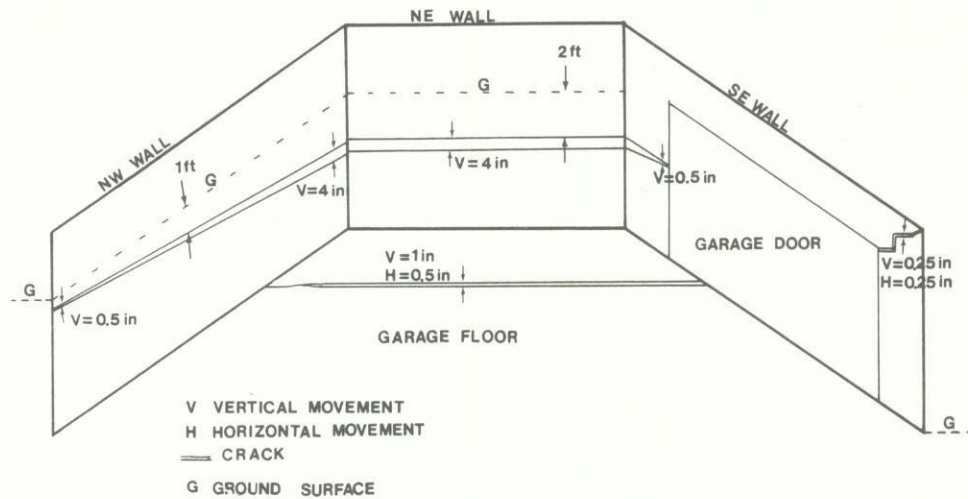


Fig. 7. Damage to the brick portion of the house.



Fig. 8. 4-inch crack on the northeast wall. (after Peng and Hsiung, 1986)

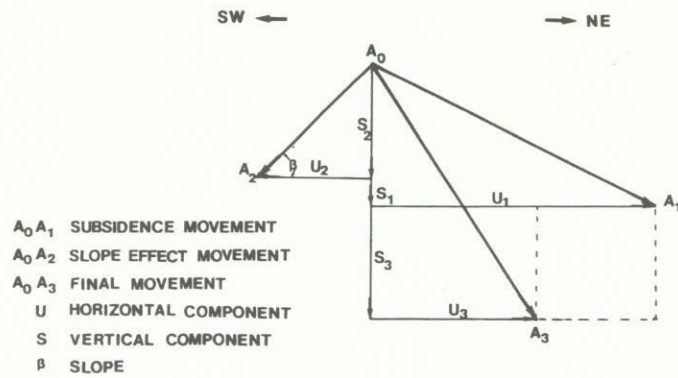


Fig. 9. Geometric analysis of earth movement and subsidence.

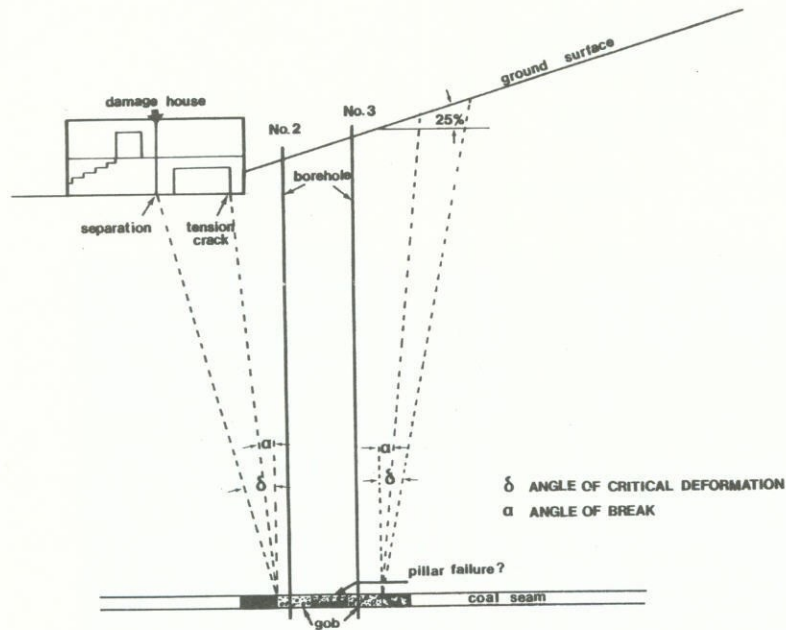


Fig. 10. The relationship between the damaged house and the predicted underground layout.

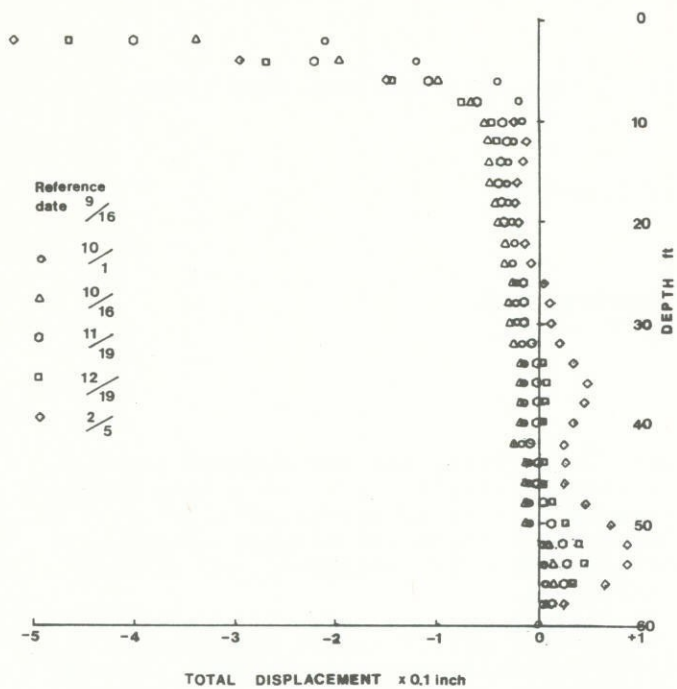


Fig. 11. Inclinator measurement along A-axis in borehole #2.

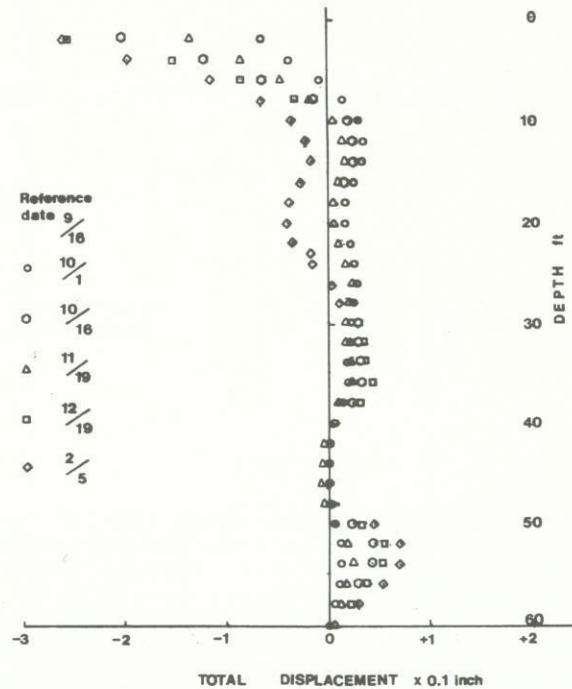


Fig. 12. Inclinator measurement along B-axis in borehole #2.

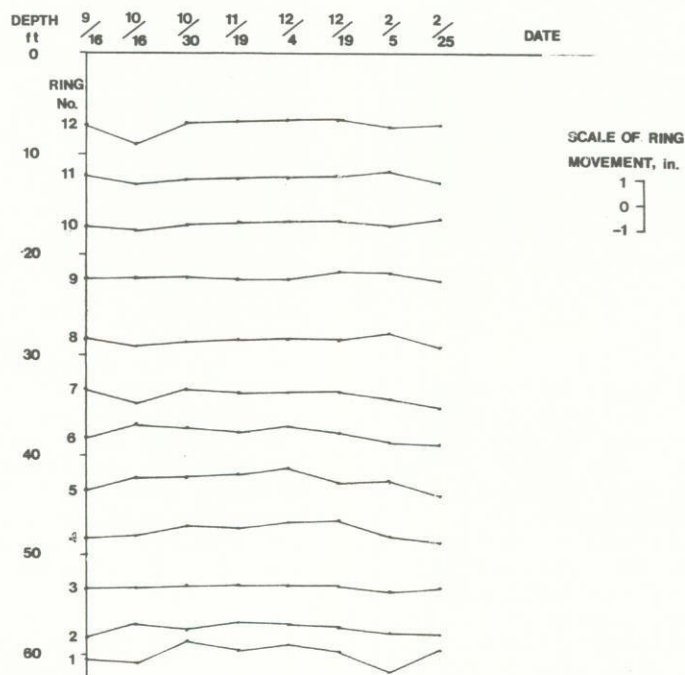


Fig. 13. Sondex measurement in borehole #2.

Design of Support Systems for Mining Tunnels in Carboniferous Rock Conditions

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ABSTRACT

The paper briefly reviews the type of support and stability problems encountered in mining tunnels in U.K. Carboniferous rock conditions, particularly the Coal Measures where almost all such tunnels are located. Stability prediction of mining tunnels is discussed and a computer-based model which deals with stratified rock conditions is described. Various schemes for monitoring the stability of mining tunnels are examined and the application of the results discussed. Comparisons between predicted and measured closure values are presented for a major coal mining tunnelling project at Selby in the North Yorkshire Coalfield. Conclusions are drawn concerning the application of the present prediction technique together with the importance of underground monitoring. Examples of different types of mine tunnel stability problems encountered in the U.K. Coal Measures are discussed and comments made on the secondary effects of tunnel peripheral rock weathering and corrosion of the associated support systems.

INTRODUCTION

The main problem associated with mine roadways and tunnels in U.K. Coal Measures is control of strata deformation and protection against potential rock falls from the roof and sides. The mine roadways serving longwall coal extractions are particularly prone to deformation and require special support considerations; this is usually provided for by the extensive use of steel arched girder supports, commonly described as colliery arches, in view of their favourable strength-deformation characteristics. Floor movement and its control is a special problem in the Coal Measures, especially where a weak seatearth forms the floor of a roadway. The deformation of gate roadways serving longwall faces is mainly associated with the proximity of the working face(s)

and consequently the deformation phase may be short-lived, but in deep mining conditions an increasing degree of time-dependent deformation becomes evident in certain types of geological and mining conditions.

The term mining tunnel is usually differentiated from mine roadway in respect of the type of service each is designed and constructed to provide. The latter serve current mining extraction operations and have life spans which cover a few years; in addition mine roadways are subjected to relatively larger stresses and deformations by virtue of the mining operations. Mining tunnels provide the access to major extraction areas which may include several coal seams, and consequently their design and support needs to be focussed on minimising any subsequent deformation; such tunnels in addition to providing long-term ingress and egress need also to provide the necessary stability for high speed transport of men, materials and mineral. Their life spans can cover periods of up to 80-100 years. The first logical step in the design process is to assess the likely degree of deformation taking rock strengths and properties into account and then to make comparison of the effects of applying different types of support with increasing support strength. This has been the approach to the mine tunnel stability problem adopted by investigators at the Department of Mining Engineering, University of Nottingham, and the present paper describes current progress in the application of this approach to major tunnelling projects in the U.K.

MINE TUNNEL STABILITY IN COAL MEASURES ROCKS

The stratified character of Coal Measures rocks follows an approximately predictable sequence involving sandstones, mudstones, siltstones, coal seams and seatearths. Mudstones usually form the roof of the coal seams, whilst seatearths

(under-clays) usually form the floors. These rocks are of low strength and readily fracture during the mining operations. The subsequent strata movements have a significant influence on the choice of support system since these rocks exhibit marked post-failure deformational behaviour.

The use of strata bolting in U.K. coal mining roadways, face-headings and other excavations has proved successful where short-term support has been the main objective. The application of steel mesh with end-plates on the bolts has been an integral feature adding greatly to the successful contribution made by strata bolting to support of coal mining excavations. Longwall face installation and recovery operations have benefited considerably from the use of this form of support system. The increased control of strata deformation by the bolting action has led to increased stability.

Rectangular steel sets continue to find favour in those geological conditions where strata deformation is anticipated to be relatively light. The ease of transport and speed of erection of such supports are major factors attracting increased attention. However the roof beam and form of leg to roof member joint need careful consideration in relation to anticipated duty. This is especially so where appreciable sag of roof beds and/or substantial lateral movements are expected.

The arched profile is still the predominant shape for mine roadways and tunnels in U.K. coal mines, owing to its naturally stable shape for Coal Measures rock types. Floor lift is still a major factor contributing to deformation of such excavations and can lead to excessive inward movement of support legs with decreased support stability. The arched profile is likely to remain as the most widely used shape for British coal mining roadways in view of the compromise it offers in achieving a high measure of natural roof stability coupled with a favourable support strength-deformation performance.

Circular tunnels offer a more stable shape than that of the arch for stratified rock conditions. However, the banded character of Coal Measures rocks usually results in uneven loading of the support system, and such effects need to be taken into account during the design stage. The application of circular tunnels in British coal mines has met with appreciable success in promoting increased stability.

STABILITY PREDICTION OF MINING TUNNELS

The mine tunnel stability problem is one of many variables if analysed to the

highest degree. From a mathematical analysis point of view it is impossible to consider all of the variables in one unique prediction technique. However it is possible to select the dominant variables according to the specific tunnelling problem all others being disregarded or considered as constants. In this way a prediction technique can be derived which can be used at the design stage to assess the likely behaviour of various support systems in proposed tunnels so that an optimum selection is made [2].

Tunnelling in stratified Coal Measures rocks is a unique situation as is tunnelling in massive hard rock formations or clayey conditions. As such, the factors effecting tunnel stability are different in each case and it is inappropriate to apply a prediction technique derived for one situation to one that is dissimilar.

In the stratified Coal Measures rock situation the dominant variables have been identified as:

- (i) the weak nature of the rock mass along with the subsequent formation of a zone of broken rock,
- (ii) the plastic properties of the broken rock,
- (iii) the strata sequence in which the tunnel is placed, and the relative displacements induced into the support around the tunnel periphery,
- (iv) the initial stress field, although in the light of previous work[1] it is reasonable to assume that the stress field is hydrostatic if it is unaffected by geological features, eg faulting, folding, etc. This is the general assumption made for U.K. Coal Measures rock at depths of between 500-1000 m.

Given then that the original stress field is hydrostatic a prediction technique is required that incorporates factors (i)-(iii).

U.K. Coal Measures rocks consist of sandstones, siltstones, mudstones, seathearts and coals arranged in cyclic series or a cyclothem. The mining depth and in situ stress field dictates that all but the strongest sandstones will fracture to some degree. Therefore it is essential to have information concerning both their pre- and post-failure strengths (see Figure 1).

Data from triaxial testing can be plotted as discrete points as shown in Figure 2. However a continuous relationship is required if the information is to be utilised in a mathematical prediction technique. This relationship is known as a 'failure criterion' of which there are many forms ranging from the straight line Mohr-Coulomb criterion and Griffith's Crack Theory to Hoek and Brown's curved formulae.

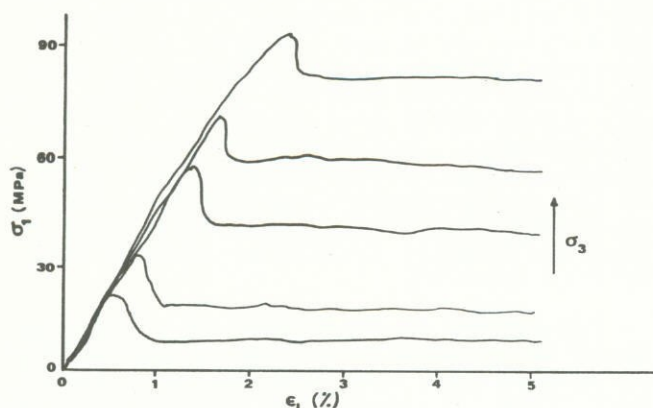


Figure 1 Stress-strain relationships for Coal Measures rocks

Work by Whittaker[3] confirmed that these criteria do not adequately fit the strength data for Coal Measures rock, especially at low confining pressures. This has led to the use of a general power curve criterion for both the pre- and post-failure strengths, defined by:

$$\sigma_1 = A \sigma_3^B + C \quad \dots (1)$$

where A, B and C are particular to a given rock type. Hobbs [20] performed earlier investigations into such relationships and especially on their relevance to roadways.

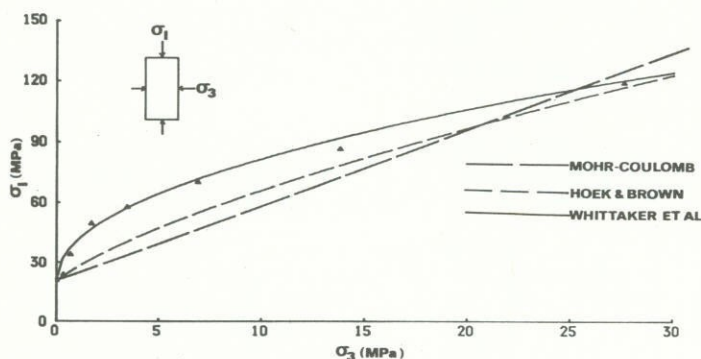


Figure 2 Illustrating relationship between major and minor principal stresses

The fit of this criterion along with the Mohr-Coulomb and Hoek and Brown criteria is shown in Figure 2, which serves to illustrate the point just mentioned.

A final point concerning the strength characteristics of the rock is the difference between laboratory strengths and the rock mass strength. Hoek and Brown[4]

have performed triaxial tests on rock samples of varying competency and formed relationships between their strength constants m and s and various rock mass classifications eg NGI and CSIR. No such data are presently available concerning Coal Measures rocks but it is the opinion of the authors that an adequate assessment of this can be made by assuming that the original rock mass is characterised by the post-failure strengths of the laboratory samples. In reality this represents the worst possible case as the rock mass cannot be weaker than failed rock.

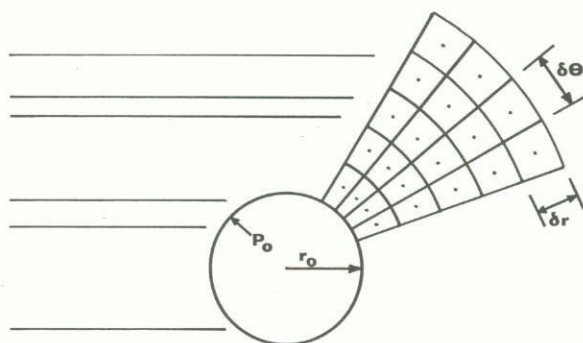


Figure 3 Representation of the rock mass around the tunnel by mutually independent elements

Figure 3 shows how the rock mass is split into mutually independent elements defined by δr and $\delta \theta$, thus allowing the presence of strata layers to be considered. The strata sequence is set up in a database with the rock properties of each element being assigned according to their position within the database.

Stress analysis is performed on all of the elements along a particular radial line. The assumption that stress is continuous across the strata layers is required to allow this analysis to be feasible, a description of which now follows.

The analysis starts with the stress equilibrium equation:

$$\frac{\partial \sigma_r}{\partial r} = \frac{\sigma_\theta - \sigma_r}{r} \quad \dots (2)$$

where σ_θ and σ_r are the tangential and radial stresses as shown in Figure 4.

The pre- and post-failure rock strengths for any element are defined by the equations:

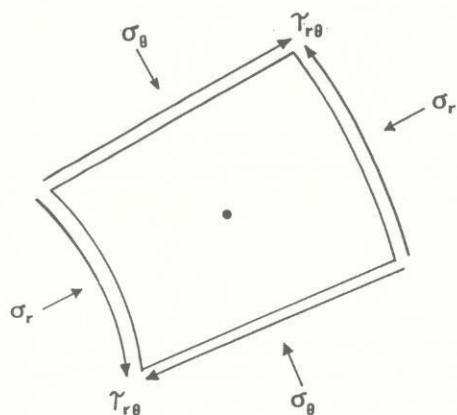


Figure 4 Stresses acting on a single element

$$\text{Pre-failure } \sigma_1 = A \sigma_3^B + C \quad \dots (3)$$

$$\text{Post-failure } \sigma_1 = A' \sigma_3^{B'} + C' \quad \dots (4)$$

Under axisymmetric conditions these can be re-written as:

$$\sigma_\theta = A \sigma_r^B + C \quad \dots (5)$$

$$\sigma_\theta = A' \sigma_r^{B'} + C' \quad \dots (6)$$

Substitution of (6) into (2) produces an expression for the change in radial stress over a radial increment of the rock mass, ∂r , given by:

$$\partial \sigma_r = \frac{(A' \sigma_r^{B'} + C' - \sigma_r)}{r} \cdot \partial r \quad \dots (7)$$

In a closed-form solution [5] this equation is solved by separating the variables and integrating both sides, using the appropriate boundary conditions, to produce an expression for σ_r at any radius r .

Equation (7) cannot be easily integrated and to allow the inclusion of the different strata, σ_r needs to be known for each element. Thus the equation is solved using a standard Predictor-Corrector technique for each element, starting at the tunnel boundary, the boundary conditions being that at $r = r_0$ (tunnel radius), $\sigma_r = P_0$ (support pressure).

The position of the Yield Zone boundary is fixed by the point where $\sigma_\theta + \sigma_r$ for the original rock mass = $2q$ (where q is

the hydrostatic stress). More often than not, at one element $\sigma_\theta + \sigma_r < 2q$ and at the next one $\sigma_\theta + \sigma_r > 2q$, thus the Yield Zone boundary lies between the two. Interpolation is required to fix the Yield Zone boundary accurately.

By performing this analysis along each radial line around the tunnel periphery the Yield Zone boundary is produced along with a complete stress analysis of the broken rock within it.

In order to calculate the closure at the tunnel boundary knowledge is required concerning the behaviour of the broken rock within the Yield Zone. When Coal Measures rock fails it expands quite substantially, the amount being determined by the confining pressure and axial strain. However, regardless of confining pressure it has been found that the maximum bulking occurs at approximately 5% axial strain [19]. Thus an equation can be formed relating the maximum bulking or volumetric expansion of the rock to the confining pressure, and it takes the form:

$$V = ae^{-\sigma_r/b} + c \quad \dots (8)$$

where a , b and c are particular to a given rock type.

For each element, σ_r of which is known, the volume increase can be calculated. If it is assumed that all of the movement is directed towards the tunnel then a radial expansion of each element is found. Thus, by summing the radial expansion of all the elements along a radial line, together with the elastic movement at the Yield Zone boundary, a value of closure at the tunnel boundary is obtained.

The final result is a varying closure profile representative of the strata sequence which can be used to assess where bending moments will be induced around the periphery, which strata layers are causing the problem and if the tunnel will be stable or not.

Examples of output are given later in the text.

MONITORING STABILITY OF MINING TUNNELS

Stability data from underground monitoring schemes is an integral part of any tunnel design project. Coupled with a reliable prediction technique such as the one outlined in the previous section, it will provide the best possible assessment of the likely behaviour of any future tunnelling operation, and lead to a successful design being implemented.

Roadway repairs in U.K. coal mines can

cost up to four times the initial drivage costs [6]. Consequently it is essential to have a complete understanding of the mechanics of roadway closure, so that they may be designed to minimise the need for repair work in the future [7].

Any monitoring or measuring scheme needs careful planning whereby, objectives, equipment, time scale, personnel and data processing and presentation are all clearly defined.

Initial objectives are essential to any scientific investigation, with tunnel monitoring being no exception. There now follows a description of the objectives relating to a tunnel stability assessment scheme within Coal Measures rocks.

