

**ROCK MECHANICS IN
PRODUCTIVITY AND PROTECTION
25TH SYMPOSIUM ON ROCK MECHANICS**

ROCK MECHANICS IN PRODUCTIVITY AND PROTECTION

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Chapter 22

MINING-INDUCED STRESS MEASUREMENT WITH HYDRAULIC BOREHOLE PRESSURE CELLS

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ABSTRACT

This paper proposes an unconventional, simple, and practical method of measuring mining-induced ground stresses with hydraulic borehole pressure cells. The method is a direct ground stress measuring technique. It does not require the value of deformation modulus for back-calculation of stresses; instead, it directly measures the ground stress in situ. The applicability and practicality of the technique are shown by two case studies. The technique may be applied to monitoring a stress buildup that may lead to mine roof failure or to rock and coal outbursts.

INTRODUCTION

The current state-of-the-art for ground stress determinations features overcoring stress relief and hydraulic fracturing techniques. These are indirect methods in which the magnitude of the stress is back-calculated from the elastic stress-strain relation by measuring strain or displacement in place, and the modulus of deformation in the laboratory. The in situ moduli of deformation of most types of rock are impossible to determine accurately in the laboratory. Furthermore, these batch methods are not suitable for monitoring continuous ground stress changes induced by mining and excavation. The proposed method, however, does not require the value of deformation modulus for back calculation of stresses. Instead, it directly measures the ground stress in situ. Therefore it is also suitable for monitoring the continuous change of in situ ground stress.

The author recently proposed a simple and practical method of measuring in situ existing ground pressure with hydraulic borehole pressure cells (Lu, 1981). In this paper, a technique to monitor continuous changes in mining-induced ground pressure with similar instrumentation is proposed. This paper describes the basic concepts and instrumentation. It also presents two case studies that verify the applicability and practicality of the technique.

BASIC CONCEPTS

The magnitude of biaxial ground pressures existing in a rock mass can be determined by pressure convergence tests using a combination of one cylindrical (CPC) and two pre-encapsulated flat (BPC) hydraulic borehole pressure cells installed in a single drillhole as shown by figure 1a. Based on the elastic theories of plates and thick walled cylinders, and by application of the elastic and viscoelastic correspondence principle, two equations were derived to calculate the biaxial stresses, N_1 and N_2 , existing in a viscoelastic rock mass (Lu, 1981):

$$N_1 + N_2 = P_{e-CPC} / (1 - \nu) \quad (1)$$

$$N_1 / N_2 = (Q - S) / (1 - Q \times S) \quad (2)$$

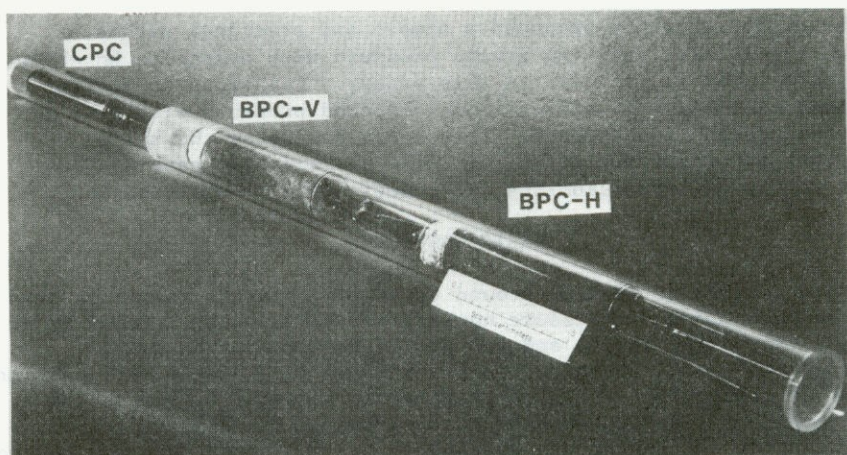
where P_{e-CPC} is the equilibrium pressure of CPC (fig. 2a), ν is Poisson's ratio of the rock mass, $Q = P_{e1} / P_{e2}$ where P_{e1} and P_{e2} are the equilibrium pressures of two BPC's (fig. 2a, P_{e-BH} and P_{e-BV}), and S is the transverse sensitivity of the BPC, which is 0.185 as calculated from the geometry of the current Bureau of Mines cells.