It is a proven fact that roadways within Coal Measures rocks are greatly effected by the presence of a zone of broken or failed rock immediately surrounding it, termed the 'Yield Zone' [8]. The volume increase of the rock mass within the Yield Zone is primarily responsible for the large deformations often seen around such tunnels. Obviously, the larger the Yield Zone, the larger the deformation at the tunnel periphery. Various theories have been put forward to predict the size of the Yield Zone, all of which need to be assessed in relation to actual occurrences [8, 9, 10, 5, 11, 12]. Thus the first objective of any monitoring scheme within the Coal Measures is to measure the size of the Yield Zone at points around the tunnel. A secondary controlling factor of tunnel behaviour is the stress field prior to excavation. Hoek and Brown [4] compared examples of stress measurement worldwide and showed that the ratio of horizontal to vertical stress varied from 0.48 to 5.56, with the overlying conclusion that this ratio tended to unity as the working depth increased from 500 m and approached 1000 m. This depth range is comparable to the working depths in the U.K. coal mines and it is generally accepted that the stress field is hydrostatic. However, local trends differ greatly and stress measurement should still be part of any mining operation.

Once the causes of tunnel behaviour have been identified, their effects can be measured and this is the second objective of a monitoring scheme. Support deformation and loading are the two main aspects to be monitored although useful information can be obtained using visual observations such as the onset of plastic deformation of steel supports, the standard of support construction, support condition with respect to corrosion and the presence of large quantities of water.

Other objectives can be defined which

are common to all investigations namely use of reliable equipment, standards of data processing and presentation, error analysis, required measuring accuracy and cost effectiveness. However, the two previously mentioned are by far the most important, particularly in what needs to be measured in order to obtain as complete a picture as possible.

The range of equipment to perform such investigations is enormous ranging from simple tools to sophisticated state-of-the-art electronic devices. Many of these instruments have been developed for Civil Engineering purposes where highly accurate measurements are required. However the design approach to mine tunnels is different to that of Civil Engineering and in all UK coal mines there are certain restrictions on what is and what is not acceptable for use underground.

To re-cap then the required measurements are:

- (i) Strata Displacement and Yield Zone size,
- (ii) Support Deformation,
- (iii) Strata Loading.

Strata Displacement

The size of the Yield Zone cannot be directly measured but an adequate assessment of its greatest extent or Yield Zone boundary can be made by considering the relative displacement of points within the rock mass. Considering Figure 5 it can be seen that there is a distinct increase in the magnitude of the movements between numbers 2 and 3. Thus it is reasonable to assume that the Yield Zone boundary is situated between these two points. Number 2 is being subjected to the elastic movement of the intact rock beyond it, whilst number 3 represents the much larger deformations associated with the plastic movement of broken rock. Extensometers are used to measure these relative deformations.

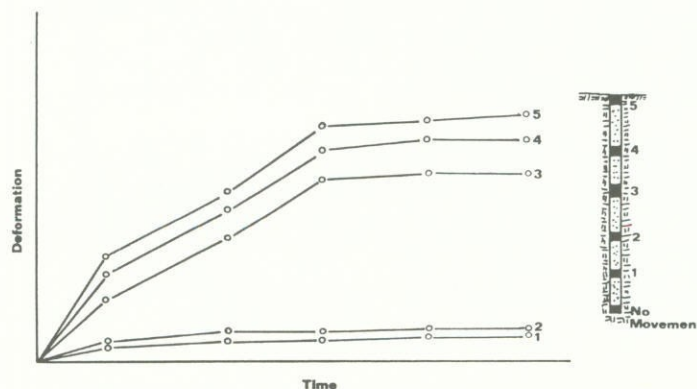


Figure 5 Extensometer results

An extensometer can vary in complexity from a single mechanically anchored rock bolt, allowing the movement of the tunnel wall to be monitored relative to the solid ground beyond the Yield Zone to the Trivec Probe [14] which allows three-dimensional movement to be monitored for several positions within the rock mass. In general the former method does not yield sufficient information for the study concerned whereas the latter is not practicable for mine tunnels. In general the requirements of an extensometer to be used in a British coal mine are:

- (i) ease of installation especially when close to the working face,
- (ii) anchorage capabilities beyond the Yield Zone in competent rock,
- (iii) intrinsically safe and/or flame-proof features,
- (iv) accuracy to within ± 1 mm,
- (v) robust and reliable for mine environments.

Using these guidelines, two principal extensometers have evolved for use in coal mines. The first is a Multi-anchor, strain wire type as shown in Figure 6, which is mainly used by Nottingham University. Basically hollow serrated steel anchors, linked by flexible plastic tubes are grouted into a drill hole at regular intervals. Each anchor has a steel wire attached to it which passes through the pipe to be exposed at the hole collar. By measuring the differential change in the wire length at the collar an indication of the displacement of the strata horizon in which the anchor is situated can be obtained.

The second type, uses a magnetic technique to monitor the movement of the anchor points (see Figure 7). Magnetic anchors are placed at pre-determined distances along the borehole and their movement is monitored by the use of a probe rod.

Both of these instruments satisfy the above requirements and have been proven in many investigations to give useful and reliable information.

Support Deformation

Deformation surveys are one of the simplest and cheapest forms of underground monitoring, and they yield much useful information. Support deformation data are vital in assessing the accuracy of computer prediction techniques. By comparison of predicted and measured closures more advanced and reliable numerical models will evolve. A secondary use of closure data is in evaluating the yield state of a support. Deformation monitoring gives advance warning of the onset of major loss of strength and allows remedial measures to be taken. In general, the maximum

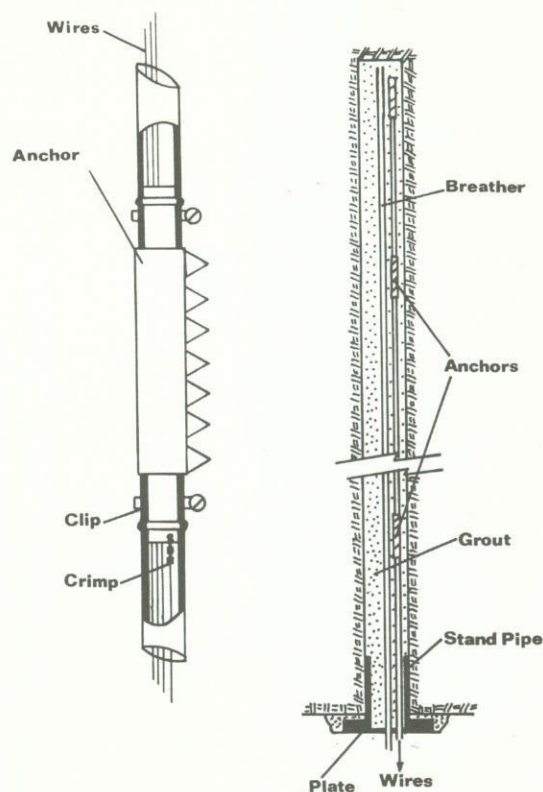


Figure 6 Multi-anchor strain wire extensometer

allowable deformation of a steel support is 2-3% radially with 6% being the absolute maximum if long term stability is to be retained (Figure 8). For concrete, the deformation value is 0.1 to 5% radially depending upon whether it is monolithic or segmental.

Careful thought must go into the planning of a deformation survey with respect to the required accuracy, number of observation points and time between readings. In the longwall coal mining situation there are two very different situations where deformation surveys are required and they are:

- (i) Gate Roadways,
- (ii) Major Access Tunnels.

Knowledge is required in many situations when the tunnel periphery has ceased moving so that any remedial work can be correctly phased. The majority of the movement in a gate roadway will occur within 250 m of the face line although there are some exceptions particularly in thick coal seams

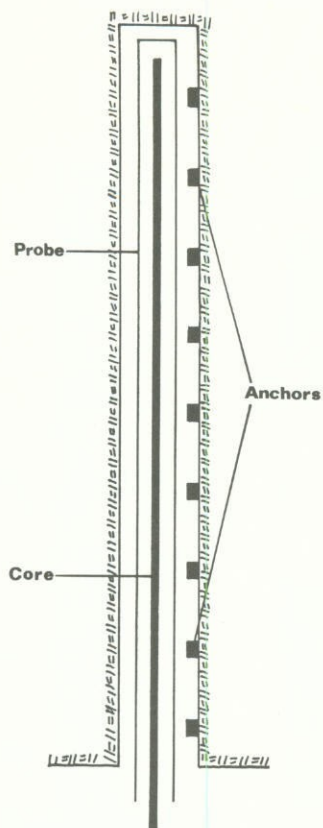


Figure 7 Magnetic probe type of extensometer

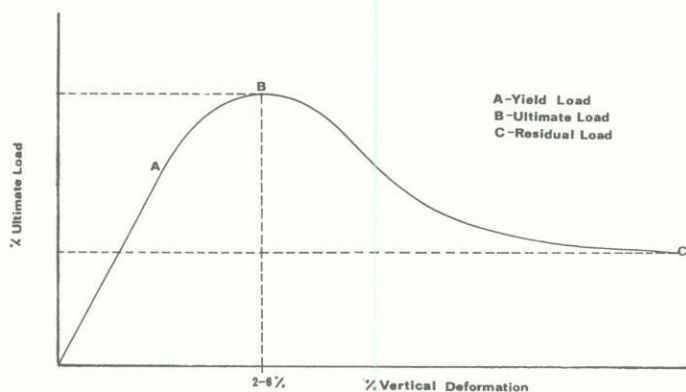


Figure 8 Load-deformation characteristic for rigid steel support systems

in deep mining conditions.

Measuring stations should be concentrated in the face area (0-100 m) and thereafter to 250 m outbye so that a complete picture of the deformation behaviour is obtained. The required accuracy of the measurements need not be too high and the order of 10-20 mm is often sufficient as the magnitude of the movements will be frequently as much as + 1 m. In general, simple taping methods taking height and width measurements will suffice. In-plane and out-of-plane leg angles also yield important information concerning the behaviour and effectiveness of a steel arch in a gate roadway.

Major access roadways are usually designed to have a working life of up to 50 years or more and contrasts markedly with that of a gate roadway of up to 5 years. The design approach is to implement a support system that will control the excavation such that its long-term stability is retained, thus virtually eliminating the need for repair work. However, the deformation to be measured will be in the order of millimetres or even centimetres and the rates of closure very low. However, failure can still occur and the need for monitoring of deformation is still as important as in the case of the gate roadway. The length of tunnel to be monitored may be up to 10 km, therefore measuring stations may be spaced at 100-200 m intervals, with the measuring scheme being accurate to $\pm 1-5$ mm.

Figure 9 represents a typical access tunnel, containing a large conveyor belt, false floor and pipe ranges, all of which impede the taking of measurements. A very accurate scheme as devised by Kovari and Amstad [13] could not easily be implemented in such a roadway. Nevertheless a method has been developed that allows the determination of the complete deflection and profile of the top half of the lining. A steel cable is placed across the tunnel as shown in Figure 9, under a constant tension. Offsets are taken using tapes at intervals along the line, defined by either marked changes in curvature or simply regular intervals. A smooth profile is generated by computer using this data, an example being given in Figure 10.

Support Loading

Load measurement at the support-strata interface, or in the legs of an arch support can be achieved by either of two distinct techniques. The first is the physical measurement of load directly, the second being the measurement of strain in a component of the support system and relating to load by previous calibration.

The direct measurement of load is achieved by cells placed between the two surfaces

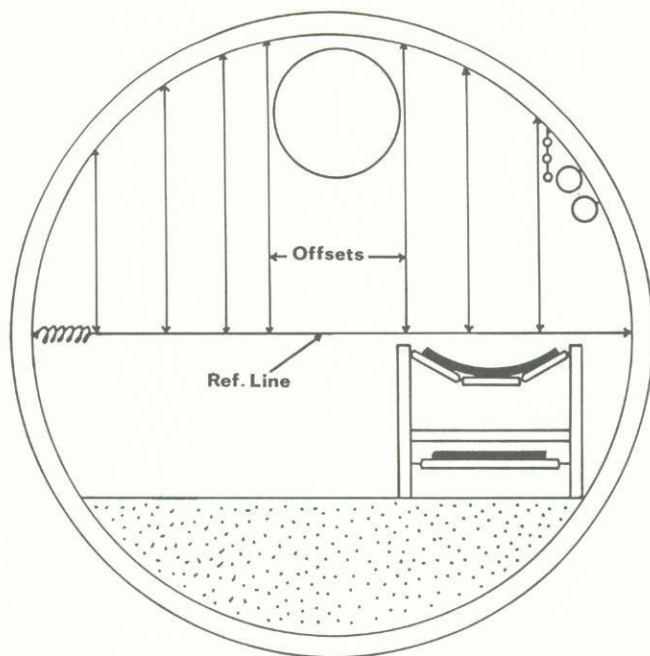


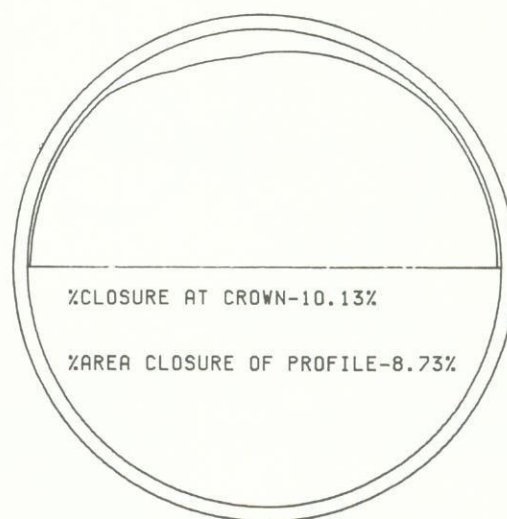
Figure 9 Typical access tunnel

where load is required to be known. As with extensometers there are several factors governing the choice of load cell in mines, these being:

- (i) total cost,
- (ii) nature of mining environment,
- (iii) type of load to be measured i.e. dynamic, static, compressive or tensile,
- (iv) accuracy required,
- (v) availability of instruments, recording systems and trained personnel. [15]

Consideration of all of these factors with respect to the difficult and hazardous mine environments has led to the extensive use of the hydraulic load cell. This type of cell relays the applied loads via a fluid, either oil or mercury, compressed during strata movement. Figures 11a and 11b show two commonly used hydraulic load cells. The mercury filled Glotz cell has been a popular choice in the U.K. but Whittaker concluded that its cost made it prohibitive when placing several on one support to measure load distribution [16]. As such the Nottingham University oil-filled cell was developed as a low-

GASCOIGNE WOOD MINE-SOUTH DRIFT
METRE MARK-6221
6X5-SUPPORTS
SQUARE STRUTS



MEASUREMENT DATES

— 12/6/86

Figure 10 Computer treated output of tunnel deformation data

cost form of instrumentation and has been used at many sites within the U.K.

The indirect method is in principle the reverse of the method of structural analysis whereby load is calculated from elastic displacement rather than the other way round. In order to successfully calculate loads, the elastic displacement must be measured very accurately which can pose problems in difficult mining conditions.

Strain gauges, mechanical, electrical resistance or vibrating-wire types can be implemented to evaluate the elastic strain. The mechanical and vibrating-wire types are noted for their robustness and reliability. The vibrating-wire system is particularly suited to measuring small strains as in a concrete lining. The strains induced in steel linings are generally much larger and has resulted in preference being given to the Demec system.

An effective technique, is the Demec system [17] which works on the principle of measuring the elastic shortening under load change. Figure 12 shows the Demec gauge

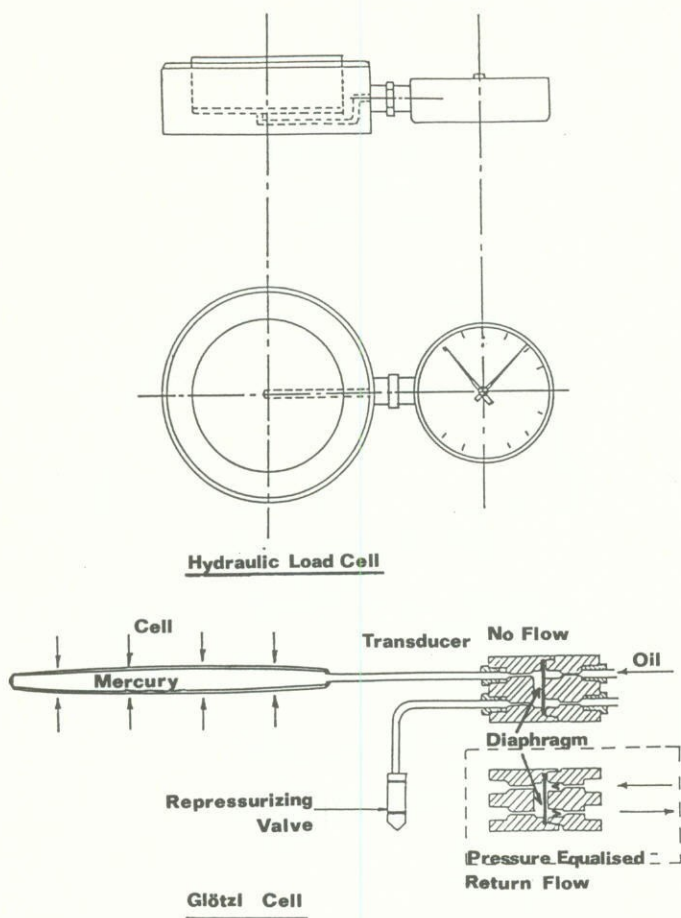


Figure 11 Cells for load determination on mine roadway support systems

along with the positioning of the targets on H-section steel supports. By measuring the gauge length and knowing the cross-sectional area of the element and its modulus of deformation the average load in the member can be calculated, providing the effects of bending moments are minimal.

All of these monitoring techniques have been successfully implemented at many locations in U.K. coal mines, ranging from gate roadways in thick coal seams to the major access tunnels at Gascoigne Wood Mine, Selby. The instruments have proved to be very reliable.

COMPARISON OF PREDICTED AND MEASURED TUNNEL BEHAVIOUR

The most important criterion of any stability prediction technique is the accuracy of its results in relation to a wide range of real situations. The prediction technique outlined earlier in the

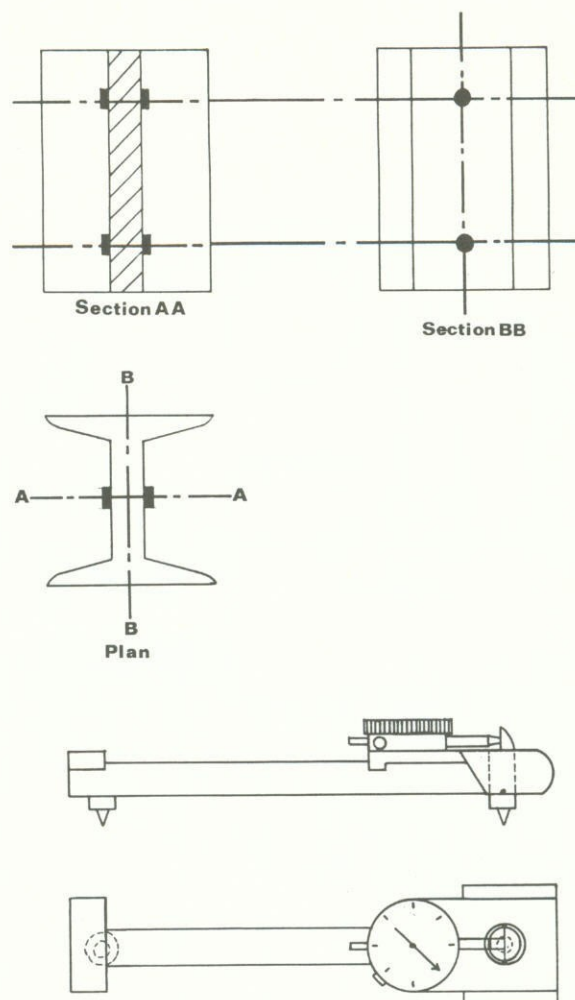


Figure 12 Demec gauge system for arch leg load determination

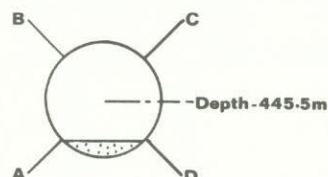
paper can only produce positive deformations, although in reality both positive and negative or out-of-plane deformations will occur due to the varying deformational properties of the strata around the tunnel periphery. Therefore, engineering judgement is required in the interpretation of the predicted results in relation to realistic support behaviour.

The prediction technique has been assessed using data obtained from the circular tunnel at Gascoigne Wood Mine, Selby, supported by steel rings of various section.

Kapusniak et al [18] undertook the first assessment whereby four multi-anchor, strain wire type extensometers were placed around the tunnel periphery. From these, data were obtained for the radial closure and extent of the Yield Zone, which was compared with the predicted values. Figure 13 is reproduced from their paper [18]. They concluded that the predicted and measured results compared favourably in that they were of the same order both for closure and

Yield Zone extent. It is also interesting to note that the predicted values are all slightly over-predictive, which is to be expected from the assumptions made in the analysis that

- (i) the rock mass is characterised by the post-failure laboratory strengths,
- (ii) the rock exhibits maximum bulking.



	Extensometer Data		Predicted Values	
	Closure (mm)	Yield Zone Extent (m)	Closure (mm)	Yield Zone Extent (m)
A	6	<1	19	1.022
B	5		14	0.743
C	2		14	0.743
D	9		19	1.022
Floor	-		20	-
Roof	-		13	-

Figure 13 Comparison of predicted and measured values of tunnel closure (Selby Coalfield) [18]

However, from a design consideration, this is acceptable. Kapusniak [19] went on to compare the predicted and measured deformations at 14 stations within the Gascoigne Wood Tunnel. Figures 14 and 15 show two of the comparisons with the measured data being in brackets. It should be noted that for measured values greater than 2-3% (5.8 - 8.7 cm), the support will be starting to exceed its maximum loading capacity. However, the prediction technique takes no account of this and in this instance will tend to become under-predictive.

Figure 14 is representative of the steel support still being within its elastic range and the compared values agree to ± 1.2 cm with the exception of one set. Figure 15 represents an area where the support has been loaded beyond its maximum strength and is now well within its plastic range, with reduced loading capacity. The prediction technique is now under-predictive as it is still applying the full support pressure to the rock mass. However, had the technique been applied at the design stage it would have indicated a potentially unstable area with insufficient support capacity.

Kapusniak [19] concluded that the study promoted confidence in the ability of the convergence prediction technique to esti-

mate differential radial closures for a circular tunnel in a stratified rock sequence.

This view is shared by the authors, and the technique has been used to assess the likely stability of the proposed pit-bottom tunnels at the new Asfordby Mine in Leicestershire.

PRINCIPAL FACTORS INFLUENCING LONG-TERM STABILITY OF MINE TUNNELS

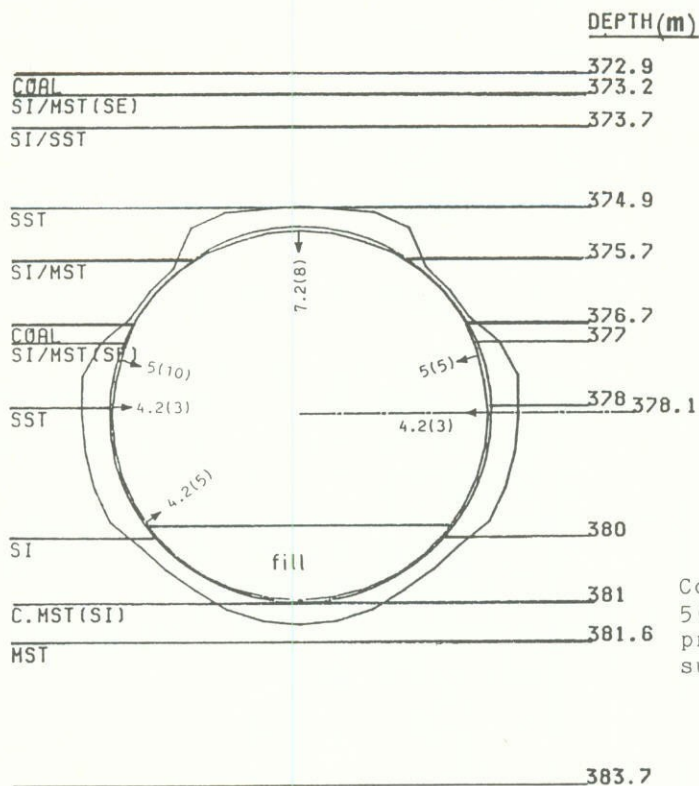
The design and implementation of the support system within a mine tunnel is a major factor influencing long-term stability. Geological and rock stress factors together within interaction from mining are also important. The rock mass properties are inherent to the design, with the only variable being the support itself. Thus every effort should be made at the design stage to select a support system that will promote long-term stability under the action of the rock mass, otherwise stability can be impaired during the working life of the tunnel with the possibility of costly repair work being needed.