Using the same rationale, the changes in the in situ biaxial ground stresses induced by mining or excavation can be determined with the same instrumentation (fig. 1a). If the biaxial stresses are vertical and horizontal pressures as shown in figure 2b, the two equations required are

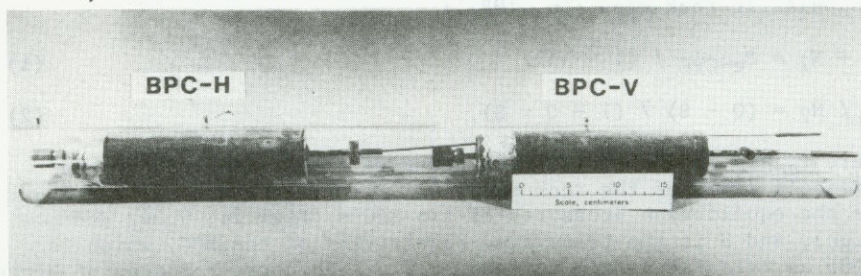
$$N_H + N_V = P_{CPC} / (1 - \nu) \quad (3)$$

$$N_H / N_V = (Q - S) / (1 - Q \times S) \quad (4)$$

where N_H and N_V are horizontal and vertical ground pressures respectively, P_{CPC} is the pressure reading of CPC, and $Q = P_{BH} / P_{BV}$ where P_{BH} and P_{BV} are horizontal and vertical BPC pressure readings respectively (fig. 2a). In this case, P_{CPC} , P_{BH} , and P_{BV} are equivalent to P_{e-CPC} , P_{e-BH} , and P_{e-BV} respectively at the time of gage reading. Thus with the cell pressure divergence curves of figure 2a, the ground pressure histories shown in figure 2b can be obtained.



a) Combination of one CPC and two BPC's in one hole



b) Combination of two BPC's in one hole

Fig. 1. Hydraulic borehole pressure cell packages.

The induced biaxial ground stresses (say, vertical and horizontal pressures) also can be measured and determined by a combination of only two BPC's installed in a single drillhole, as shown in figure 1b, if the pressure response ratio K , defined as the ratio of P_{BH} or P_{BV} to the effective directional ground pressure, is known. This response ratio K is determined by

$$K = P_{BH} / (N_H + S \times N_V)$$

or

$$K = P_{BV} / (N_V + S \times N_H)$$

(5)

where N_H and N_V are derived from equations (1) and (2) or (3) and (4).

Therefore, with known values of K and S , N_V and N_H can be calculated using equation (5) and the P_{BV} , P_{BH} readings. The value of K varies with the rock type. For example, K_{coal} (tested with Pittsburgh seam) is 1.00 ± 0.05 within a 95-pct confidence interval

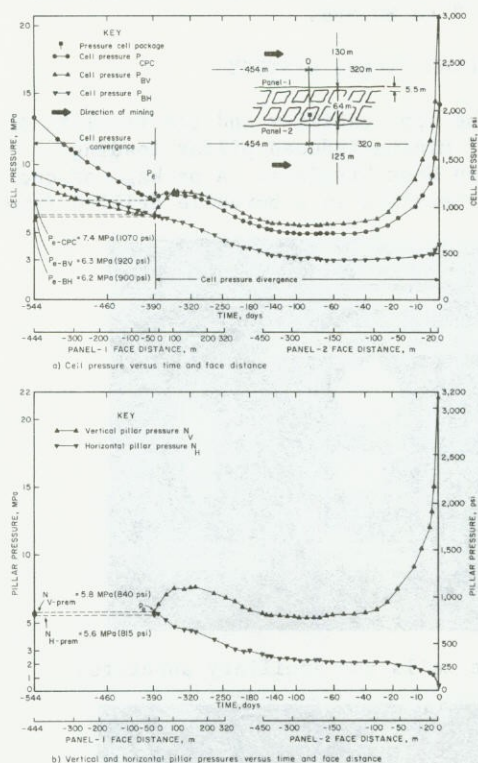


Fig. 2. Mining induced pillar loading.

which is forced against the wall of the drillhole.

The BPC has a 59.3-mm diameter capsule and is designed for a BX-size (60-mm) hole. The other parts of the system are those described above for use with the CPC. The flat cell body is made of 1.52-mm-thick mild steel and is 20.3 cm long, 5.08 cm wide, and 8.9 mm high. The same fluid used in the CPC is forced into the cell to inflate it and press it against the Portland cement capsule. Two 8.4-mm-diameter holes are provided in the capsule for a tubing passage (fig. 3).