Nevertheless, given that a satisfactory design is made, there are secondary external factors that can effect the support's long-term stability. Long-term weathering of the rock mass within the Yield Zone can cause a decrease in its cohesive properties leading to generation of a dead-weight acting on the support. A progressive build up of dead-weight can lead to major failure affecting a section of the tunnel. Steel support distortion usually precedes such occurrences and serves as a warning of the existence of such potential problems. Knowledge of the significance of distorted tunnel supports can be assessed by monitoring support deformation and comparing to pre-determined limits.

Corrosion is another factor that can be detrimental to tunnel stability in the long-term. Corrosive waters can lead to significant loss of supporting ability of steel mesh lagging, and steel girders can also be affected in time. Sprayed concrete is probably the most effective means of providing protection where appreciable corrosion of steel supports is anticipated.

CONCLUSIONS

- 1 Support systems should be designed following a detailed assessment of the tunnel geological factors, and in particular the stratified character of the rocks through which the tunnel is to be driven should be assessed in terms of both short- and long-term support requirements, and the effect of post-failure behaviour also

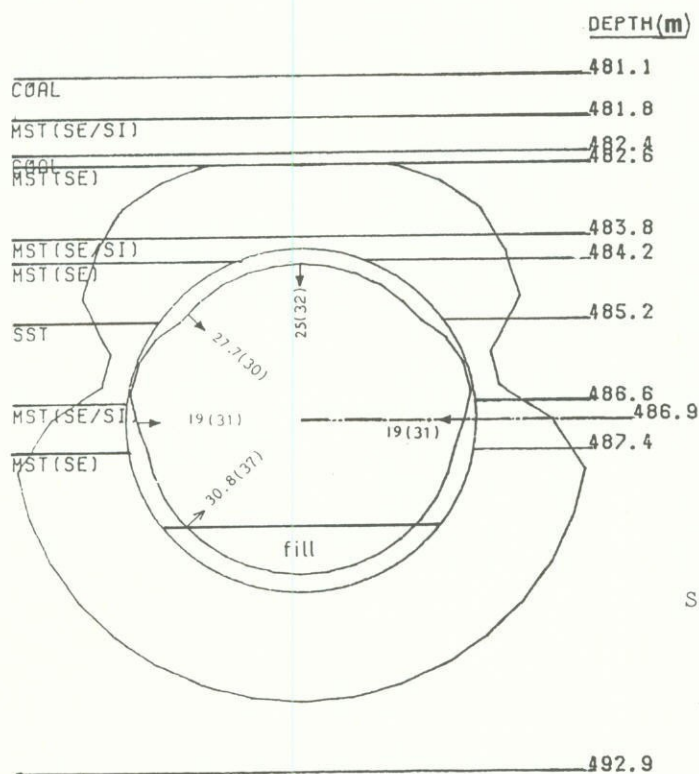


SUPPORT PRESSURE= .253MPa
 RADIUS OF TUNNEL= 2.9M

ANGLE (DEGREES)	RADIAL CLOSURE (MM)	EXTENT OF YIELD ZONE (CM)
0	72	320
10	72	320
20	79	331
30	42	331
40	18	297
50	17	297
60	17	297
70	48	321
80	62	335
90	42	331
100	42	331
110	42	331
120	42	331
130	29	315
140	24	314
150	24	314
160	31	320
170	38	323
180	38	323

Convention for predicted/measured closures:
 5(10)
 predicted (measured) radial closure (cm)
 support: 10 x 10 cm steel rings.

Figure 14 Comparison of measured and predicted closures under light loading conditions (Selby Coalfield)



SUPPORT PRESSURE= .4MPa
 RADIUS OF TUNNEL= 2.9M

ANGLE (DEGREES)	RADIAL CLOSURE (MM)	EXTENT OF YIELD ZONE (CM)
0	250	427
10	258	436
20	255	455
30	257	461
40	273	463
50	277	442
60	115	419
70	36	335
80	36	335
90	190	419
100	307	478
110	307	479
120	307	479
130	308	479
140	308	480
150	308	480
160	308	480
170	308	480
180	308	480

Support: 15 x 12½ cm steel rings.

Figure 15 Comparison of measured and predicted closures under heavy loading conditions (Selby Coalfield)

taken into account.

2 The prediction of tunnel closures should include stratification effects, and this can form the basis of determining the anticipated support strength to achieve stability against strata deformation, which is not necessarily uniform around the tunnel periphery.

3 The strength-deformation characteristic curve for mine tunnel supports should be used to judge permissible limits of tunnel deformation; guidance can be given from such characteristic curves on the deformation limits at which remedial action is required.

4 Instrumentation and monitoring continue to play a vital role in giving on-going assessments of tunnel stability, especially in comparing different lengths of the tunnel where changes in geology and support type may exist; monitoring stability changes is really the only effective means of providing a comprehensive assessment of the roles played by time-dependent closure and corrosion/weathering influences.

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ACKNOWLEDGEMENT

The research work described in this paper was carried out with the financial aid of the European Coal and Steel Community and that of British Coal, formerly the National Coal Board. The authors record gratitude to the ECSC and BC for the financial and technical support provided and to numerous engineers and scientists within British Coal for their useful comments and criticisms given to this work. The views expressed are those of the authors and not necessarily those of British Coal.

SUPPORT OF TUNNELS IN SOUTH AFRICAN GOLD MINES

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ABSTRACT

Tunnels in South African gold mines are developed at depths down to 3 600 m below surface where the virgin rock stress approaches 100 MPa and, on occasions, through rock where the field stresses exceed 150 MPa. In addition, many tunnels are subjected to increases in field stress of over 50 MPa during their useful lifetime. As a consequence, the walls of most tunnels are fractured causing the rock to dilate into the tunnel. Quantitative data on the extent of this fractured zone are presented. The function of support in these circumstances is to reinforce and maintain the integrity of the fractured rock. In 80 per cent of the 800 km of tunnels developed annually, the support methods used are capable of maintaining the stability of the tunnels. However, the remaining 20 per cent of tunnels is subjected to severer conditions where dilation in excess of 250 mm occurs and/or where the probability of strong seismic ground motion is high. In these tunnels, additional support is required. The type and pattern of support for both normal and severe conditions is described. In spite of the improved support used, damage to tunnels may occur. To minimize this damage, the concept of a rock reinforcing tendon that will yield and do work, when subjected to quasi-static or rapid deformation, has been put forward. Specifications for such a tendon are presented and the development of certain yielding tendons are described.

INTRODUCTION

Tunnels form the essential communication network between shafts and the stopping areas. Their function is to allow access for men and materials, to enable the transportation of rock out of the mine, and to allow passage of ventilation air and other necessary services such as water columns, backfill or grout ranges and power cables.

The results of a recent survey into tunnel development and methods of support in South African gold mines have shown that about 6 million rock tendons are installed in over 800 kilometres of tunnels which are developed annually (1).

This development takes place at depths below surface ranging from a few hundred metres to 3 600 m, and under a variety of geological conditions. The influence of geology on tunnel condition is, however, generally overridden by that of the high ambient stresses which exceed the strength of the rock, and thus the tunnels are enclosed by a zone of fractured rock.

Supporting this fractured rock constitutes the major support problem in gold

mine tunnels. The design of support systems for tunnels in deep, hard rock mines is based mainly on experience gained in the development of thousands of kilometres of tunnels over many years.

Tunnel support systems consisting of grouted tendons in combination with wire mesh and rope lacing are very effective in maintaining tunnels under most stress conditions including some rockbursts. However, under the severest conditions serious damage to tunnels can occur, often due to the stiffness and lack of yieldability of all the commonly used rock tendons.

This lack of yieldability has serious consequences in the support of tunnels where large rock deformations and/or rockbursts occur. The ability to yield and to do work is a major requirement for rock tendons under these conditions. Recognition of this has led to the preparation of specifications for yielding rock tendons to be used as a guideline for the design of such tendons. Modified tendons and end-anchors are being developed to conform to these specifications.

FACTORS INFLUENCING TUNNEL CONDITION

Geology

The goldfields in South Africa occur along the periphery of the Upper Witwatersrand sedimentary basin and all the major mines are situated down-dip of the outcrop or sub-outcrop of the gold bearing conglomerates. Figure 1 shows the position of the goldfields and indicates their average mining depths (2).

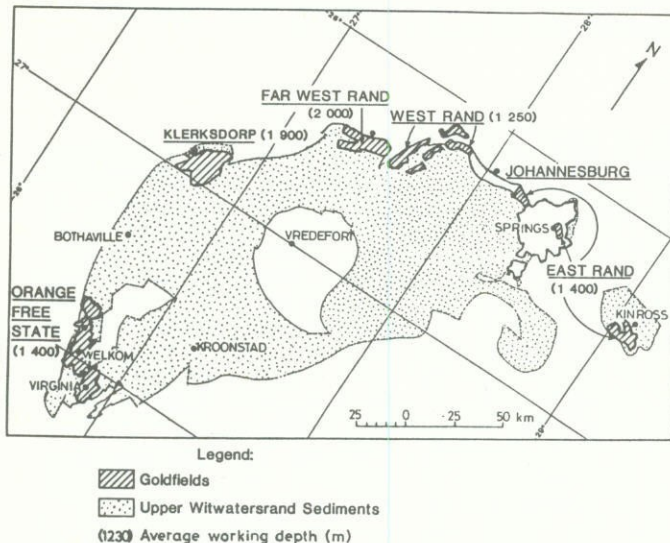


Figure 1 Map showing the position of the goldfields along the periphery of the Upper Witwatersrand Basin. Also indicated are their average working depths, in metres

Most tunnels in South African gold mines are driven through slightly metamorphosed, predominantly arenaceous sediments of the Witwatersrand Supergroup. The most common rock types are slightly argillaceous to argillaceous quartz waxes with uniaxial compressive strengths ranging from 120-200 MPa. Shaley parting planes are ubiquitous, most frequently occurring at stratigraphic intervals of between 0,3 m and 2,0 m. About 20 per cent of the tunnels are developed in hard, brittle ortho-quartzites and less than 10 per cent in shales, siltstones and lavas. Support problems caused by inherently poor rock quality are not common and are confined to certain areas and specific formations which have to be traversed. The frequency of structural weaknesses, in the form of faults, dykes and joints, is high in most areas. These features have to be well supported particularly in areas where the seismic risk is high.

Effect of Stress

Both the ambient state of stress exist-

ing in the rock at the time the tunnel is developed and subsequent stress changes brought about by mining in the vicinity of the tunnel are of prime importance in determining the condition of tunnels at depth and hence their support requirements.

Field studies indicate that damage to a tunnel in the form of extension fracturing and dilation of the wall rock, is initiated when the maximum field stress has a magnitude of $0,3 \sigma_c$, where σ_c is the uniaxial compressive strength of the rock. The support of tunnels becomes unduly costly when the stress exceeds $0,6 \sigma_c$. Since the uniaxial compressive strengths of most Witwatersrand rocks range between 120-250 MPa, tunnels developed at depths of 2 to 4 km in unmined ground are damaged on excavation, and support problems are experienced (3).

This is particularly important for tunnel development which takes place in areas where scattered mining methods are practiced and tunnels are excavated in advance of the area of active mining. These tunnels comprise 80 per cent of the total flat development carried out.

Typical stress changes to which a pre-developed tunnel, at a depth of 2,5 km, can be subjected during its overmining are illustrated in Figure 2.

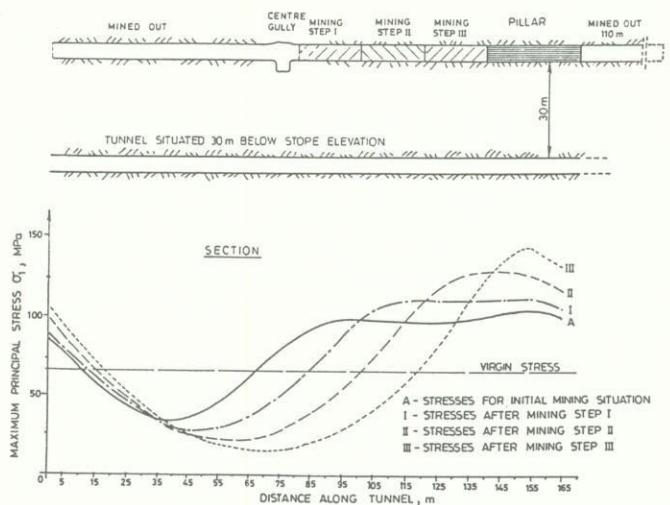


Figure 2 Variation of maximum principal stress along a pre-developed tunnel situated 30 m below the reef plane during overmining.

This figure shows how the maximum principal stress acting on a tunnel below the unmined ground between two stopes increases to more than twice the virgin stress as the size of the remnant pillar decreases and how subsequently the stress drops to well below virgin stress once the tunnel is overmined.

The stress levels are generally considered to be most important in influencing the design of the support system and stress analysis programs such as MINSIM-D (4) are run routinely to determine field stresses in areas with a complicated mining geometry.

About 20 per cent of tunnel development is associated with longwall mining where tunnels are developed in stress-relieved ground behind the mining face. These tunnels usually suffer little damage and support requirements are limited to maintaining the integrity of an excavation situated in slightly fractured rock. In the longwall situation tunnels are only subjected to high stresses when they have to be advanced ahead of the stope face or are situated within the zone of influence of regional stabilizing pillars or a siding abutment. In the latter cases the tunnels are subjected to high horizontal stresses resulting in damage primarily to the hangingwall. In both of these situations severe stress damage can be expected requiring particularly good support especially in seismically active areas.

However it is the support of the pre-developed tunnels in high stress conditions which requires most attention.

Stress related and other factors which lead to poor tunnel conditions are summarized in Table 1. This information was obtained during the survey on tunnel support practices in the different mining districts. In the Table the different factors are ranked in importance with respect to cause of tunnel damage.

Table 1 The relative importance, in per cent, of the different conditions which necessitate additional support.

Mining district Type of conditions	East Rand	West Rand	Far West Rand	Klerksdorp	Orange Free State
High initial field stress	9	10	14	7	6
High anticipated stress changes	73	15	34	33	40
Fault and dyke intersections	5	19	14	14	11
Poor rock quality	11	51	16	8	38
Anticipated rockburst conditions	1	3	22	38	3
Other	1	2	-	-	2

The table shows that the major problem area is the anticipated stress change

induced by mining close to tunnels. Poor rock quality, which results from low rock strength, and geological weaknesses, and the possibility of rockbursts are also important in specific districts.

Although fault and dyke intersections comprise only a small percentage of the length of tunnels driven, they are responsible for a significant proportion of the causes of poor ground conditions. These features, particularly in seismically active areas, are usually regarded as hazardous, requiring additional support.

DAMAGE TO TUNNELS

The major cause of damage to gold mine tunnels has been established as being directly or indirectly related to the high stresses acting on the wall rock of the tunnels causing intense and extensive fracturing and large dilations. Typical fracturing and dilation around a tunnel is shown schematically in Figure 3.

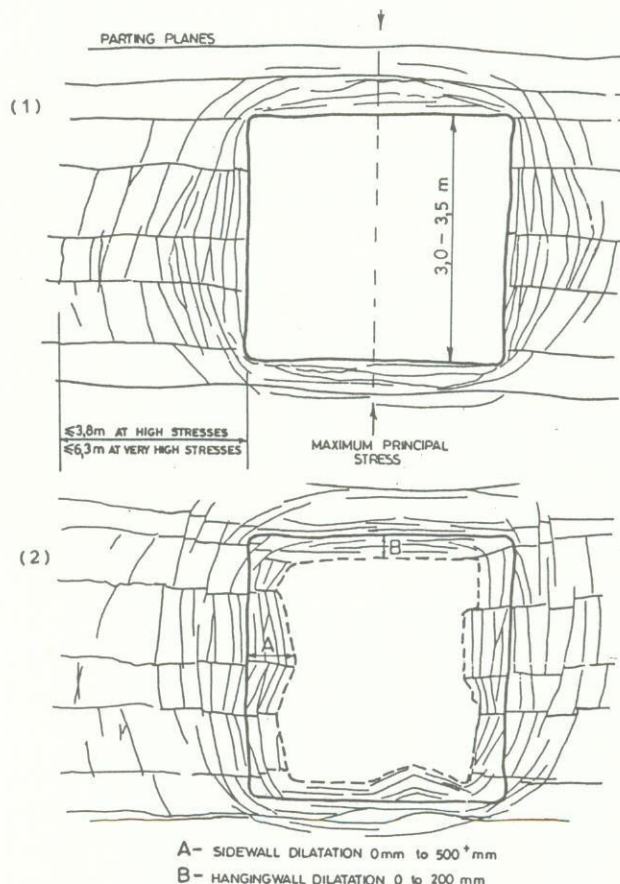


Figure 3 Schematic diagrams showing (1) typical depths of fracturing around tunnels subjected to moderate and high levels of stress and (2) typical deformation of tunnel walls.

The fractures are spaced 10 mm to 200 mm apart to form large, thin slabs of rock parallel to the walls of the tunnel. The height of the slabs is usually controlled by bedding planes and can vary from a few centimetres to over 2 m. The distance to which the rock is fractured into the sidewalls is typically 3-4 m in highly stressed tunnels but can extend to over 6 m. In the roof and floor, fracturing usually extends to less than a half of the distance observed in the sidewalls. The overall cross-sectional shape of the fracture zone is elliptical. Most fractures are extensional but occasionally shear fractures with displacements of up to 10 mm occur (5). The dominant mode of deformation of the fractured rock is by dilation towards the tunnel opening. Sidewall deformations of up to 200 mm are common and displacements of over 500 mm can occur. The rate of this movement is usually measured in mm/week in response to stress changes and growth of the fracture zone but can occur at more than 1 m/s when associated with the passage of seismic waves.

Buckling of thin rock slabs is another mode of failure which is sometimes observed (6). This type of damage is enhanced by the presence of weak bedding partings.

The stress induced damage thus transforms a competent rock into one which is highly fragmented and deformable.

To establish the proportions of the dominant types of damage to tunnels in the different goldfields, and hence where improvements in support methods could be developed, data was collected on the relative frequency of occurrence of falls from the hangingwall or sidewall, of excessive sidewall closure and of footwall heave in tunnels. Figure 4 summarizes this data.

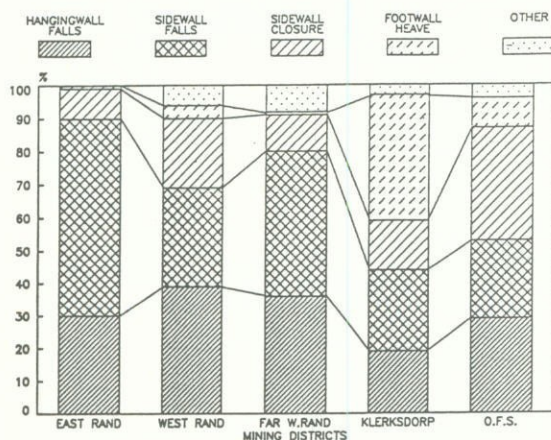


Figure 4 Proportion of the different types of damage observed in tunnels for the different gold mine districts.

The most common type of damage varies from goldfield to goldfield. Comparing the relative proportions of damage types in the figure it is clear that damage to the sidewalls of tunnels is far more common than falls of ground from the hangingwall. From observations and by relating data from Figure 4 and Table 1 it appears that sidewall falls and excessive closure tend to be associated with large stress changes, whereas in the West Rand and Orange Free State (OFS), poor rock quality is an important cause of damage, resulting in a relatively high proportion of hangingwall falls. The East Rand and to a lesser extent, the Far West Rand have a relatively high proportion of their tunnels in shale which also tends to aggravate hangingwall falls.

In the Klerksdorp district, footwall heave related problems are dominant, probably because a high proportion of tunnels are located in well bedded argillaceous quartz wackes.

THE FUNCTIONS AND REQUIREMENTS OF SUPPORT

The basic function of support in tunnels is to ensure that the tunnels remain serviceable during their required operational life, which may be tens of months or tens of years.

The requirements of the support are determined by the strength and nature of the rock through which the tunnel passes, the ambient field stress at the time of excavation together with the anticipated stress changes brought about by mining and, finally, by the probability of seismic activity in the vicinity. Various combinations of these factors lead to a wide range of conditions varying from good to severe. Experience gained in the development of thousands of kilometres of tunnels has led to an understanding of the support functions and requirements for a wide range of conditions. Examples of these are discussed below.

In the most favourable situations i.e. those found in shallow tunnels or in tunnels developed in stress relieved ground, the function of the support is merely to pin the occasional loose block to the competent rockmass. This is best achieved by the installation of end-anchored tendons such as rock studs which can be pretensioned to provide active support.

Under difficult or severe conditions the functions of the support system become more complex and the requirements more stringent. In the case of weak, friable rock or rock which is likely to deteriorate by weathering such as some

shales and dykes, the prime function of the support is to hold in place all the friable material by supplying maximum area support of the exposed rock and to seal it from the atmosphere. Shotcrete is eminently suitable for this function.

However if such a rock type were to be intersected where the field stresses are high and the tunnel walls likely to deform, shotcrete alone will be unsuitable as it will crack in response to the deformation. A flexible, full cover support is needed in such a situation and is best provided by a fine wire mesh held in place by full column grouted tendons.

Where a combination of joints and bedding planes result in blocky ground with blocks ranging in size from very large to small, conditions are particularly difficult for the installation of adequate support. The function of support is to hold all unstable blocks in place and to achieve this as close to the face of the tunnel as possible so as to avoid rapid deterioration in conditions. Rockbolting with mesh and wire lacing is suitable as are steel arches or timber sets. Where large deformations of the tunnel walls are expected the stiff steel arches and timber sets are unsuitable.

In all the examples so far discussed support of the hangingwall has been the main problem. More commonly in deep gold mine tunnels where extensive fracturing occurs, the main problem lies in the support of the sidewalls and top corners of the tunnel. The main function of support in highly fractured rock is to reinforce the fracture zone by rockbolting and by so doing, to assist the rock to support itself. This is achieved by friction between slabs and by building up confining forces in the rock normal to the tunnel wall and thus inhibiting further fracturing and dilation. Where the intensity and depth of fracturing is moderate and the anticipated stress changes low, a pattern of tendon support is often found to be suitable.

However, where conditions are more severe, with a greater depth of fracturing and a propensity for dilation, tendon support alone is inadequate as the rock slabs held in place by the tendons are broken by dilatationary forces between the tendons. These smaller rock slabs are then forced inwards towards the tunnel void and appear to bulge around the anchored tendon. An added function of the support under these conditions is to prevent fallout and to control the deformation of the rock slabs between tendons. This is achieved by installing wire mesh reinforced and held in place by steel cable lacing. In this type of support system, a major portion of the

support effect is often supplied by the wire mesh and lacing. The tendons primarily anchor the lacing to the rock and reinforce the fractured rock deeper into the sidewall. An added advantage of the mesh and lacing is that it imparts some degree of yieldability to the support system, a function which is essential under conditions where large dilation occurs. It has been found that it is not necessary for the tendons to be anchored in solid rock, though around certain particularly important excavations, long 1 000 kN cables are used.

In rockburst conditions or where large wall rock deformations occur, tendons should have the ability to yield and do work during either slow or rapid yielding. The requirements of such a tendon are discussed in a later section.

TYPES OF SUPPORT IN USE

Of the almost 6 million tendons installed in flat-end development per year 82 per cent are grouted. Slightly more than half the grouting is done using cementitious capsules.

The percentage of the different types of tendons installed is shown in Table 2.

Table 2 Proportions of the different types of tendons used

Type of Rock Tendon	% of Total Tendon Usage
Shepherd's Crooks (grouted)	66
Steel ropes of different types (grouted)	14
Rebars (grouted)	2
End-anchored rock studs	14
Split Sets	3
Mechanically anchored cables	1
Total	100

Load-deformation curves of 2 m long, end-anchored rock bolts and grouted ropes are shown in Figure 5. From this figure the failure strength of these tendons is seen to be about 100 kN while the elongation or displacement at failure varies from 20 mm in the case of mechanically end-anchored tendons, to 50 mm in the case of the full column grouted tendons (7).

Large, asymmetrical deformations of tunnel walls preclude the use of yielding arches and timber sets for normal tunnel support in deep mines, mainly due to their inability to withstand even moderate lateral deformations. This type of

support is therefore seldom used in deep gold mines.

Fully grouted tendons are preferred to mechanically anchored tendons because of their better yieldability characteristic and because of the difficulty of installation of end-anchored tendons in highly fractured rock where deformation of drill holes is a common feature.

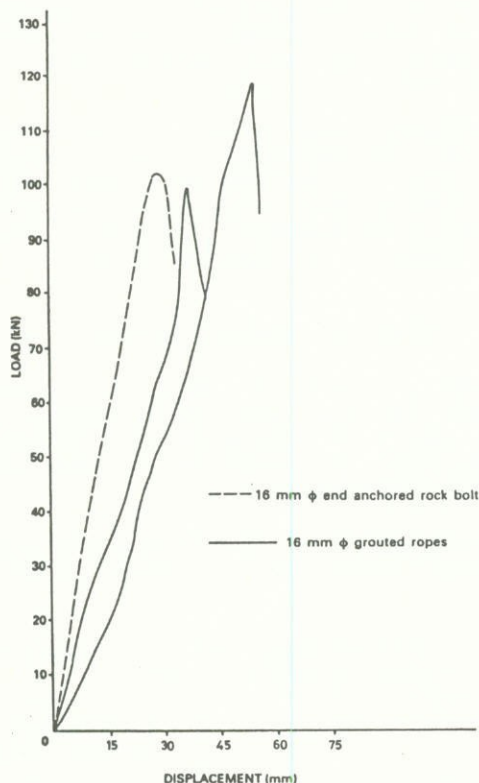


Figure 5 Load-deformation characteristics of different support tendons.

TUNNEL SUPPORT PRACTICE

Tunnel support practice has improved considerably in the last two decades and it is evident that highly stressed excavations can be safely supported for long periods.

The conditions under which standard support systems, in the form of mechanically end-anchored or fully grouted support tendons such as studs, Shepherd's Crooks, steel ropes or rebars integrated with a system of rope lacing and wire meshing, perform successfully are called 'normal conditions'. These include tunnels where the depth of fracturing can be in excess of 3 m and where sidewall dilation is up to 250 mm.

'Severe conditions' are defined as those conditions where large rock deformations (>250 mm) combined with extensive fracturing to depths of over 4 m occur or

where geological conditions are particularly complicated and where rockbursts are likely to occur. Where such situations are identified, additional support of a higher standard is installed.

Support for Normal Conditions

Tunnel support is normally installed in two stages : primary (temporary) support which is installed by the miner as close as possible to the face of the advancing development end, and secondary (permanent) support which is installed by specialist support crews sometime later, depending on mining and geological conditions.

Primary support is installed on average 2,4 m behind the face, varying from 0,5 to 6 m, while the closest distance for the installation of secondary support is about 13 m. At times the installation of secondary support may be deliberately delayed so that it is only installed shortly before stoping in the area begins. This deliberate delay is intended to overcome the problem of corrosion of support elements.

The proportions of the different types and lengths of support tendons used for primary/secondary support are summarised in Table 3 for normal conditions.

Table 3 The proportions of the different types and lengths of support tendons used for primary/secondary support.

Type of Rock Tendon	Length (m)	% Total Tunnel Dist. Supported
Shepherd's Crooks	1,5 - 2,2	28,6
	2,3 - 3,4	27,0
Fully grouted steel ropes	1,5 - 3,2	20,4
Grouted rebars	1,8 - 2,3	3,0
End-anchored rock studs	1,2	< 1,0
	1,5 - 1,8	18,0
Split Sets	1,8	1,0
Mean length	2,1	

Grouted Shepherd's Crooks and grouted steel ropes varying in length between 1,5 m and 3,2 m are the most commonly used tendons followed by 1,8 m long end-anchored rock studs. Split Sets are used on 3 mines only, primarily as a temporary support during the tunnel development stage and are often only installed in potential problem areas. No account is taken of the Split Sets when secondary tunnel support is installed as they are susceptible to corrosion.

Of the total flat development carried out in South African gold mines:

- 58 per cent has secondary support installed as standard practice;
- 33 per cent has secondary support installed only where poor conditions justify its use;
- 8 per cent has no secondary support; in these cases primary support is considered to be sufficient, and
- 1 per cent has no standard support because of shallow mining and good, competent rock conditions.

The primary and secondary support patterns are usually designed to combine in a specified manner to produce an overall integrated support pattern, an example of which is given in Figure 6. Similar tendon patterns with basic spacings of 1,5 m and 1,0 m are also commonly used.

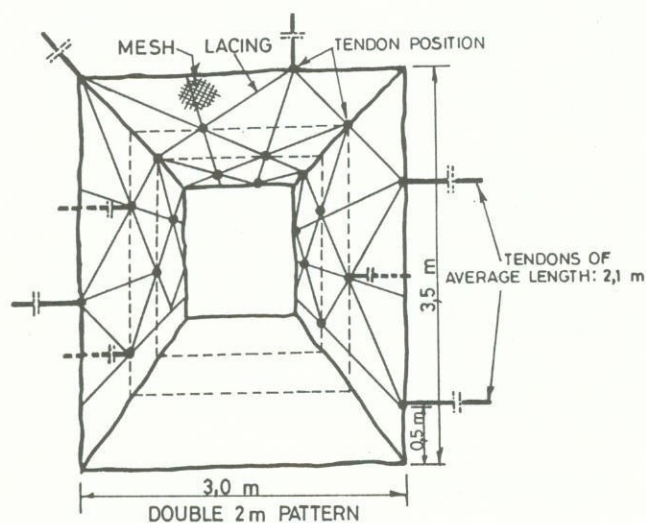


Figure 6 Diagram showing the basic 2 m and double 2 m support and lacing patterns.

Support for Severe Conditions

Severe conditions are encountered in approximately 170 kilometres of tunnels developed annually. This comprises 21 per cent of the total length of gold mine tunnels developed annually. The additional support required for these conditions may be installed either at the same time as the secondary support or at a later date when conditions start to deteriorate.

The proportions of the different types of support used as well as the range of tendon lengths used in severe conditions are given in Table 4.

Table 4 The proportions of the different types and lengths of support tendons used for severe conditions.

Type of Rock Tendon	Length (m)	Frequency of Length Range %	% of Total Additional Support Installed
Shepherd's Crooks	1,8 - 2,3	61	25,7
	2,5 - 3,4	39	16,5
Long, fully grouted end-anchored tendons	3 - 4	35,6	3,3
	5 - 6	61,1	5,7
	8 - 10	3,3	0,3
Grouted rebars	2,3 - 3,4	100,0	3,5
Grouted steel ropes	2,3 - 2,4	80,7	33,2
	3,2 - 5 m	6,3	2,6
	6,0 - 8,0	13,0	5,4
Anchored and grouted rock studs	1,8 - 3	100,0	3,8

This Table illustrates that grouted ropes and Shepherd's Crooks of length between 1,8-3,4 m are by far the most commonly used support units in severe conditions. The mean length of the additional support installed is 3 m, which is approximately 50 per cent greater than the length of tendons used under normal conditions.

It was also found that:

- tendons of length > 10 m are very seldom used (less than 1 per cent of long tendon usage)
- mesh and lacing is used in 82 per cent of cases where severe conditions are encountered
- lacing only is used in 11 per cent of flat development under severe conditions
- in 15 per cent of flat development the same pattern of support is used for normal and severe conditions. In these cases the support density, expressed in number of support units per square metre of tunnel wall, remains the same,

but the length of the tendons is increased and additional lacing and/or heavier mesh is installed.

Figure 7 compares the proportions of the different types of support used for normal and severe conditions. It is seen that there is a significant increase in the usage of fully grouted support tendons in the case of severe conditions.

For example, usage of grouted steel ropes increases from 20,4 per cent of the total tendons used under normal conditions to 41,2 per cent under severe conditions. Also, long cable tendons are introduced and end-anchored tendons are always fully grouted. Split Sets are generally not used for severe conditions.

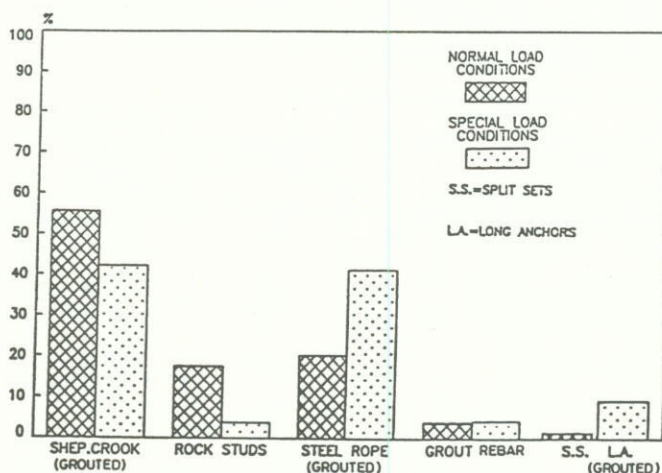


Figure 7 Proportions of the different types of support used for normal and severe loading conditions.

Despite the installation of improved support systems for severe conditions, including higher support densities and, on average, longer grouted tendons integrated with wire mesh and rope lacing, serious damage to tunnels nevertheless occurs. The survey shows that at least 160 km of tunnels developed in the preceding 10 years have to be re-supported each year. This is equivalent to about 20 per cent of the total flat development carried out each year.

The main reasons for this appear to be the limited yieldability in the support systems and, on occasions, the result of corrosion of the support elements.

SUPPORT DENSITY

Support Density for Normal Conditions

The density of support tendons installed in gold mine tunnels varies in accordance with the severity of conditions.

Taking into account the typical cross-section dimensions of these excavations (3,0 m x 3,0 m) and the fact that the footwall is generally not supported, the combined primary/secondary support densities, expressed in the number of support tendons per square metre of tunnel wall, are as follows.

No. of tendons/m ²	% of total flat development
0,6 - 0,7	49,0
0,8 - 0,9	22,0
1,0 - 1,1	29,0
Mean 0,8	100,0

It is evident that the most common support density range for normal conditions is 0,6 - 0,7 tendons/m².

Support Density for Severe Conditions

To control large deformations of the fractured zone around the tunnels and maintain the stability of these excavations under severe conditions, higher support densities than those used under normal conditions are generally employed. On most mines one or more standard support systems designed for severe conditions are available.

Once a section of tunnel is declared to be in a potentially high stress region the additional standard support is installed. In only about 10 per cent of tunnels encountering severe conditions are specific support designs made for each particular situation.

The following table summarizes the standard support densities for severe conditions.

No. of tendons/m ²	% of total flat development under severe conditions
0,7 - 0,8	9,0
0,9 - 1,1	22,0
1,5 - 1,9	60,0
Mean 1,3	

No standard support density is used for the remaining 9 per cent of tunnels. Each situation requiring additional support is assessed separately and recommendations as to support type and density are made by the Rock Mechanics personnel.

The support densities used for severe conditions are on average 60 per cent higher than those used under normal conditions but on occasions can be more than double the normal support density.

Figure 8 shows the theoretical relationship between support density and support resistance for 16 mm and 20 mm

high tensile steel tendon support with a failure stress of 600 MPa, assuming a linear load-deformation characteristic for the tendon, and a load capability equal to its failure load. Also shown in Figure 8 are the ranges of standard support density for severe conditions.

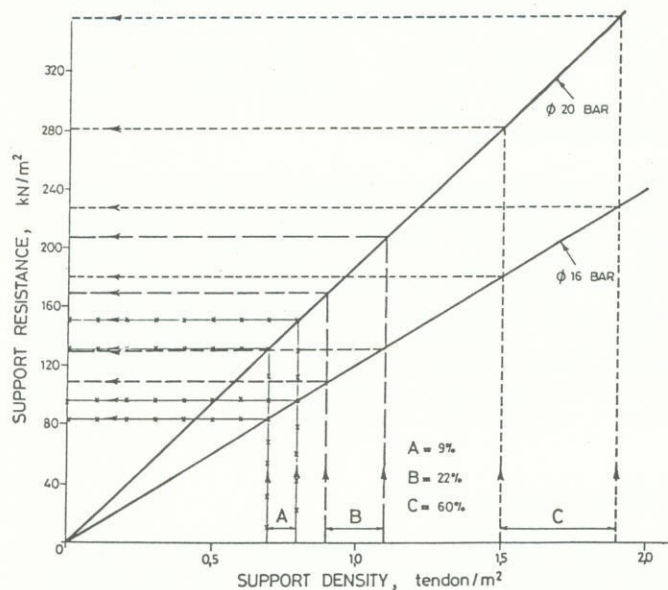


Figure 8 The correlation between support density and support resistance for support layouts using 16 mm and 20 mm diameter rock tendons. A, B and C indicate the standard support densities for severe conditions.

A theoretical assessment of the effect of support resistance on the areal extent of tunnel damage caused by seismic events, ranging in magnitude from $M_L=2$ to $M_L=5$, showed that little improvement was gained by increasing the support resistance higher than 200 kN/m² when supporting a thickness of fractured rock of 1 m or less. However, the extent of tunnel damage increases significantly if the support resistance is less than 50 kN/m² (6).

Approximately 67 per cent of the tendons supplied to the gold mines are between 14.5 mm and 16 mm in diameter. Thus it is seen from Figure 8 that the majority of tunnels subjected to severe conditions are supported close to the optimal theoretical resistance of 200 kN/m² and that all the support is installed at a resistance well above the theoretical minimum value of 50 kN/m². The occasional failure of the system can be attributed to limited yieldability of the tendons or exceptionally difficult conditions mainly caused by rockbursts.

The Use of Wire Mesh and Rope Lacing in Tunnel Support

Maintaining the integrity of a typical gold mine tunnel surrounded by a zone of fractured rock generally requires the application of wire mesh and rope lacing, sometimes together with shotcrete.

Mesh and lacing used either together or separately, are common elements of tunnel support systems in both normal and severe conditions. This is installed in addition to the rock tendons thus creating an integrated support system which can limit the amount of damage to the tunnels.

Figure 9 shows an example of a tunnel where tendon support is integrated with wire mesh and rope lacing to produce a permanent support system. It should be noted that the lacing of the sidewalls is kept separate from that of the hanging-wall. This pattern is used to ensure that deformation of the one wall does not lead to loosening of the lacing on the other.



Figure 9 A typical deep mine tunnel supported by grouted tendons, mesh and lacing.

In areas where the rock in tunnel walls is slabbled but not severely broken by bedding planes, lacing in conjunction with the rock tendons can be used to maintain stability of the tunnel.

In some cases, where the surrounding rock is initially intensely fractured but further fracturing and large rock deformations are not anticipated, mesh only is sometimes used with rock tendons. However, in more than 80 per cent of tunnels where severe conditions are encountered, both wire mesh and lacing are employed.

A range of combinations of wire mesh sizes and rope lacing are used on the gold mines. Mesh size and wire diameter range from 50 - 150 mm and from 3 - 6 mm respectively, while lacing rope diameter ranges from 8-25 mm.

FUTURE DEVELOPMENTS

An analysis of the available data on support elements and support systems employed in gold mine tunnels indicate that the main aspect of tunnel support design which requires attention is a need to improve the capacity of support tendons to yield, particularly in rockburst-prone areas (7,8,9,10).

There are several possible ways to improve the yield characteristics of rock tendon support (7), namely, (i) to use a ductile material for the manufacture of the tendon itself, (ii) to allow the tendon to move relative to the walls of the borehole at high loads; this can be achieved either by a controlled breakdown in bond between the support tendon and the surrounding grout material, or by some yielding mechanism either along the tendon or in the end-anchor, (iii) to employ a collapsible washer outside the borehole, or (iv) to employ a friction type clamping mechanism at the outside of the borehole.

Whatever means is used for attaining a yield capability in tendons, the resultant performance of the tendon under slow or rapid yielding must conform to a set of minimum specifications (12). These specifications and the factors used to determine them are discussed below.

PARAMETERS INFLUENCING THE DESIGN OF YIELDING TENDONS

Rock Condition and Required Support Performance

Any discussion on adequate support should take into consideration the expected total rock movements under 'static' and rockburst loading conditions. This movement may exceed 500 mm of sidewall dilation. However, for design purposes the total useful yielding range of a rock tendon should be limited to 500 mm as any further deformation of the sidewalls of a tunnel renders the excavation unusable. Once this stage in deterioration has been reached, rehabilitation of the tunnel has to be undertaken.

An important consideration in the design of a yielding tendon is the dilation distribution within the fractured zone and its relation to the length of the tendon. Specifically, how much of

the dilation, which defines the yieldability or strain required from tendons of different lengths, is expected to take place within the thickness of rock being supported. Typical proportions of the dilation distribution observed in a highly stressed tunnel between 0-2 m, 2-3 m and >3 m into the sidewall are approximately 46 per cent, 16 per cent and 38 per cent, respectively, of the total dilation (3). These values can be used in determining the specifications for different lengths of tendon.

Another important criterion is the thickness of the rock strata to be supported. Not enough is known about the mechanisms of rockbursts or what thickness of fractured rock is accelerated during a seismic event and has to be retarded by the support. However, the basic assumptions made here, namely, 0,95 m of hangingwall strata and 2 m of sidewall, compare with the 1 m or less assumed by Wagner (7).

Seismic Parameters

Peak ground velocity is the seismic parameter that has been used to calculate the kinetic energy imparted to the rock by a seismic event. Peak ground velocities are dependent on the magnitude of the causative seismic event and the distance from the event.

From analysis of the available data on this parameter, it would appear that velocities in excess of 3 m/s are rare away from the immediate source area of an event and can be expected to be associated only with events of magnitude M_L 4,0 or larger. This velocity has been used as an upper limiting value for calculating the work required to be done by a tendon during the absorption of the kinetic energy imparted to the fractured rock.

Capacity of Tendons to do Work

In order to reduce damage in a tunnel subjected to a rockburst the support has to do work to arrest the accelerated rock slabs surrounding the tunnel.

It is considered therefore, that the most important design parameter for a yielding rock tendon is the amount of work that must be done by the tendon during yielding. Assuming a support density of one tendon per square metre of tunnel, a work capacity of >25 kJ was arrived at from the following calculated values:

- i) the kinetic energy of a 2,0 m thick slab in the sidewall accelerated to a velocity of 3 m/s by a seismic event, is less than 25 kJ,

- ii) the sum of the change in the potential energy and kinetic energy of a 0,95 m thick slab in the hanging-wall, which is accelerated to a velocity of 3,0 m, is less than 25 kJ.

Other factors considered were that the maximum tolerable dilation of a tunnel sidewall is 500 mm, and that the minimum quasi-static yield load required to limit normal tunnel damage is 100 kN, which is the approximate strength of currently used tendons. This allows the dynamic yield load to be as little as half the quasi-static load and still enable 25 kJ of work to be developed.

SPECIFICATIONS FOR YIELDING ROCK TENDONS

The specifications have been prepared as a guideline for the design and manufacture of yielding support, primarily for service excavations subjected to high stress and/or rockburst conditions. Final specifications will only be made once a fuller understanding of rockbursts has been obtained.

These specifications are:

- (1) Amount of work to be done by yielding tendon : > 25 kJ.
- (2) Static yield load : > 100 kN.
- (3) Mean dynamic yield load : > 50 kN for tendons which can yield by 500 mm. Proportionately higher yield loads are necessary for tendons with lower yield capacity.
- (4) Maximum yield range : 500 mm.
- (5) Required yield range due to the dilation distribution within the supported rock thickness:
 - 230 mm, for a tendon length $\ell < 2$ m
 - 310 mm, for a tendon length $2 < \ell < 3$ m
 - 500 mm, for a tendon length $\ell > 3$ m
- (6) Strength of material under dynamic loading to be higher than the static yield load of the tendon by at least 25 per cent, to allow for transient shock loads.
- (7) The tendon must be able to yield initially by a certain amount under quasi-static loading, then yield under dynamic loading after which it should retain its load bearing capacity to yield further under quasi-static loading.

DISCUSSION

The combinations of yield load and displacement which would satisfy the design specification of work done equal to 25 kJ are depicted in Figure 10 - Curve A.

It is apparent that tendons with a wide range in load-displacement characteristics could fulfill the requirements, and that some types of tendons would tend to

fall in the lower range of the acceptable yield loads thus necessitating larger displacements to achieve 25 kJ work output. The disadvantage of low, 50-60 kN, dynamic yield load tendons is that any dilation that occurs prior to a rockburst proportionately reduces the capacity of the tendon to resist a rockburst with the result that the 500 mm maximum limit to the displacement allowed could be exceeded.

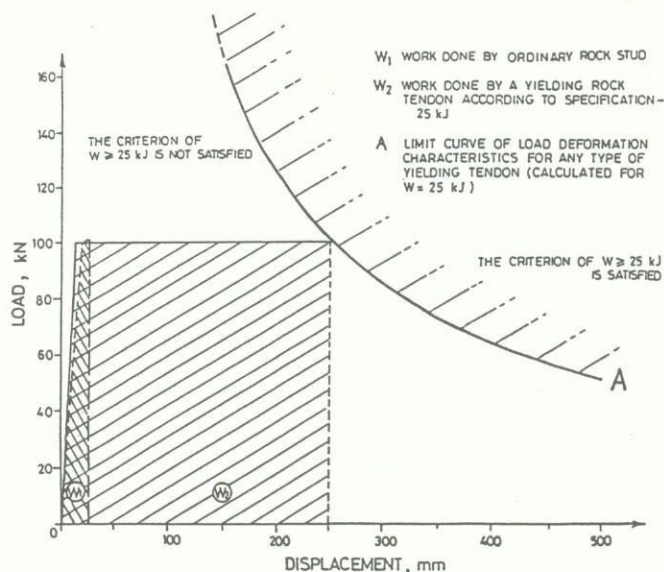


Figure 10 Load-displacement curve demarcating the boundary condition at which the work criterion $W=25$ kJ is satisfied. W_1 and W_2 indicate the work done by a conventional rock-stud and a yielding tendon respectively.

Other tendons using steels or alloys exhibiting limited but sufficient ductile behaviour to satisfy the work criterion would have relatively high yield loads and fall on the upper portion of curve A. A disadvantage of such a tendon is that, if it were subjected to moderate, stress induced dilation prior to a rockburst, it could be brought close to failure and would therefore probably fail during a rockburst. The implication is that tendons of this nature should be of limited length and not be anchored in the deeper unfractured or slightly fractured rock so that they are not subjected to the total dilation of the rock. Another potential problem with such a design is that most metals tend to deviate from homogeneous yielding at high strain rates.

The potential disadvantage of an end-anchored yielding rock tendon is its relatively greater vulnerability to shear movements in the rock traversed by the tendon and small tolerance to drill hole deformation at the time of installation.

Taking into account current technology and materials there are three possible combinations of end anchored/bar systems. These are discussed below.

1. End-anchored yielding tendon consisting of a yielding end anchor attached to a high tensile bar. Static and dynamic yield load of the bar is much greater than that of the anchor.

In this case only the anchor will yield under both static and dynamic conditions. The bar is protected from the shock load by the anchor. The disadvantage of this system is that if the anchor is prevented from moving due to distortion of the hole, either the anchor or the bar will fail when subjected to large dynamic loads. These types of tendons should be anchored in the unfractured rock so that when yielding the tendon does not encounter any shear displacements of the borehole.