A smooth-walled drillhole with a precise diameter is critical for using this instrumentation; every effort should be made to keep the tolerance within 1.0 mm. The Bureau of Mines has developed a combination bit for drilling smooth and precise instrumentation holes in soft rock by joining a drag bit and a diamond bit (Ferrari, 1982). The sketch and the assembly of the bits are shown in figure 4.

from 24 tests, and K_{potash} (Carlsbad, New Mexico, potash ore) with 3 tests is in the range of 0.535 to 0.549 with an average of 0.54.

Also with known values of K and S , the uniaxial rock pressure can be measured with a single BPC installation, and calculated from equation (5).

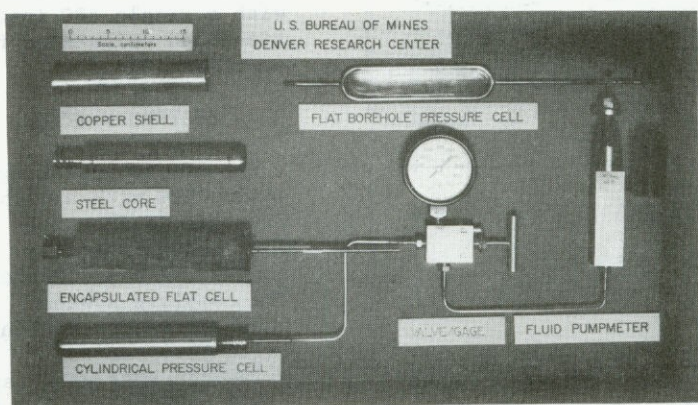
INSTRUMENTATION

The pressure-sensing device CPC with a 37.5-mm diameter is installed in the 38.0-mm-diameter portion of a drillhole. The complete system consists of CPC, tubing, valve-gage and/or valve-recorder unit, and fluid pumpmeter. The CPC is 20.3 cm long and has an effective length of 17.8 cm when installed in a drillhole. Glycerin or hydraulic oil is forced through the steel tubing of 6.35 mm I.D. into the annulus between the steel core and the copper shell,

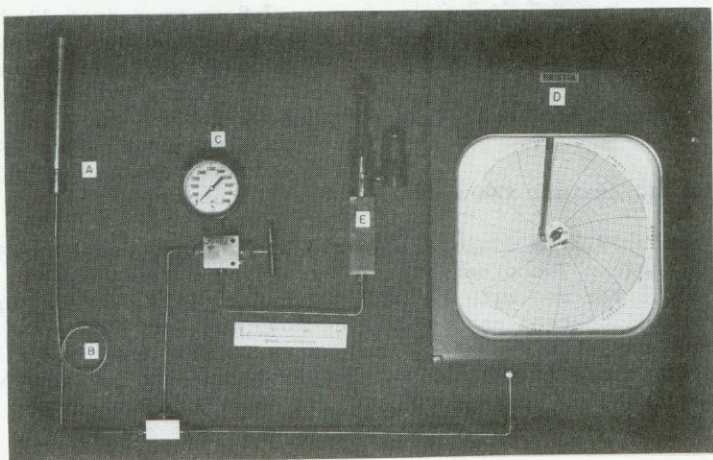
With the current instrumentation, the sensitivity is 0.1 MPa and the measurable pressure range is 2 to 80 MPa.

MINING-INDUCED PILLAR STRESS HISTORY

The first example of proving the applicability and practicality of this technique is a case study of mining-induced pillar loading history at a longwall coal mine in West Virginia. A package of one CPC and two BPC's (fig. 1a) was installed in a borehole drilled into



a) Hydraulic borehole pressure cells and ancillary apparatus



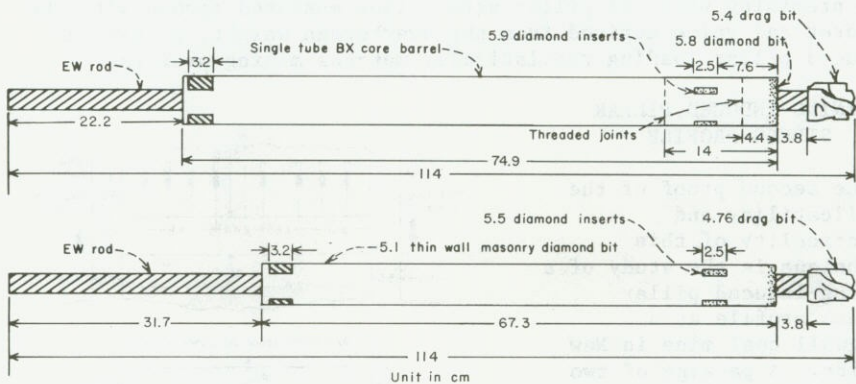
b) CPC system assembly, A) CPC; B) Tubing; C) Valve-gage unit; D) Recorder; E) Fluid pumpmeter