2. End-anchored yielding tendon consisting of a yielding anchor attached to a semi-ductile bar. Static and dynamic yield load of the bar is slightly greater than that of the anchor.

In this case the anchor will yield under static conditions until jammed by an obstruction in the hole. The load will then build up slightly to reach the yield load of the bar which will then yield statically.

In the event of a subsequent rockburst the tendon will probably fail because of the intense heat generated within the yielding portion of the bar.

3. End-anchored yielding tendon consisting of a yielding anchor attached to a semi-ductile bar. Static yield load of the bar is slightly lower than that of the anchor; the dynamic yield load of the bar is slightly higher than that of the anchor.

In this case, under static loading, the bar will yield preferentially to the anchor thus eliminating the possibility of the anchor becoming jammed.

During dynamic loading the anchor will yield in preference to the bar thus protecting the bar from the shock loadings.

The only instance when this design will fail is when an obstruction in the hole occurs along the section traversed by the anchor, during the dynamic displacement. It is clear that if shearing of boreholes is a serious problem options 2 and 3 are to be preferred to

option 1. Moreover option 3 has certain advantages over option 2.

The specification that the strength of the tendon should be 25 per cent greater than the static yield load is derived from rapid loading laboratory tests. These showed an initial transient increase in load of up to 25 per cent at the onset of the rapid load before dropping to the dynamic yield load. The maximum displacement rate used in the tests was 1,7 m/s. These tests were carried out on a yielding anchor type tendon. With a system having a higher yield load, higher initial shock loads may be experienced. Tests to determine the importance of this phenomenon should be carried out on all new designs of yielding tendons.

The work done by conventional rock studs prior to failure is depicted in Figure 11 (W_1). This work amounts to less than 2 kJ for a 16 mm diameter, 2 m long tendon. The work done by the proposed yielding anchor (W_2) is thus more than twelve times that of the rock stud.

Yield prior to failure of this type of rockstud is about 30 mm. The capacity of the yielding anchor exceeds this figure by a factor of sixteen.

Different methods for improving the yield characteristics of rock tendon support, have been attempted (12,13) with varying degrees of success.

The Chamber of Mines Research Organization is at present pursuing ways of achieving improved yieldability by allowing the tendon to move at a controlled rate through the borehole while maintaining a constant resistive force. This is being attempted in two ways - for grouted tendons, by altering the shape of the tendon to modify the bond between the tendons and surrounding grout, and for non-grouted tendons by developing a yielding mechanism in the end-anchor attached to the rock stud.

Following these approaches prototype tendons have been developed. The performance of these tendons at slow and rapid yield rates is shown in Figure 11.

A number of variations of a grouted tendon known as a 'pinch-flat' have been tested. These show an improved yielding capability and reasonable dynamic yield forces (Figure 11 curve 3).

An improved, expanding end-anchor has also been devised and manufactured from a high modulus rubber compound. The frictional contact between the rubber

anchor and the sidewall of a borehole permits the tendon to move rapidly through the borehole without an excessive drop in the resistive force (Figure 11 curve 2).

A promising end-anchored, yielding rock tendon is being developed in collaboration with a support manufacturer which takes the form of a deformable element enclosing the tendon inside the drill hole. This tendon has exhibited encouraging load-displacement characteristics under slow and rapid yielding conditions (Figure 11 curve 4).

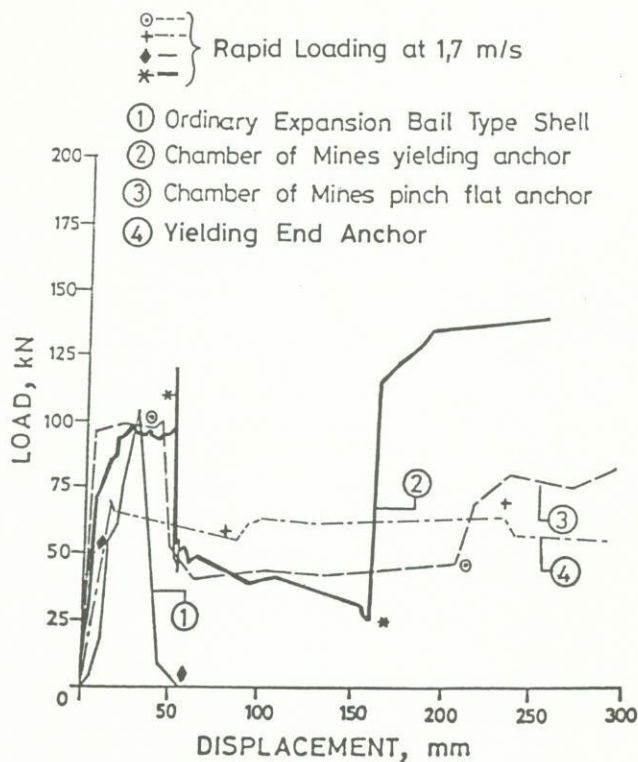


Figure 11 Load-displacement characteristics for different rock tendons under static and dynamic loading conditions.

However, a problem which is common to end-anchored-type tendons is the difficulty of installation into holes drilled in fractured rock, particularly where deformation of the drill holes occurs.

CONCLUSIONS

The main requirement of tunnel support systems in deep mines is to ensure the stability of a tunnel under both quasi-static and rockburst conditions, by reinforcing and maintaining the integrity of the fractured, deforming rock surrounding the tunnel.

The conventional support practice of primary and secondary support systems

comprising grouted tendons, wire mesh and rope lacing, meets the required support needs under most conditions. However, during rockbursts, ground velocities of several m/s may act at the tunnel walls imposing large deformations on the fractured rock. To control these movements tendons capable of yielding and doing work in excess of 25 kJ are required.

Research into ways in which the yielding characteristics of support systems can be improved as well as providing guidelines for the design and manufacture of yielding rock tendons is being actively pursued by the Chamber of Mines of South Africa Research Organization in collaboration with support manufacturers. A number of potentially viable yielding tendons have been developed and are currently being tested.

ACKNOWLEDGEMENTS

This work forms part of the research programme carried out by the Chamber of Mines of South Africa Research Organization. The authors would like to acknowledge the assistance given by mine management and rock mechanics engineers in making available unpublished data.

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THE STRESS MEASUREMENT AND UNDERGROUND ENGINEERING

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ABSTRACT

There are many methods developed for measuring the crustal stress in China. Among them only two methods have been adopted in practice, i.e., overcoring method (for which the sensors include: Piezo-magnetic stress gauge, steel-ring strain cell, 3-dimensional hollow-inclusion stress gauge, and Leeman's-like stress gauge) and hydraulic fracturing method. For the former method, one of those sensors is installed in a borehole, 36 mm in diameter, then prestressed, and finally overcored with a drill bit of 130 mm in diameter. After that the deformation of the borehole wall is measured, from which the in-situ stresses are determined. In order to avoid the errors resulting from the difference in elastic modulus for various kinds of rocks as well as from the installment of the cells, we have developed a technique to calibrate the sensor on-spot using a confining pressure calibrator and produced a very good result. For the later method, the measurements are achieved by isolating a certain section of a borehole and then injecting water to pressurize it till a tensile break is induced. It is identical to the typical technology used internationally.

For many years; through field observations and a large number of laboratory experiments, we believe that the most efficient way of stress measurement for engineering purposes is the overcoring method using Piezo-magnetic stress gauge. It is characterized by stable and reliable measurement values, high accuracy and high adaptability. Taking the measurements in the intact rock mass as an example, the measured stress magnitudes are within accuracies of 8% and the errors in orientation are about 5 degrees. It can even be used perfectly in deep water. Up to now, we have performed in-situ stress measurements in more than 70 sites and obtain more than one thousand sets of data. The deepest depth used in the overcoring method is 80 m in vertical borehole and 780 m in tunnels.

INTRODUCTION

In China the in-situ absolute stress measurements started in the late fifties. At the outset the measurements were conducted with the pressure cells. Then, early in the sixties, research into the high-accuracy stress measurement systems began and became widely used approximately at the beginning of the seventies. At present more than 20 agencies are directly engaged in in-situ stress measurements. Due to the difference in technological concept, purpose, and demand, the techniques adopted in those agencies are not the same. In general, they can be grouped into two categories: the overcoring and hydrofracturing stress measurements. As the transducers and probes used in overcoring are different in their structure, the measuring instruments can be divided into piezomagnetic stressometer, steel-ring strain cell and hollow inclusion stress gauge. These instruments have, being suited to different conditions, their own advantages and defects. They are chosen according to the objectives and conditions of use. The hydrofracturing method can be applied to measure the stress at great depths with higher reliability in stress orientation. So it is often used in seismic and geodynamic researches, nuclear waste disposal, and petroleum production. On the other hand, the overcoring technique is reliable and highly accurate. It does not require a large scale equipment. Thus it is usually used for stress measurement in shallow crust, particularly in underground excavations. It has been widely used in hydroelectric engineering, hard-rock mining, coal mining, tunneling, seismic research, military industry, etc.

According to statistics, there are more than 150 stress measurements sites in China, locating in all provinces and regions except Inner Mongolia and Zhejiang Province. From these measurements several thousand sets of data have been collected and most of them were measured by using the overcoring method with the piezomagnetic cell. We began to investigate this method earlier than others. Thus all necessary equipment are available and the technique mature. Consequently the results are highly accurate, reliable and stable. Being adaptable to various conditions it

can be operated normally 100 meters deep in the water. For these reasons, it has come into wide use and produced good social and economic benefits. Statistics show that overcoring stress measurement has been conducted at more than 700 sites and more than one thousand sets of data have been collected, which provides important basis for major projects. Up to now the method has been successfully applied in the 80 m deep vertical borehole and in tunnels the depth of measurements reaches 780 meters.

Piezomagnetic Overcoring Technique and Its Principles

For this technique, the measuring instrument consists of mandrel, coil, support, wedge, spacer, upper and under cover plates (Fig. 1). The stress is measured in accordance with the piezomagnetic effect of an iron-nickel cell. Each probe unit is composed of three cells mounted at 60 degrees to each other. The unit is 10 mm long and 36 mm in diameters.

The stress measurement is carried out in a smooth 36 mm diameter borehole, in which a piezomagnetic cell is placed at a predetermined direction. As the sliding block in the cell is moved with the help of a mechanical device, the cell is held open both sides against the hole wall and prestressed. The cells are loaded separately and freed from the interference with each other. After loading the stress is relieved by overcoring with a 130 mm diameter drilling bit. For sake of recording the parameter variation of each cell (Fig. 2) and for understanding the stress state at the bottom of the borehole, it is necessary to measure every one of the cells successively or at regular intervals. Then cut off and retrieve the core that contains the probe.

In order to avoid the errors due to nonuniform contact between the cells and the hole wall and Young's modulus of the surrounding rock. Each core with the probe in place is subjected to in-situ confining pressure for calibration (Fig. 3) and determine the initial stress value of each cell.

In order to ensure that the magnitude and orientation of the stress are determined correctly, generally several to more than 10 measurements are made to obtain 10-30 effective values. Then the magnitude and orientation of the principal stress are determined by the least square method and elastic stress formulae.

For three-dimensional in-situ stress measurements three boreholes in three different directions must be drilled and measurements made in each of the three boreholes.

In order to ensure the reliability and accuracy of this method, we conducted a large-scale simulation test of stress relief using a drilling machine. Most of the samples were granite and limestone. The results show that the error in measured stress was less than 5% and the directional error was 1°-2° [1]. Generally

speaking, the errors in stress values calculated (by least square method) from the "recorded stress values" obtained by stress relief in three directions and measured several times in a hole drilled in unbroken rock were no more than 8% (with 90% confidence) and the directional errors were about 5° [2].

Stress Measurements in Jinchuan Mine and Their Application in the Design of Underground Projects

Jinchuan is a multi-mineral deposit located in the western part of China. Its mining depth has reached more than 600 meters. The ore production was seriously affected by the severe deformation of the drifts. From 1975 to 1980 in cooperation with several related organizations we performed the stress measurements at 8 sites in this mine and studied the relation between in-situ stress and drift deformation. Among the 8 sites, four were plan stress measurements while the other four were three-dimensional. Figure 4 shows the distribution of these sites and the measured stresses are listed in Table 1.

As shown by the measured results, the maximum and intermediate principal stresses are nearly horizontal. It means that the horizontal stress is predominant and compressive. It also shows that the mine is located in a high-stress area. The stress near the ground surface is about 3 MPa and increases rapidly with depth. At depths of 200-500 meters it is more than 30 MPa and reaches a maximum of 50 MPa. Therefore, the stress in ground surface increases at a rate of 5 MPa/100 m and in deeper places it increases to 10 MPa/100 m. Meantime the shear stress also tends to increase with depth. In addition, the stress distribution in this mine is not homogeneous. The magnitudes and directions of stresses measured for all sites are not the same. They differ from each other due to differences in lithologic characters and geological structures. The maximum principal stress in the eastern part of the mine is farther to the east than that in the western part. The angle between the maximum principal stress and the horizontal plane is smaller in the eastern part than in the western part.

Investigation of the deformation of openings and the stress analysis shows that the opening deformation is related to the orientation of the maximum principal compressive stress. Generally, the opening deforms much easier when it intersects the maximum principal stress at an obtuse angle. Conversely the opening is subjected to little deformation. This phenomenon is very conspicuous in broken rocks, in incompetent strata and in the fault zones (Fig. 5). For instance, in a vertical shaft which was excavated in the mid-workings of Level 1250 of the second mine, the openings intersected the maximum principal compressive stress at an angle of about 60°, the development was difficult because of the severe sloughing and caving of the openings, walls and faces in stopes. Opening deformation remained very severe even though prefabricated block wall arch, reinforced concrete horseshoe arch and full

face granite arch were employed. An opening 2.5 meters wide was squeezed to 1.7 meters wide. The deformed opening was reconstructed three times without success. This was attributed to the fact that the opening intersected the maximum principal compressive stress at a large angle resulting in high lateral pressure on the opening. Using the measured maximum principal compressive stress, the stress distributions for an opening oriented at various angles with the maximum principal compressive stress can be obtained. It was found that when the opening is parallel or nearly parallel to the maximum principal compressive stress, the stresses around the opening are small and uniform. Otherwise the stresses are large and nonuniform. Therefore, before the development of the mid-workings No. 1200, the opening direction was changed to parallel to the maximum principal stress and approximately perpendicular to the fault. Since its excavation several years ago, the opening stays essentially the same (Fig. 6) and the results have been overwhelming.

Secondly, the opening deformation is related to its own shape and the inclination of the maximum principal stress. When the maximum principal compressive stress is nearly horizontal and the width-height ratio of the opening is greater than 1.4, there will be a slight deformation only. Otherwise the deformation will be severe. When the maximum principal stress dips slightly from the horizontal plane, the opening will deform in the form of fractures on the ribs and in the roof due to a greater lateral pressure. Conversely when it is steeply inclined, the opening will be deformed into the shape of a crooked peach. Therefore based on the measured magnitudes and directions of the in-situ stresses and the finite element analysis, we suggested that the quasi-elliptical cross-section is the best one for use in the high-stress area, particularly in zones of broken and unconsolidated rocks. Because the stress distribution is more uniform under such an opening. In order to confirm such a conclusion, the quasi-elliptically shaped opening was adopted in the mid-workings of Level 1250. Since then no deformation has occurred (Figs. 7 and 8).

Thirdly, the deformation of the opening is also related to the magnitude of the in-situ stress. The deeper is the opening, the larger is the stress and the more severe is the opening deformation. Thus rational opening design in deep mining should consider the in-situ stress distribution.

In addition, we have investigated in considerable detail the problems regarding stope selection, stope span, sequence of extraction, and stope back-filling. Our recommendations in these areas have enabled the engineering department to gain excellent results [3].

Stress Measurements at Ertan Hydroelectric Station and Their Application in Site Selection in the Underground Generator Rooms

Ertan hydroelectric station is a large-scale hydropower station being built in the southwestern part of China. The dam will be 240 meters high. Many large generator rooms will be built underground. The largest room will cover an area of 63 x 27.5 x 240 m. During exploration a great portion of the cores exhibited discing. In order to ensure the safety of the dam and the generator rooms rational construction procedure was adopted. From July, 1980 to October, 1981 stress measurements were conducted at 7 sites located mainly along the axis of the dam, and near the underground generator rooms and water-tunnels. Fig. 9 is the plan of site distribution. The measured results are listed in Table 2.

The stress measurements show that the station lies in a high-stress area, where the maximum principal compressive stress is greater than 20 MPa and the shear stress is comparatively high, in some places reaching 14 MPa. At every site except No. 4 and 7, the inclination of the maximum principal stress is less than 30°. Thus in this area the horizontal stress is predominant and the stresses are non-uniformly distributed. Based on the stress distribution on both sides of the valley, owing to the topographic effects, this area can be divided into three regions, i.e. the stress-relief region, the stress concentration region, and the normal stress region. In the vicinity of the slope the maximum principal stress runs parallel to the mountain slope and becomes horizontal in the normal stress region (Fig. 10). At the bottom of the valley there is a high-stress region, in which the measured stress reaches 60 MPa.

In designing the generator rooms the site for building the station should be kept away from the stress concentration region on the slope and located in the normal stress region. As the shape and size of the room had been determined to meet the engineering requirements of the project, the measured in-situ stresses were fully utilized for the arrangement of the main buildings, transformer room, and pressure regulation room. The finite element analysis was also employed. It was found that the side walls of the three rooms were heavily deformed inward when the stresses were redistributed after excavation. In consideration of the joint effect and the severe stress concentration in the roof and floor of the pressure regulation room, we have noticed that they will lead to rock bursting from the floor, if the excavation is started from the roof and cut downward [4].

Stress State at a Hydroelectric Station in the Upper Reaches of the Yellow-River and Some Questions Relevant to the Designing of the Underground Buildings

Although the hydroelectric station in this case is a large one, its underground building

construction is of the cave dwelling type. Based on the original in-situ stress measurement, it is not a high-stress region. At the place where the underground buildings stand the maximum principal stress is merely 5.5 MPa. But the long axis of the building (N20°W) and the maximum principal compressive stress (N58°E) intersects at a large angle. The stress concentration at the roof calculated for that cross-sectional shape is about 3. The ratio of lateral to vertical pressure equals to 1.2. It can be seen that the design plan for the underground building in this station is not the optimum one. Considering other engineering geological conditions it seems that support reinforcement should be emphasized in the design of those underground structures (Figs. 11 and 12).

Moreover, there are seven faults around the cave-like buildings. Using the measured three-dimensional stresses, we determined the shear and normal stresses on each fault plane. The shear-to-normal stress ratio ($\tau/\sigma = K$) and the measured shear strength ($\tau F'$) of the fault plane were compared to obtain a safety coefficient for sliding ($\tau F'/K$) (there was no groundwater around the cave-like buildings). By so doing, we learned that along one of the faults the safety coefficients for sliding has approached 1. During excavation caution must be exercised against fault sliding.

CONCLUSION

In recent years great advances have been made in the field of in-situ stress measurements in China. Owing to differences in purposes and requirements the development trends of the measuring techniques are not the same. The overcoring technique is developing towards the overall stress measurement in a single borehole, being suitable for operation in greater depths and free from the constraints of rock conditions. As it provides reliable stress values, the single hole overall stress measurement technique is cheap. Therefore it has been well received by the users. As the stress measurements are often required in areas where unconsolidated and broken rocks exist, some research organizations have started to investigate new techniques for stress relief in slightly unconsolidated rocks and at great depths. The equipments for hydrofracturing stress measurement are tending to be small-sized, portable and the development of packers for various borehole diameters.

In the field of hydroelectric engineering the stress measurements are not only applied in the designing of underground buildings and selection of excavation plans, but also in the investigation of dam selection, storage room construction, induced earthquake, quantitative analysis of active faults and stability of wall rocks. With respect to seismology the stress measurements are used in the studies of the regional stress field and in stress field near earthquake epicentre. Meanwhile there are rapid developments in the determination of relative stress changes with time. At present a high-accurate piezo-capacitive stress meter has

come into use. Its sensitivity reaches $10^{-10} \mu\epsilon$. The stress meter consists of 3 directional cells which intersect each other at 60°, from which the magnitude and direction of the maximum principal stress changes can be determined.

In coal production the stress measurements have been applied for the investigation of coal explosion and gas burst. In a coal mine in Southwest China, gas burst is a severe problem. In-situ stress measurements were conducted there and the tensile stress was found to occur in the mine. This project is in progress.

Finally stress measurement is widely used in mining, nuclear electric power station, tunneling and petroleum production. We believe that they will play an even more important role in many other areas, especially in underground construction.

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Table 1. Results of in-situ stress measurements in Jinghuan Mine

Measuring Site	Strata	Depth below surface (m)	Max. Principal stress			Int. principal stress			Min. principal stress			Year of Measurement
			value (MPa)	direc- tion	dip	value (MPa)	direc- tion	dip	value (MPa)	direc- tion	dip	
1	Marble	20	2.4						2.3			1977
2	Marble	44	4.2	N20E					3.5			1978
3	Marble	375	19.8	N3E					10.8			1975
4	Granite	480	24.5	N25W					15.4			1980
5	Marble	460	50.0	N13W	6SE	33.4	N76E	6NE	28.2	S63E	81NW	1976
6	Rich Ore	480	32.0	N32E	6SE	21.4	S43E	67NW	20.6	N60W	22SE	1978
7	Rich Ore	240	34.4	N42W	39NW	21.1	N48E	0	2.6	S41E	51SE	1977
8	Marble	120	16.8	N28W	57SE	12.1	S35W	16NE	5.8	S63E	28NW	1979

Table 2. Results of in-situ stress measurements in Er-Tan

Measuring Site	Strata	Depth below surface (m)	Max. Principal stress			Int. principal stress			Min. principal stress			Year of Measurement
			value (MPa)	direc- tion	dip	value (MPa)	direc- tion	dip	value (MPa)	direc- tion	dip	
1	Basalt	196	296	179	8	117	76	57	74	95	-32	1980
2	Syenite	67	96	173	-31	45	343	-58	43	261	4	1980
3	Basalt	196	384	202	-26	173	291	3	104	195	63	1980
4	Syenite	209	195	200	-57	67	340	-27	57	260	18	1981
5	Pyroclast	125	266	196	-27	168	281	10	108	172	61	1981
6	Syenite	370	244	218	6	231	315	47	98	123	42	1981
7	Syenite	291	295	198	-40	221	335	-42	187	267	24	1981

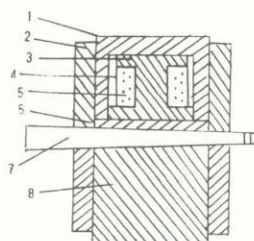


Fig. 1. Piezomagnetic cell.

- 1 - cell housing; 2 - plate;
- 3 - nickel cell spool; 4 - screen;
- 5 - coils; 6 - spacer; 7 - wedge;
- 8 - support.

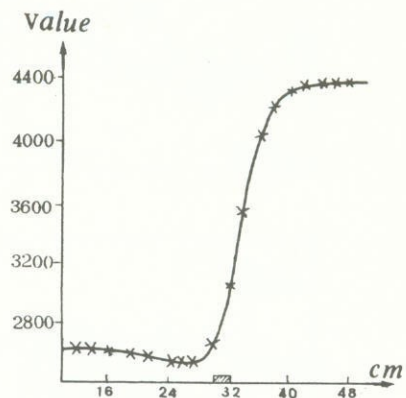


Fig. 2. The stress release curve for cell #7 at measuring point #6 in Er-Tan.

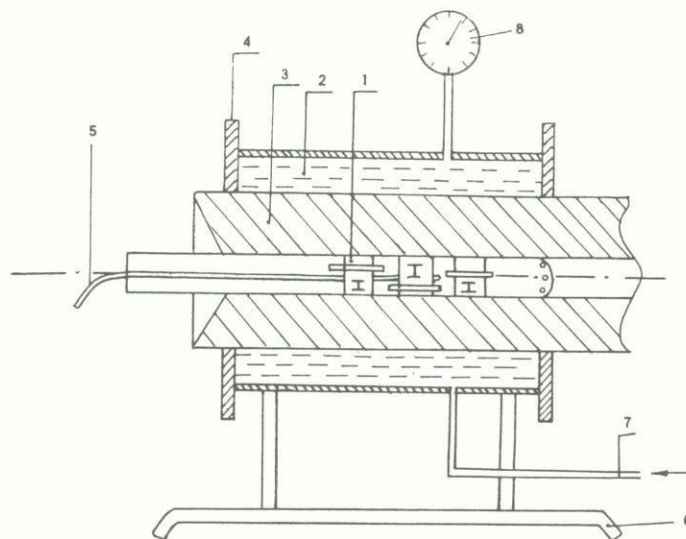


Fig. 3. Oil pressure calibration device schematics of
 1 - cell; 2 - oil; 3 - rock core; 4 - steel cylinder;
 5 - wire; 6 - shelf; 7 - oil inlet tubing;
 8 - pressure gage.

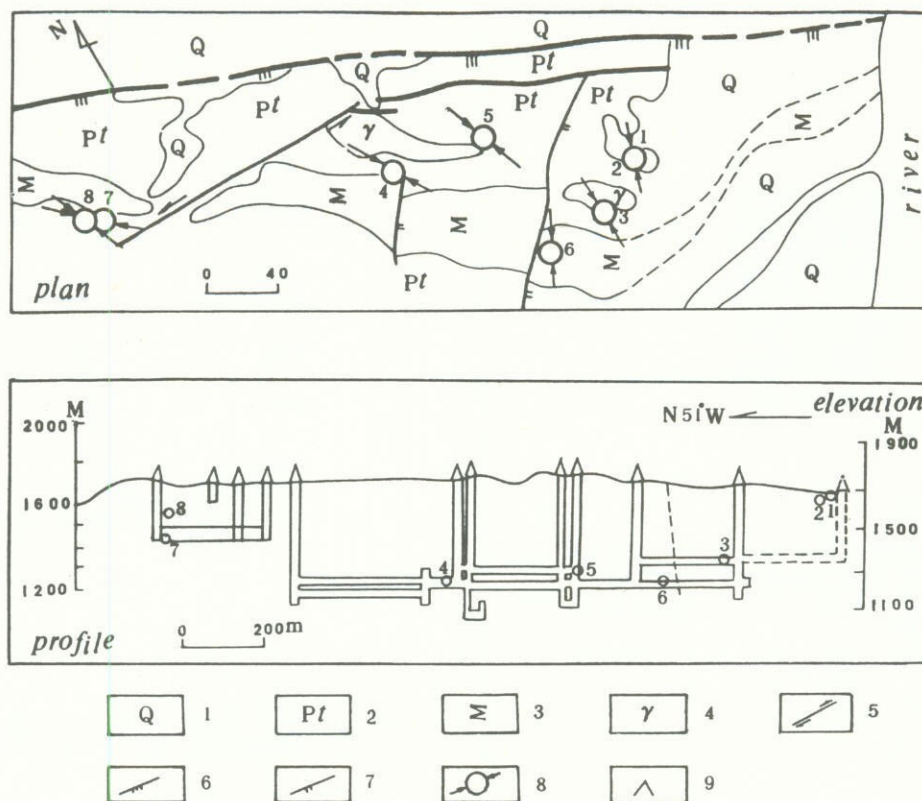


Fig. 4. Distribution of stress measuring sites in Jin Chuan mine area.
 1 - Quaternary; 2 - lower proterozoic; 3 - Ultrabasic rock;
 4 - Granite; 5 - strike-slip fault; 6 - compressive fault;
 7 - tensile fault; 8 - location and number of stress measuring sites.

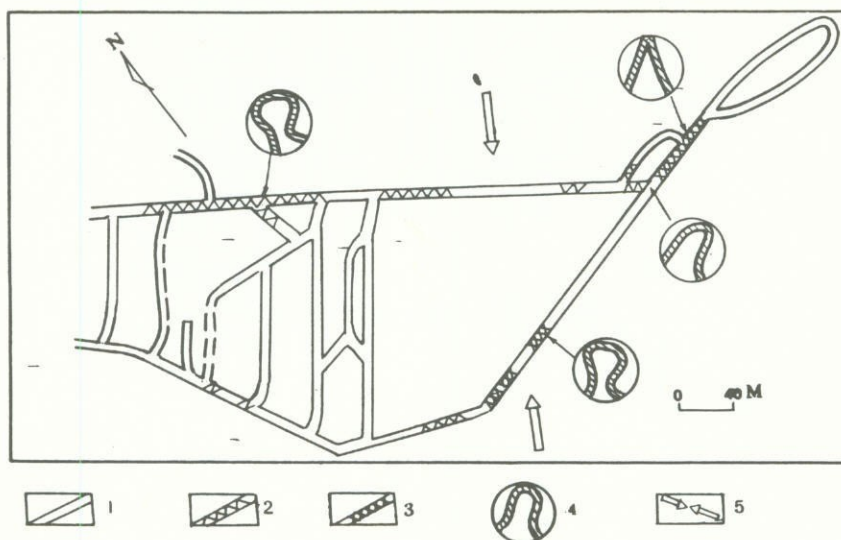


Fig. 5. Drift deformation of the first mine block at level 1460 in Jin Chuan Mine.
 1 - drifts; 2 - severe deformation; 3 - medium deformation; 4 - state of
 drifts deformation; 5 - direction of maximum principal stress.

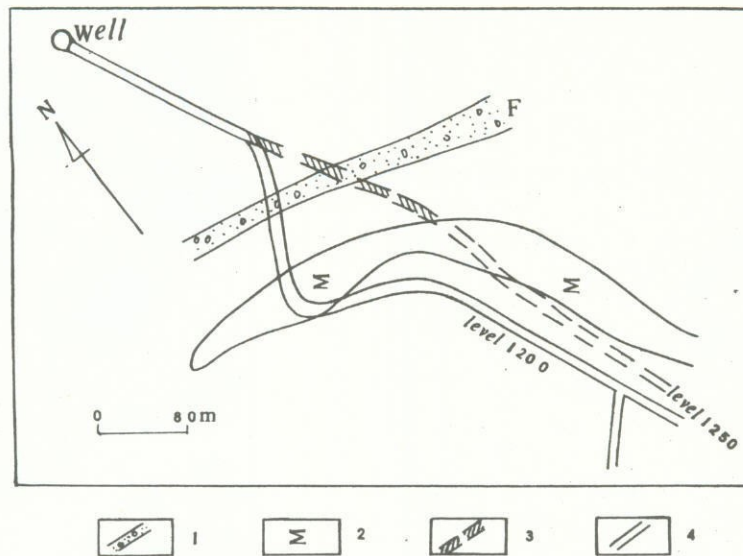


Fig. 6. Damages to the drifts at Level 1200 and Level 1250 in Jin Chuan Mine.
1 - fault; 2 - ore body; 3 - severely damaged segment; 4 - drift.

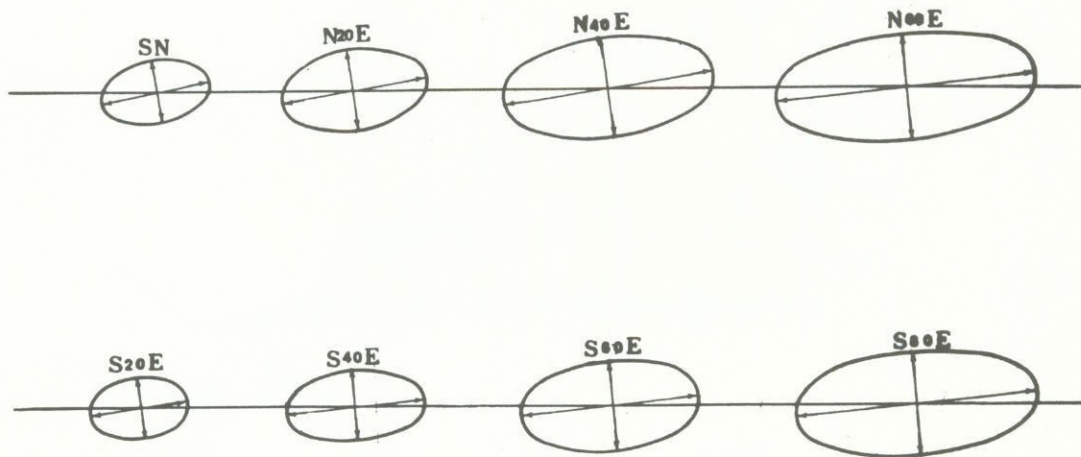


Fig. 7. Stress ellipsoid measured at different sections at the workings in Level 1300, Jin Chuan Mine.

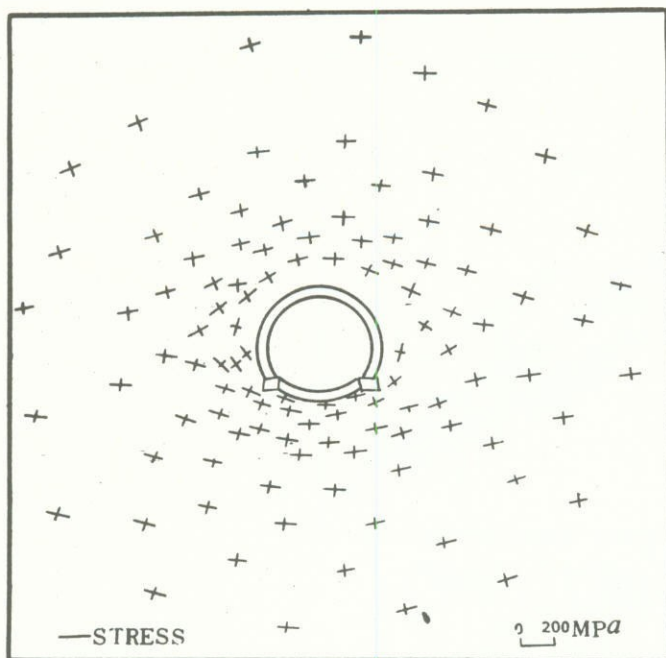


Fig. 8. The maximum principal stress distribution around the quasi-elliptical drift (by Liao Chunting).

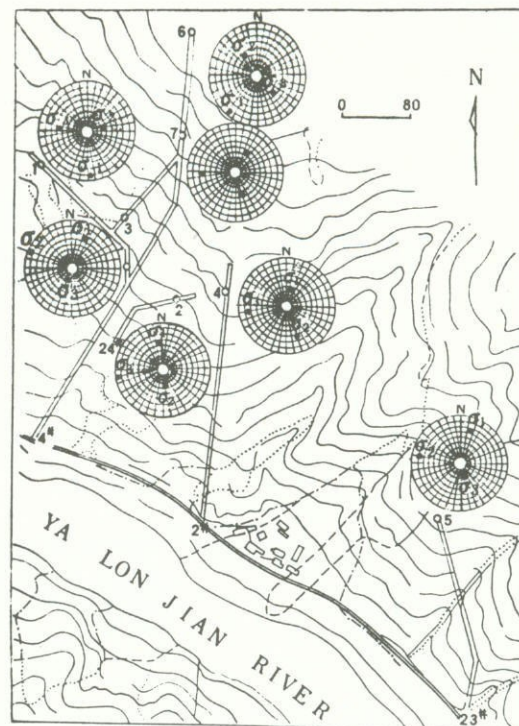


Fig. 9. The distribution of the stress measuring sites in Er-Tan. (The orientation of the maximum principal stress is by upper hemisphere projection).

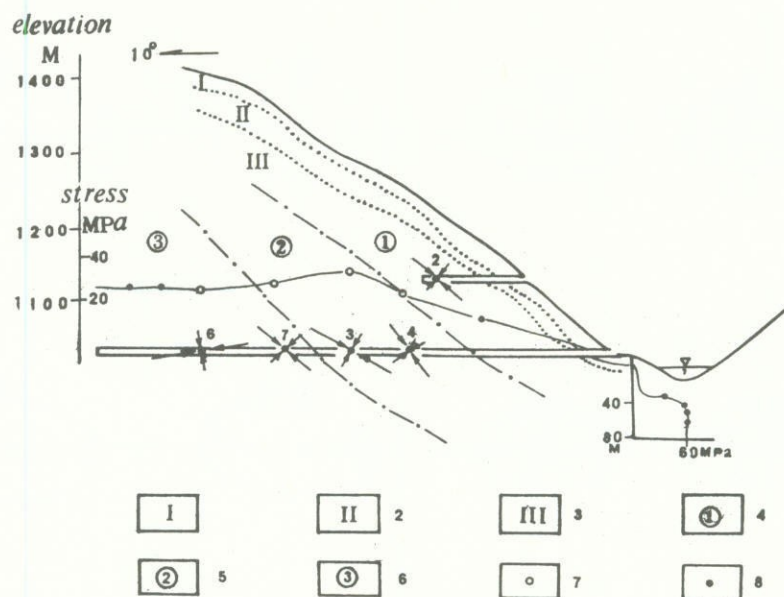


Fig. 10. The measured stress profile in Er-Tan.
1 - weathering zone; 2 - poorly weathering zone; 3 - fresh bedrock zone; 4 - stress release region; 5 - stress concentration region; 6 - normal stress region; 7 - stress measurement sites using piezomagnetic method; 8 - stress measurement sites using strain cell method.

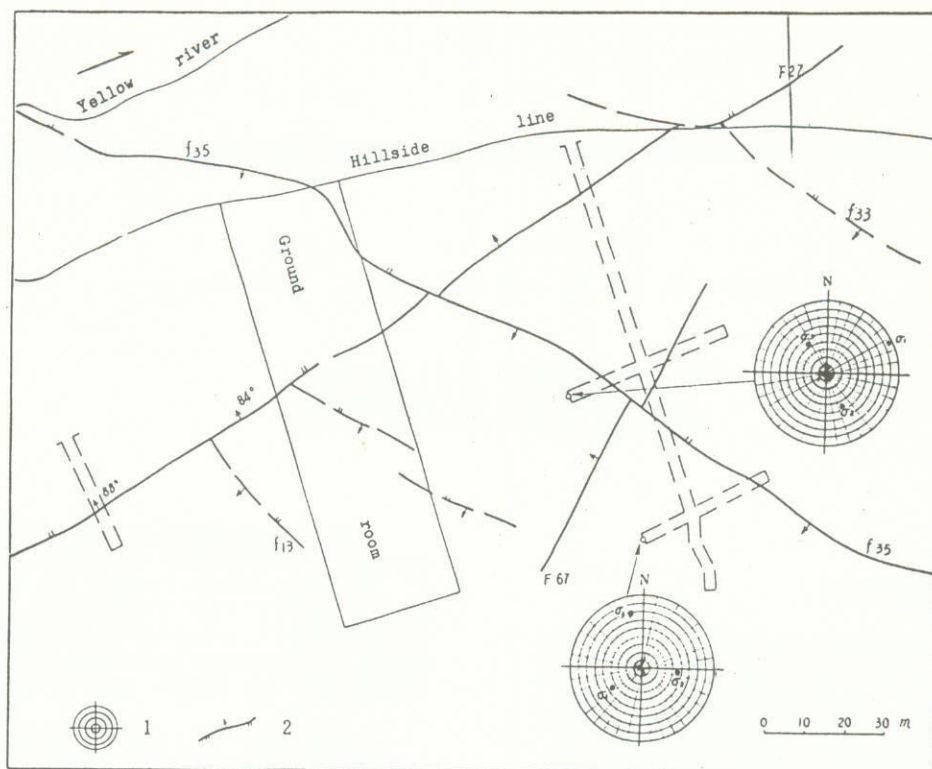


Fig. 11. Distribution of the stress measurement sites at a power station on the Upper Yellow River.
1 - stress orientation (projection of the lower hemisphere);
2 - faults.

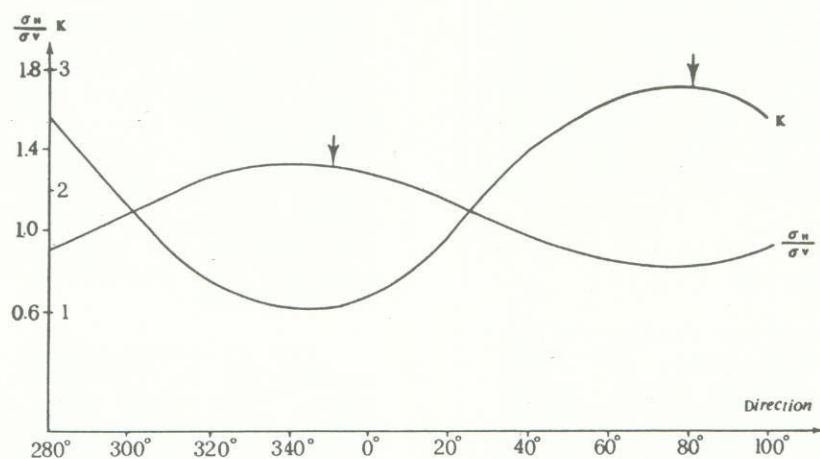


Fig. 12. Stress concentration and lateral pressure ratio for the underground room of a power station and the Upper Yellow River.

FIELD MEASUREMENTS OF OVERBURDEN AND CHAIN PILLAR RESPONSE TO LONGWALL MINING

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ABSTRACT

This paper presents the results of an extensive geomechanical testing and monitoring program conducted at a longwall panel. The field program included the monitoring of roof caving sequence and mechanism, surface subsidence, change of pillar stress, roof displacement and entry convergence, and strength characterization of the overburden rocks. The response of eight headgate pillars and the adjacent entries was monitored as the longwall face approached and passed the instrumented locations. Overburden response to longwall mining was monitored using time domain reflectometry (TDR) technology; and, the height of the highly fractured zone above the gob was determined by postmining drilling. Good correlation was found between the height of the highly-fractured zone and the TDR, subsidence, and pillar stress data. The results provide a detailed picture of the response of overburden strata and chain pillars to longwall mining. A preliminary model representing the overburden strata behavior at the test site in response to the longwall mining is proposed.

INTRODUCTION

Current longwall pillar design relies heavily upon past experience and many assumptions about pillar and entry response to longwall abutment pressures (Wilson, 1981; Hsiung and Peng, 1985; Mark and Bieniawski, 1986). Caving characteristics of the immediate and main roof strata are known to play a very important role in surface subsidence and the overall pressure redistribution, which determines the maximum abutment pressure, abutment pressure at the T-junction, and width of the influence zone.

The geomechanics instrumentation program described in this paper is part of a long-term study to further understanding of the influence of caving due to longwall mining and subsequent overburden response on surface subsidence and stability of chain pillars.

Results of the geomechanical monitoring provide a detailed picture of the response of overburden strata and chain pillars to longwall mining at the test site. This information will be used to assess existing longwall pillar design methods and to refine numerical as well as analytical models as part of an effort to improve the current state-of-the-art in longwall ground control.

PANEL GEOMETRY AND SITE GEOLOGY

The mine studied is in the Pittsburgh seam of the northern Appalachian coalfields. The average extraction thickness is 6 to 7 feet in the development entries and 5.5 feet in the panel. The longwall panel is 600 feet wide and 7,200 feet long (Figure 1). The headgate entries were driven on 100-, 100-, and 50-foot centers; the tailgate entries for this panel were driven on 100-, 75-, and 50-foot centers. The crosscuts generally are driven on 150-foot centers. The entries and crosscuts are approximately 15 feet wide.

The Pittsburgh coal seam is approximately 700 to 720 feet deep at the test site. The seam lies directly beneath about 200 feet of highly competent limestone and sandstone strata. Progressing upward from the Pittsburgh coal, the immediate roof consists of 10 to 15 feet of shale, claystone, and rider coal. Directly above this sequence lie two limestone strata, separated by a 5-foot claystone and shale interface. Above the limestone unit is 45 to 50 feet of sandstone, followed by 10 feet of highly

disturbed shale. Above this horizon is 50 to 55 feet of another limestone unit, followed by four 5- to 10-foot limestone units separated by 5 to 10 feet of shale.

Above the first 200 feet lie mostly shales and claystones with intermittent beds of limestone and sandstone. For the most part, these sandstone and limestone strata represent competent zones of moderate to high strength rocks, whereas the shale and claystone are of low to moderate strength. Interfaces between high and low strength strata, sometimes accompanied by coal or bone, also represent weak planes or planes of high differential stiffness.

Influencing anomalies present at the Pittsburgh coal horizon were the seam syncline and the clay vein disturbance. The synclinal effect of seam elevations are a potential cause for both roof and floor instability, due to slight strata warping and the resulting sump effect, respectively. The clay vein may also affect both roof and pillar stability due to the highly disturbed interface.

FIELD INSTRUMENTATION

The study site was chosen based on land access, topography, and mine layout. The field program consisted of surface as well as underground instrumentation.

Surface Instrumentation

A. Subsidence Monuments

Four survey lines were installed at the test site (Figure 1). The subsidence monuments consisted of 18-inch roof bolts, which were installed on 25-foot centers for lines A, B, and D and 10-foot centers for line C. Closed-loop spirit surveys using an automatic level were used to determine the elevations. A reference point more than 1,000 feet to the north of the panel was chosen as the bench mark for the surveys.

B. Time-Domain Reflectometry (TDR)

The first phase of the surface instrumentation program included drilling three boreholes. These boreholes (TDR #1, TDR #2, and TDR #3) were drilled to a depth of 10 feet below the Pittsburgh seam, and their spatial location with respect to the longwall panel is shown in Figure 1. A fourth hole (TDR #4) was drilled 30 months later to supplement the three original holes.

Two coaxial (RG 59) cables and one twisted pair cable were carefully grouted from the bottom to the top of each of the three original holes. Two coaxial cables

(both looped back to the surface) were grouted in hole TDR #4. These cables were monitored using TDR technology to detect strata movement. The TDR device works on the same principle as radar by sending an ultrafast electrical pulse down the cable. Any disturbance in the cable, caused by strata movement, will reflect the TDR signal back to the cable input. This reflected voltage is displayed as a step-up or step-down transition on a CRT (or strip chart) at a point corresponding to the depth of the disturbance.

Underground Instrumentation

Underground instrumentation, including vibrating wire stressmeters, borehole extensometers, and convergence pins, was installed in the headgate entries between 29+50 and 33+50 (Figure 1).

A. Stressmeters

Twenty-nine IRAD-GAGE, vibrating-wire stressmeters were installed in the headgate chain pillars. The stressmeters were set 12 to 15 feet into the rib with PVC conduit to protect stressmeter cables. The last few feet of each 1 5/8-inch hole was drilled with a 1 1/2-inch concrete rotary bit. The stressmeters were installed one foot from the end of each hole and were prestressed per manufacturer's recommendation.

B. Borehole Extensometers

Seven borehole extensometers were installed in the roof at intersections in the escapeway, track and return air entries (Figure 1). Two of the extensometers had anchors at 4 and 8 feet, two had single anchors at 10 feet, and the other three had single anchors at 8 feet.

C. Convergence Pins

Eighteen convergence stations were established at the instrumented area (Figure 1), and each convergence station consisted of two one-foot rebars--one in the roof and the other in the floor. Entry convergence was monitored using linear potentiometers mounted on spring-loaded convergence rods.

RESULTS

Surface Instrumentation

A. Surface Subsidence Measurements

The first subsidence readings were recorded 19 months prior to mining.

Accuracy of each survey loop was at least 0.025 foot, and typically 0.010 foot or less. Each shot was surveyable to within 0.005 foot.

Surface subsidence measurements at the test site revealed that detectable movement did not occur until the face was within 200 feet of center-panel monuments. Movement in advance of the face was more pronounced toward the headgate side of the panel, owing to the steep surface slope above the headgate. Initial movement over the headgate ribside occurred 250 to 300 feet in advance of the face, and less than 100 feet over the tailgate ribside. Final ribside subsidence was 0.25 foot for the headgate and 0.73 foot for the tailgate.

Development of the transverse profile (Figure 2) was skewed toward the preceding panel gob side. Initial profiles were skewed more toward the gob side than were residual profiles; this resulted from the transverse line (Line A) being angled toward setup of the previous panel (Figure 1). As time dependent response (residual subsidence) gained influence, the profile approached symmetry; however, the final profile was still 40 feet off panel centerline. Final maximum subsidence was 3.36 feet.

The traveling center-panel profile (Line C in Figure 1) indicates temporary surface response with respect to face position (Figure 3). The onset of the traveling "wave" occurred 200 feet in advance of the approaching face. The profile progressed through tensile then compressive zones, as related to profile curvature. Tensile movement continued for about 250 feet and surface compression continued to about 700 feet after face passage. The inflection point separating these zones is near half maximum subsidence. From 700 feet inby, residual settlement occurred. Residual settlement here is defined as the final 5 percent of measurable movement and is indicative of gob compaction and closure of minute fissures in the overburden.

B. Time Domain Reflectometry (TDR)

Prior to mining of the preceding panel, there were three TDR holes (TDR #1, #2, and #3), and all TDR cables were grouted to the Pittsburgh seam horizon. Following extraction of the preceding panel, all three holes (3 cables each) showed TDR cable failures at the horizon from 100 to 110 feet above the Pittsburgh coal seam. This corresponds with the absent Sewickley coal horizon, represented by a highly disturbed zone of slickensided shales. In order to provide

a cable response below this horizon prior to mining, TDR #4 was drilled at the panel center 100 feet west of TDR #1 (Figure 1). At this time, it was suspected that mining of the preceding panel provided sufficient relief for strata slip to occur within the instrumented panel. However, this was not verified, although equipment malfunction was ruled out.

Upon mining of the instrumented panel, TDR cables showed no response until the face approached to within 100 to 200 feet of the TDR monitoring holes (Figure 4). At these distances, all four holes showed renewed cable failures at elevations between 775 and 785 feet. This elevation interval corresponds with low strength shales and claystones which lie directly above a thick, competent limestone stratum and the four thinner, overlying limestone strata. Passage of the face directly beneath TDR cables caused additional cable failure. For TDR holes #1, #2 and #3, this corresponds to the 1,000-foot elevation. Cable failures continued in succession upward, exhibiting a trailing profile. Failures ceased when the face had passed 200 feet outby. Measurable failures ceased at depths of 86 to 91 feet for TDR #1 and between 33 to 39 feet for TDR #2 and #3. Although response of cables from the three original holes were equivalent to within 190 to 200 feet of surface, center-panel response (TDR #1) varied from quarter panel response (TDR #2 and #3) above this depth (Figure 5). It is suspected that center-panel compressive strain inhibited the shallow response observed in quarter-panel TDR cables.

TDR cable failures occurred typically at strong to weak rock interfaces and in weak strata units such as claystone or coal seam horizons. In addition, near surface responses correlated with interfaces at the upper side of thick sandstone beds. Sandstone was the most prominent hard rock strata located at shallower depths. Propagation of TDR failure also correlates well with the tensile zone of the traveling subsidence profile. This results from interface slippage due to overburden flexure and high differential stiffness across weak and strong rock interfaces as the strata warp downward over the advancing face-line into the gob. Finite element models, using elasto-plastic material properties with slip interfaces, verified the potential of this mechanism.

The implication of premining slippage at the 775-foot elevation was that caving of the overburden must exist below this elevation, since significant bending of competent strata into the gob was

required to produce at least 0.25" of relative movement. Postmining drilling verified the fracture height to exist only up to the 715-foot elevation, 165 feet above coal extraction horizon.

Underground Instrumentation

A. Pillar Stress

The measurement of stress buildup was monitored in relation to the face position of the retreating longwall. Figure 6 shows typical vertical stress increases in the coal pillars. A maximum pillar stress increase of 2,530 psi was recorded in the abutment pillar next to the gob. Pillar stress increase decayed quasi-exponentially from the headgate abutment pillar to the tailgate yield pillar. Front abutment increase at the T-junction area was found to be approximately 25 percent of the maximum recorded at the side abutment. The lower front abutment stress at the T-junction generally indicates better headgate stability. The majority of the stressmeters continued to show stress increase at the completion of the panel. The maximum rate of stress increase occurred between the face positions of 100 feet inby and 100 feet outby the stressmeters. The yield zone on the gob side of the first row abutment pillar extended less than 10 feet into the pillar at the completion of panel mining.

Only a very small amount of stress increase was recorded in the abutment pillars next to the panel when the face was 400 feet inby, while significant stress increase (50-100 psi) was first recorded in the abutment pillars next to the panel when the face was 150 feet inby. The average stress increase at the core of the yield pillar was 320 psi, and the yield pillar did not appear to yield completely at the completion of panel mining.

Pillar stress change as a result of longwall mining was also calculated using the finite element method. Figure 7 shows the comparison between the measured pillar stress changes and those calculated by a three-dimensional finite element program employing elasto-plastic material properties. The Mohr-Coulomb criterion was used as the yield criterion. Based on the 165-foot caving height determined by postmining drilling, pillar stress increase can be reasonably predicted.

B. Borehole Extensometers

Bed separations within the first 10 feet of the roof were monitored with respect to the face position and Figure 8

shows typical roof bed separations for three entries. In the intake escapeway (entry #3), a small amount of bed separations occurred between the roof line and the anchor (8 feet deep) when the face was parallel to the extensometer. A high rate of bed separation was indicative of roof instability. In the track entry (#2), bed separation was not measurable until the face was approximately 200 feet outby the extensometer. Separation within the first 4 feet of roof increased gradually and reached a constant value when the face was 1200 feet outby. The majority of bed separation (up to 0.7 inch) occurred between the roof bolt anchor and the deep extensometer anchor. Bed separation in the return air entry (#1) as a result of longwall mining was negligible at the completion of the panel mining.

C. Roof-to-Floor Convergence

Convergences in the headgate entries and crosscuts were recorded with respect to the position of the longwall face and typical values are shown in Figure 9. Measurable roof-to-floor convergence was first recorded when the face was 150 to 200 feet inby the stations. Three-way intersections exhibited higher rates and accumulation of convergence due to floor heave.

A roof fall in the crosscut nearest the gob was preceded by a high convergence rate of 0.16 inch per 10 feet of face advance. In the track entry, only 25 percent of the total convergence can be attributed to the bed separation within the first 10 feet of roof. A total roof-to-floor convergence of less than 0.5 inch was observed in the future tailgate entry.

Three-way intersections generally provide better roof conditions during and after entry development. However, in this study stress increase associated with longwall mining caused the floor at the three-way intersection to heave while the roof remained intact. Floor heaving in the intake escapeway, and part of the track entry did not occur until the face was 600 feet outby and did not pose any problem. Convergence near the major clay vein was negated by additional roof support. Significant stress increase in the pillars and the first noticeable roof-to-floor convergence were recorded when the face was approximately 150 feet inby.

DISCUSSION OF RESULTS

The primary objective of this research is to further the understanding of the influence of caving due to longwall mining and subsequent overburden response on chain pillar stability and surface subsidence. More data needs to be gathered under various conditions before the interaction among these parameters can be fully understood. The ultimate goal is to predict pillar stress and surface subsidence due to longwall mining. At this stage, underground and surface data from this project were cross-correlated and used to construct a preliminary overburden response model. A reasonably accurate prediction of pillar stress increase due to longwall mining was achieved based on the known depth of the highly-fractured zone.

The crest of the highly-fractured zone above the mined-out area was 165 feet as determined by post-mining drilling. Rock core clearly illustrated the transition from a relatively unfractured zone to the highly-fractured zone. The transition was marked by the onset of closely-spaced vertical and inclined fractures. Based on the known depth of fractured zone, a three-dimensional finite element model was constructed to predict the pillar stress change at the completion of the panel mining (Figure 7). It is seen that reasonably accurate prediction of pillar stress increase due to longwall mining can be achieved if the depth of the highly-fractured zone can be predicted.

Laboratory testing of TDR cable showed that the cable is highly susceptible to shear displacement. The movement required to yield a TDR break response averages 0.25" for a $\frac{1}{4}$ " coaxial cable. The result of a two-dimensional finite element simulation of TDR break response showed that at 230 feet above the coal seam horizontal slippage of 0.237 inch is possible at 100 feet in advance of the face. The in situ horizontal stress in the longitudinal direction was assumed to be 1,100 psi. Actual TDR break response indicated that horizontal slippage at the same horizon occurred when the face was 100 to 200 feet inby. This discrepancy between actual and modeled results may have resulted from the assumed values of in situ horizontal stress and interface shear stiffness. Higher in situ horizontal stress will cause more horizontal slippage.

TDR data and results of postmining drilling strongly suggest that vertical fracturing propagated only up to the 550-foot depth level, and that relative movements which occurred at depths from

50 feet to 550 feet were mostly horizontal in nature. The strata between the depths of 50 and 550 feet behaved like a thick composite beam resting on a stiff elastic foundation of variable modulus. The strata comprising the beam progressively bent downwards. This movement continued toward the surface as the longwall face passed. Differential strain due to differential stiffness between weak and strong layers might have developed which eventually sheared the interface. This progressive shearing effect was readily observed by TDR cable movements. The horizontal slippage which occurred at interfaces between weak and strong strata was correlated not only to the lithologic column but also to the strength index as determined through point-load tests, Brazilian Disc tests, core recovery, and RQD indexing.

OVERBURDEN RESPONSE AT THE TEST SITE

The response of overburden strata to longwall mining is understandably a complex phenomenon. However, based on the geomechanical data gathered over the longwall panel, the overburden strata response at the test site was distinguished into four zones as shown in Figure 10.

ZONE 1: Directly above the mined panel and to a height of approximately 4-6t (where "t" is the extraction thickness) lies a gob zone of highly rubblized material. This zone represents the immediate roof rock which caves upon extraction of coal, dilating in volume to fill the void. The zone consists of a limestone bed and its associated shaley lime and shale strata. Although the voids close with time due to compaction, the material remains highly porous due to intense rubblization.

ZONE 2: Above the gob zone with a thickness of 25t, is a transitional, highly-fractured zone defined by massive block-type caving and vertical fracturing. This zone represents the main roof rock which fails progressively after passing of the face. The zone consists of massive hard rock layers and interface strata represent weak layers. The massive layers, although strong, are quite brittle, and once unable to absorb the strain energy, fail suddenly, yielding tensile and shear-type vertical and inclined fractures.

ZONE 3: Above the highly fractured zone and progressing to within approximately 50 feet from the surface, is a relatively unfractured zone characterized by occasional horizontal slippage along weak-strong rock interfaces and little, if

any, vertical fracturing. The zone consists of a variety of strong and weak sedimentary strata. Weak zones of claystone and coal, typically represent planes of slip. Slippage occurs progressively to the surface with passing of the face. This zone deforms like a composite beam with occasional slipping along weak-strong rock interfaces. Separation closes with time and passing of the face, due to vertical settlement and compaction of the overburden.

ZONE 4: Within the near surface area and to a depth of about 50 feet lies the surface zone. This zone consists of surface soils and relatively weak limestones, shales, and claystones, which are susceptible to fracturing and movements due to the flexure of underlying strata. Fractures will tend to fill quickly due to silting and infiltration of water susceptible clays, as well as recompression of the overburden in trough zones due to passing of the face. Movements in this zone can be represented well through surface subsidence monitoring.

CONCLUSIONS

1. Correlations between pillar stress increase, subsidence, TDR results, and other forms of instrumentation can be used to define characteristics of overburden movements and caving vital for pillar design, and surface subsidence prediction.
2. TDR response occurred typically in weak strata units and at strong to weak rock interfaces. Propagation of TDR failure correlates well with the tensile zone of the traveling subsidence profile. TDR cable failures observed at deep horizons in advance of the face, and at shallow horizons behind the face are mostly a result of shear slippage due to both bending of the strata into the gob and to high differential stiffness at weak and strong rock interfaces. TDR failure at shallow horizons does not signify so-called fracture or caving zones associated with longwall mining.
3. Four zones are suggested to exist in the overburden strata influenced by longwall mining at the test site, including a thick composite beam zone which is relatively unfractured and retains its lateral continuity and integrity. Effect of the occasional, transient horizontal slippage in this zone disappears with overburden recompaction.
4. Pillar stress increase decayed quasi-exponentially from the abutment pillar to the yield pillar, and indicated that the abutment pillar design protected the future tailgate adequately.
5. Bed separation and roof-to-floor convergence in the future tailgate were negligible, indicating that the future tailgate is adequately protected.
6. The current headgate design provides stable entries for longwall mining at the test site; occasional roof, floor, and rib instability is a direct result of local geological anomalies.

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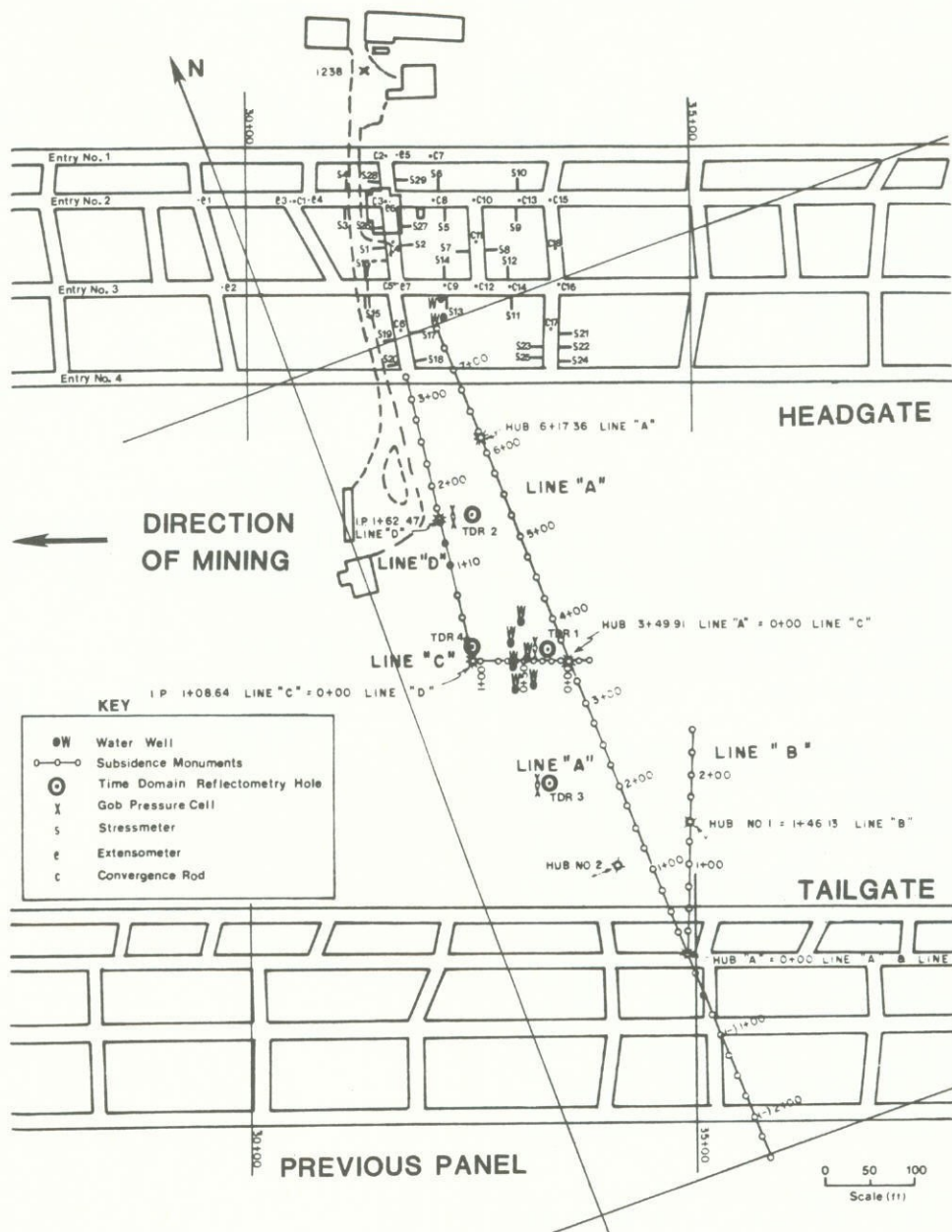


FIGURE 1. SURFACE AND UNDERGROUND INSTRUMENTATION

FIGURE 2.
TRANSVERSE SUBSIDENCE PROFILES
LINE A

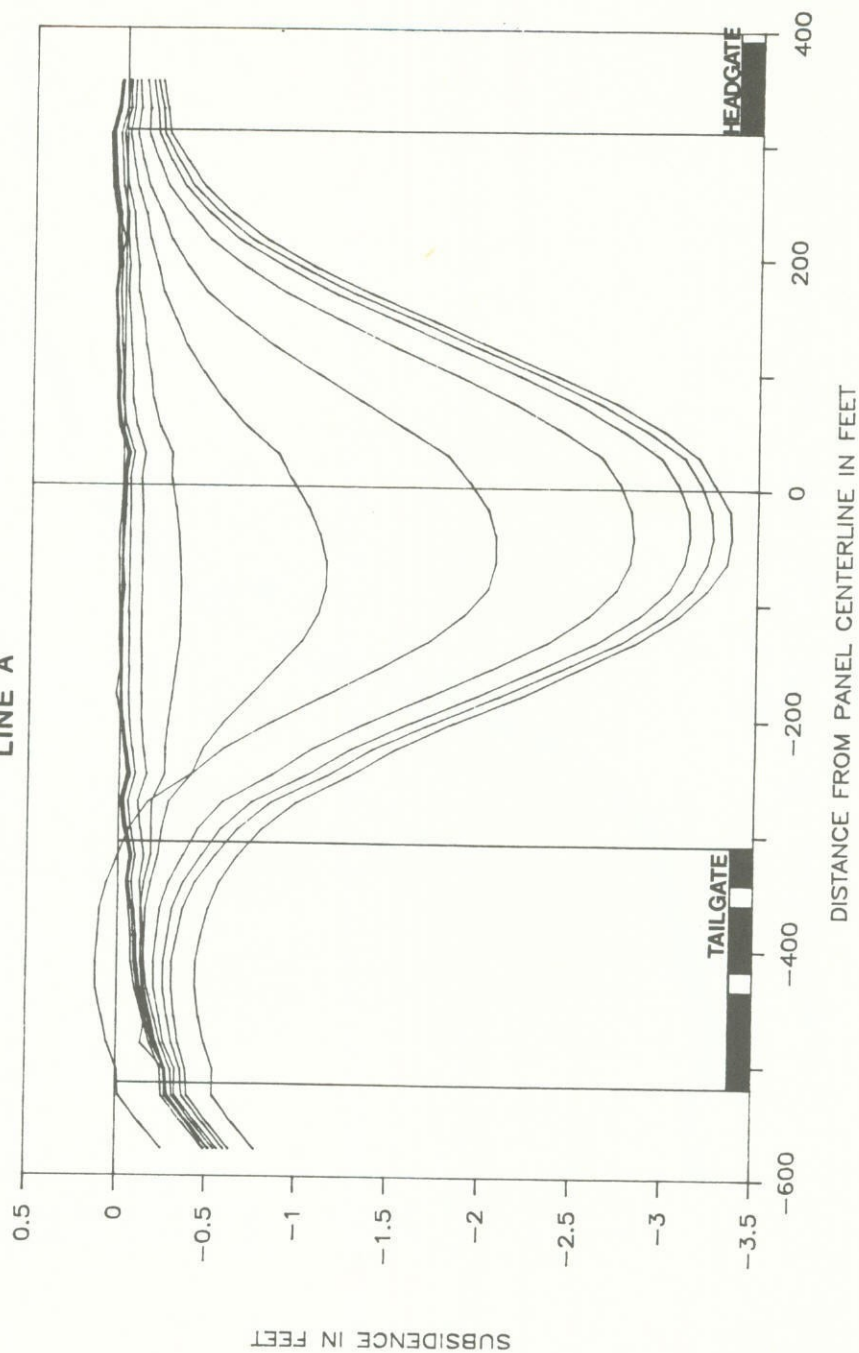


FIGURE 3.
TRAVELLING SUBSIDENCE PROFILE
LINE C

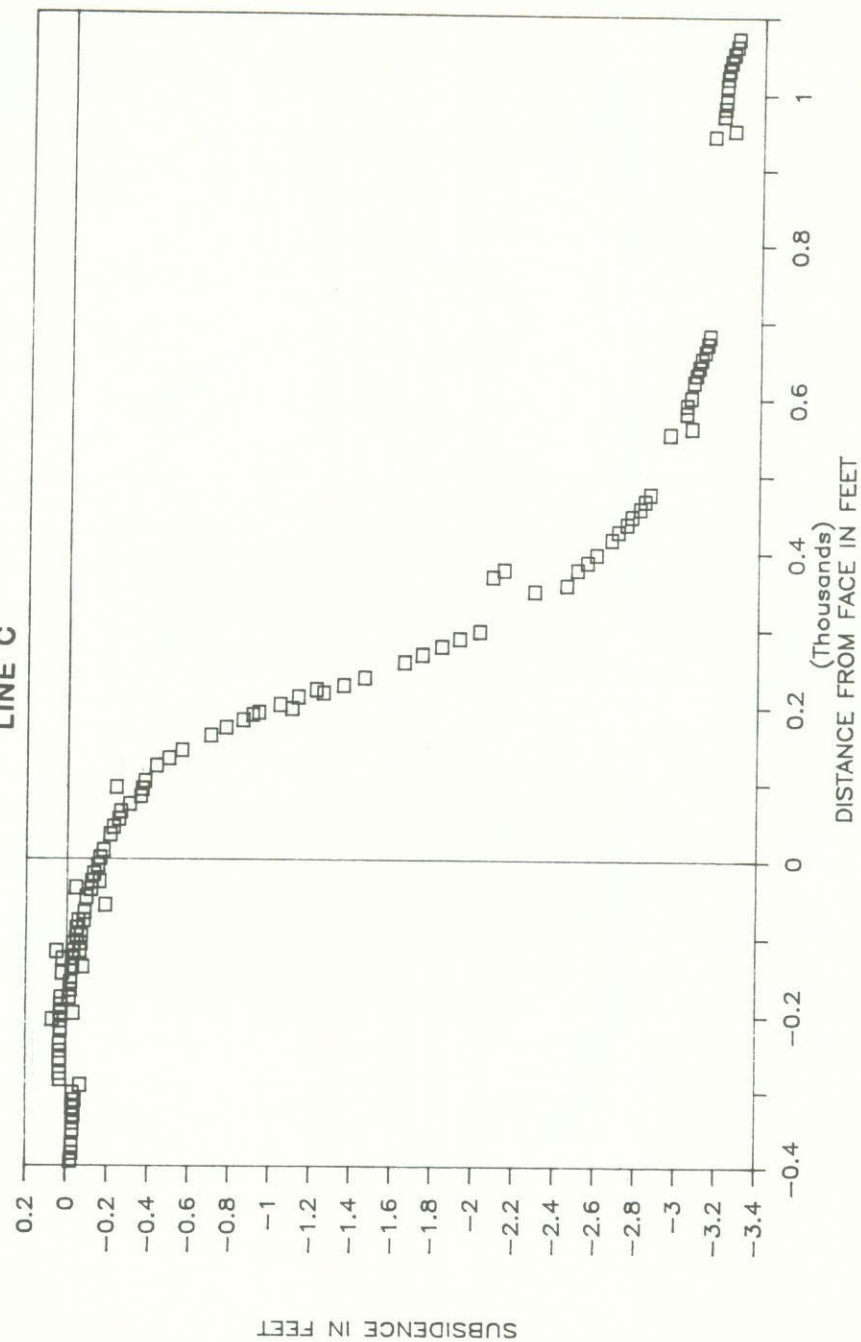


FIGURE 4.
FACE POSITION VS TDR RESPONSE
TDR #3 CABLE #1

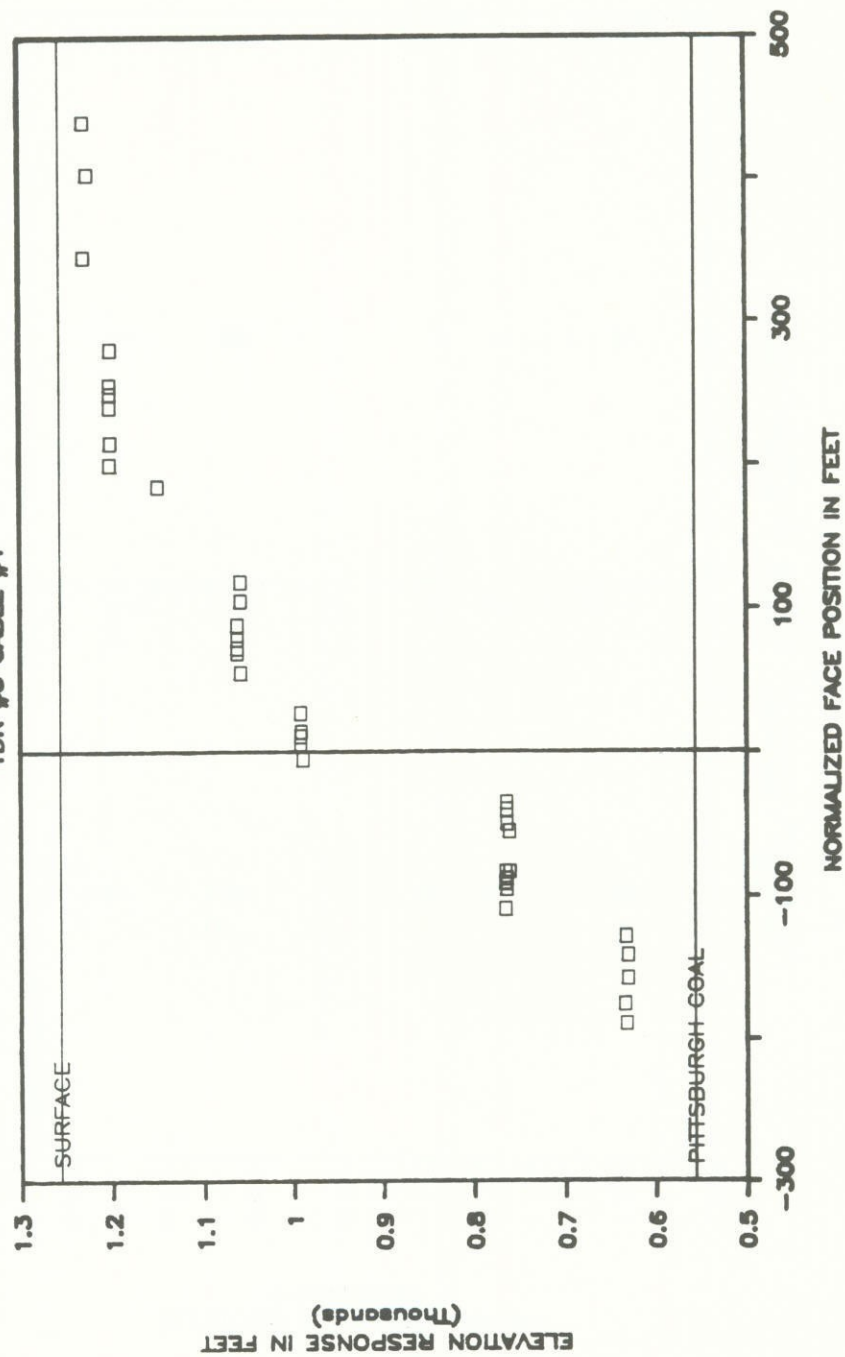


FIGURE 5.
TRANSVERSE TDR RESPONSE

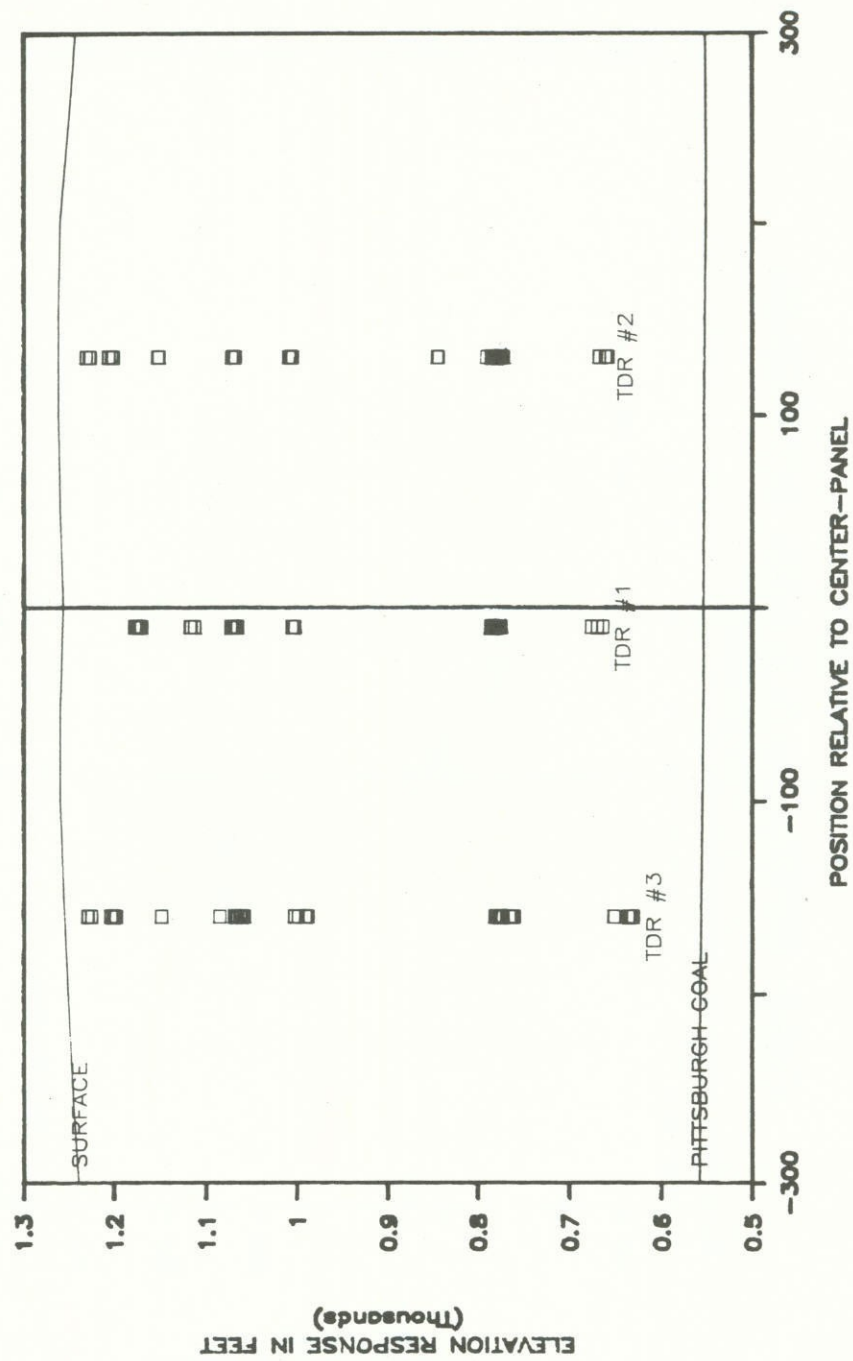
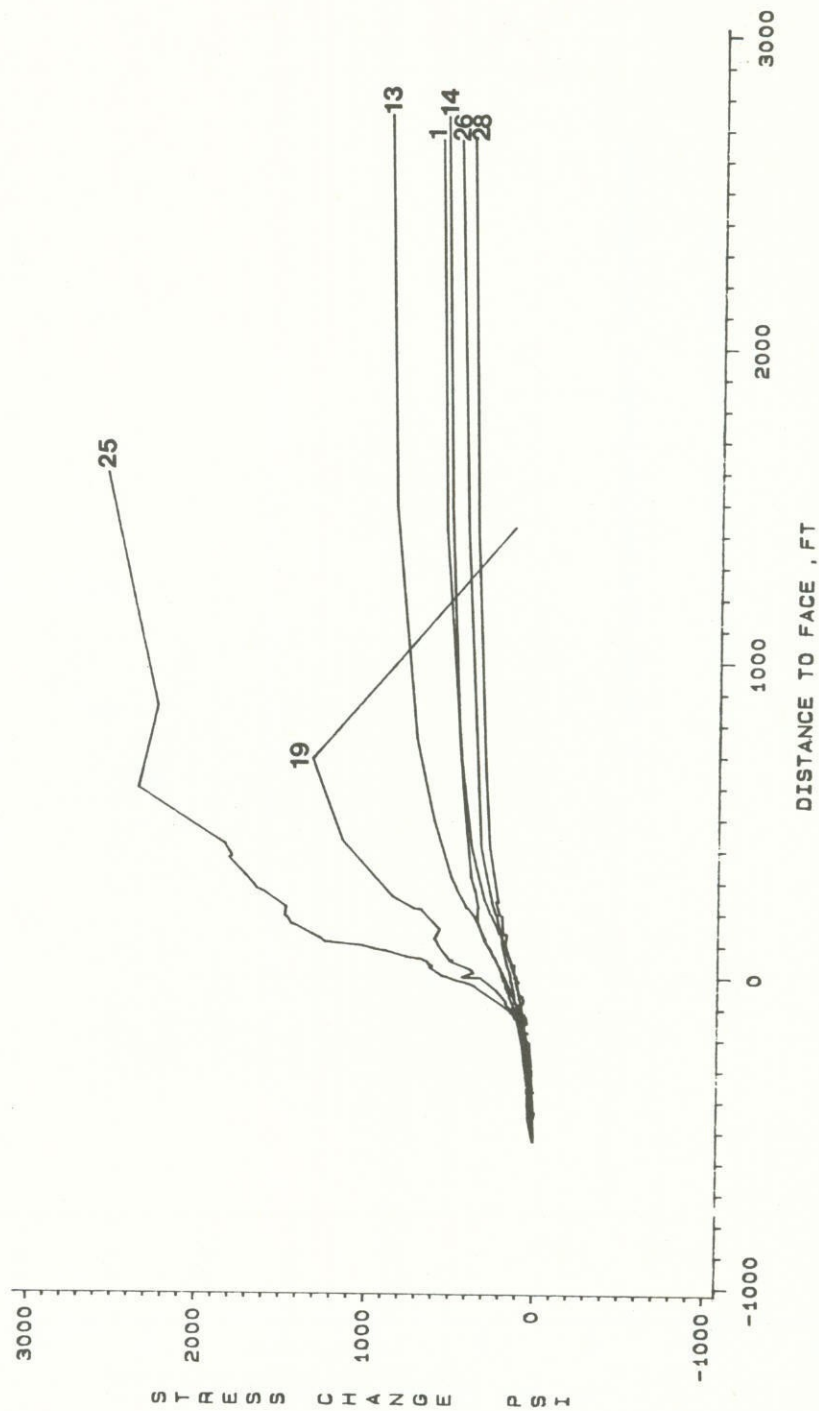


FIGURE 6.
STRESS CHANGE VS. FACE POSITION
STRESSMETER NOS. 1, 13, 14, 19, 25, 26, 28



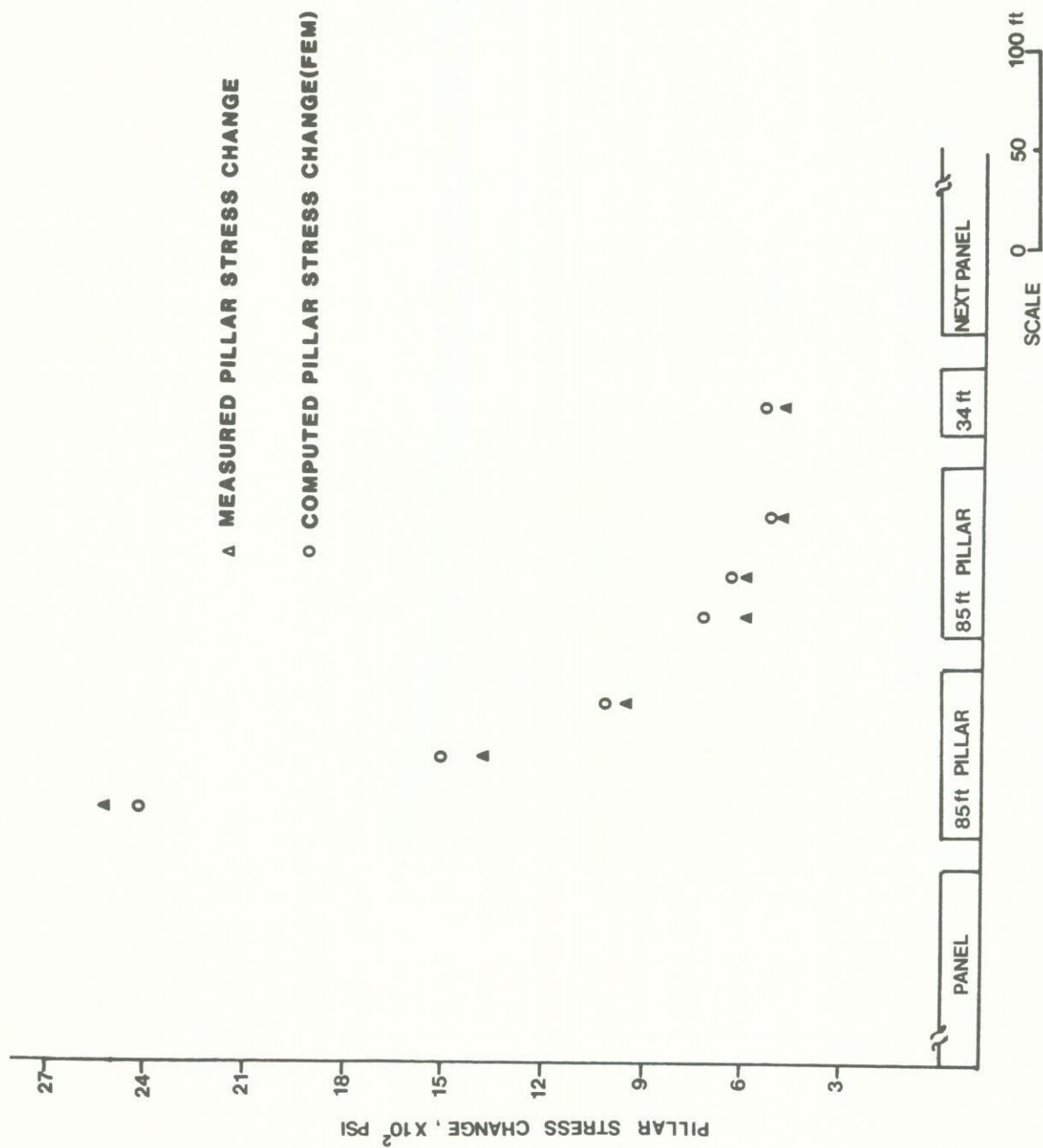
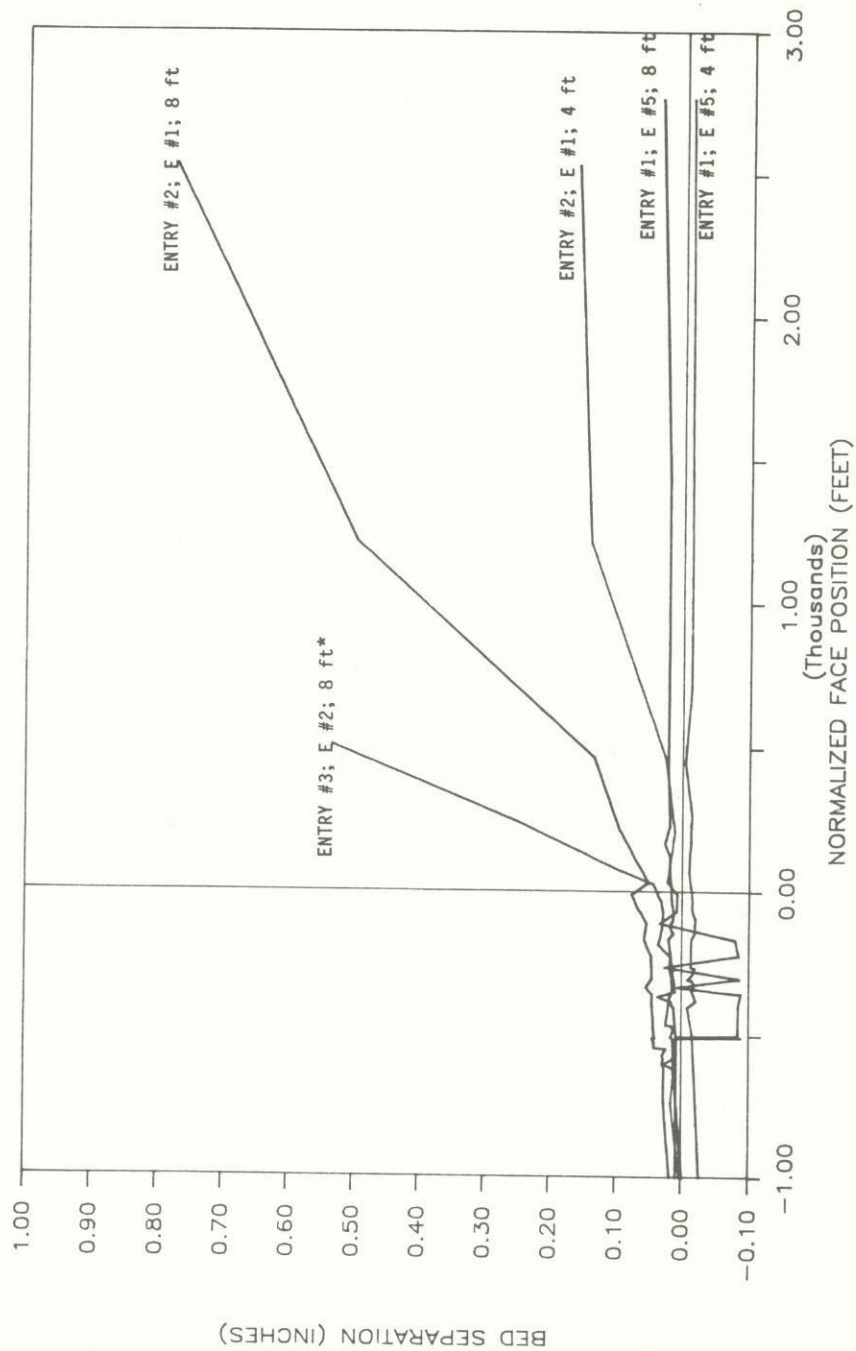


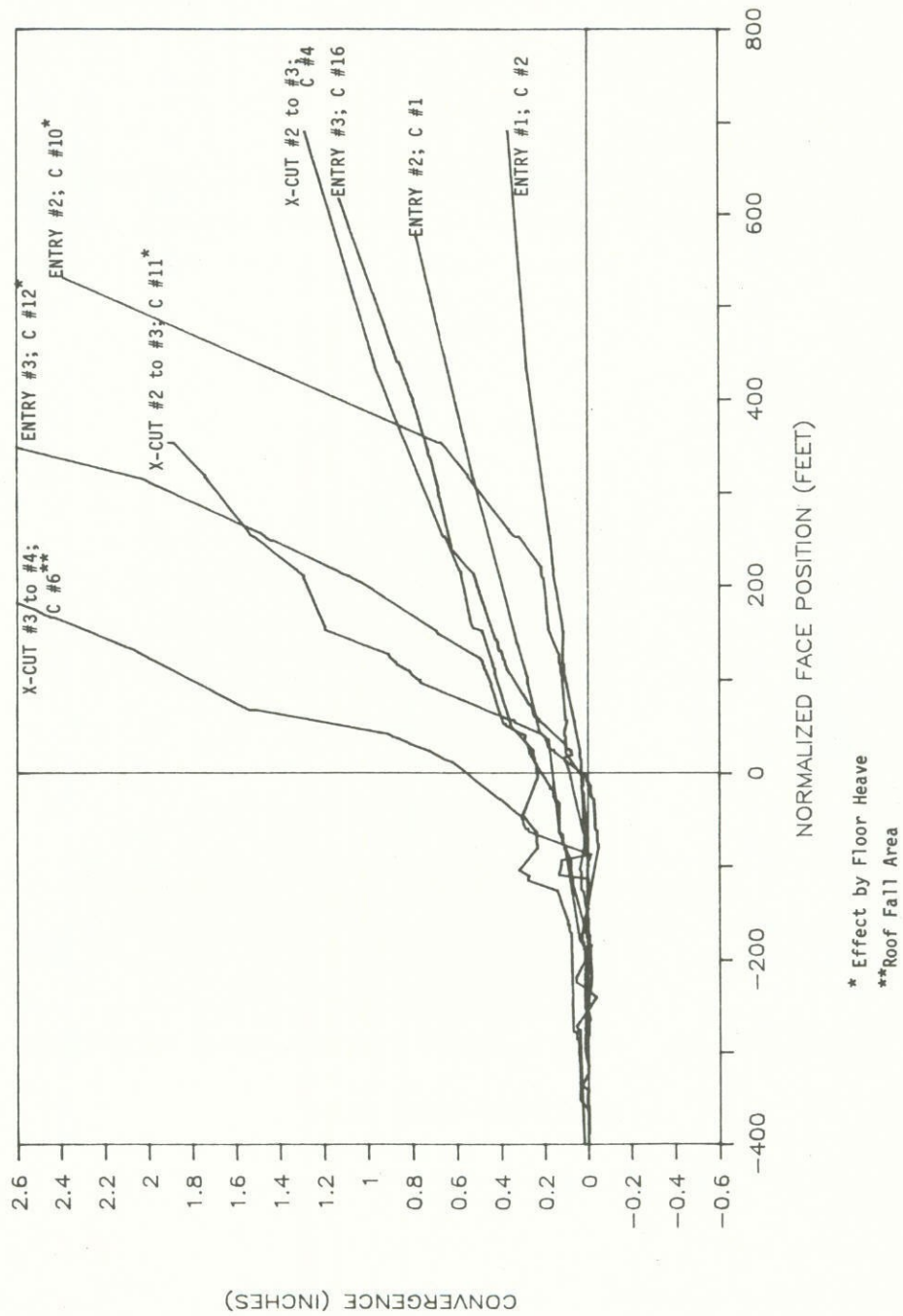
FIGURE 7. COMPARISONS BETWEEN MEASURED AND COMPUTED STRESS CHANGES

FIGURE 8.
ROOF BED SEPARATION RESPONSE



* Roof Fall Area

FIGURE 9.
ENTRY CONVERGENCE RESPONSE



zone 1-gob
zone 2-highly fractured,severely delaminated
zone 3-composite beam with slipping
along weak/strong rock interface
zone 4-surface layer

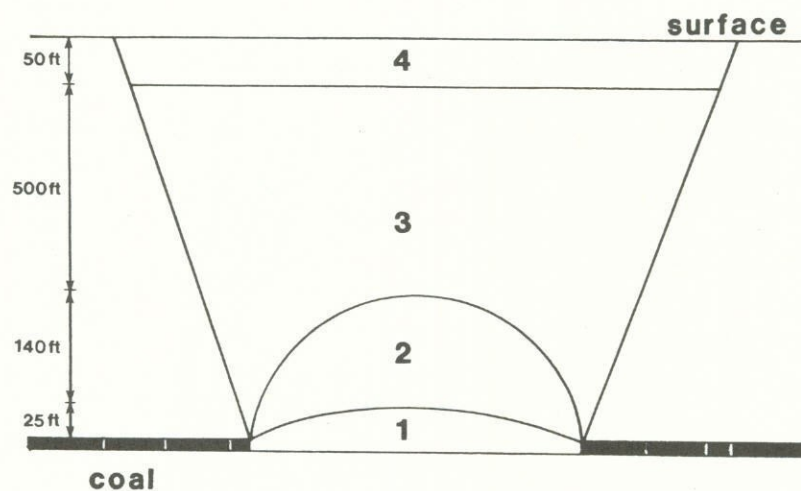


FIGURE 10. PRELIMINARY OVERBURDEN RESPONSE MODEL
FOR THE TEST SITE



Notes