Fig. 3. Hydraulic borehole pressure cell equipment (Lu, 1981).

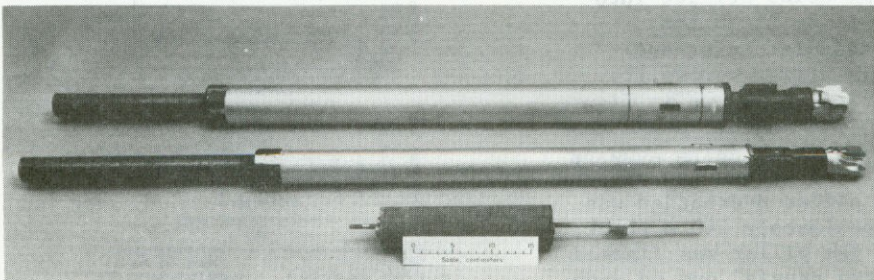
the center of an interpanel chain pillar from the entry side (fig. 2a). All the cells were connected to hydraulic pressure recorders with steel tubing to provide continuous pressure readings. The pillar with dimensions of 25 by 30 m was under 230 m overburden.

The cell pressures, initially set at 14 MPa (2,000 psi), required approximately 160 days to converge to equilibrium pressures (P_e 's, fig. 2a) at -35 m face distance (negative distance indicates that the instrumentation plane is in the solid area) of Panel-1. Thereon the cell pressures diverged in accordance with mining progress until the cells were destroyed at zero face distance of Panel-2.

Thus from the cell pressure convergence curves and cell equilibrium pressures (fig. 2a, left of point P_e), the premining horizontal and vertical pillar pressures are calculated with equations (1) and (2) and plotted in fig. 2b (left of P_e). Then from the cell pressure divergence curves (fig. 2a, right of P_e), the mining-induced vertical and horizontal pillar pressures are calculated with equations (3) and (4) and plotted in figure 2b, right of P_e .



a) Sketch of the bits



b) Bit assembly

Fig. 4. Combination precision bits in two sizes (Ferrari, 1982).

Exactly the same pillar-loading curves of figure 2b (right of P_e) are obtained from the readings of the two BPC's of the above mentioned cell package, by using the value of K derived from equation (5) and P_{e-CPC} , P_{e-BH} , and P_{e-BV} readings. The value of K for this particular cell package was 0.94.

While the Panel-1 face was approaching, the vertical stress of this pillar started to increase at -35 m face distance. The stress reached a peak, which was approximately 30 pct higher than the premining stress, when the Panel-1 face passed 160 m. Then the stress gradually dropped to 95 pct of the premining level. When the Panel-2 face was approaching, the vertical stress again started to increase rapidly at -20 m face distance, and reached the maximum at zero face distance, which was 375 pct of the premining stress. The horizontal pillar stress decreased gradually from the time the Panel-1 face approached to -35 m distance through the remaining life of that pillar (fig. 2b).

The results revealed three significant facts: 1) the 3-cell and 2-cell methods give the same answer for the mining-induced stress, 2) the premining vertical pillar stress thus measured agrees with the theoretical value derived from the overburden weight, 3) mining-induced pillar loading realistically matches mining progress.

MINING-INDUCED PILLAR STRESS PROFILE

The second proof of the applicability and practicality of this technique is the study of a mining-induced pillar stress profile at a longwall coal mine in New Mexico. A package of two BPC's (fig. 1b) was installed in a series of 60-mm holes drilled into the 12.8- by 30.5-m rectangular interpanel chain pillar across its section, as shown in figure 5a. The objective was to obtain progressive loading profiles across the pillar. The pillar overburden was approximately 240 m thick, and the pillar was flanked by two panels, one on each side. Panel-1 was 168 m wide and Panel-2 was 183 m wide. The interpanel

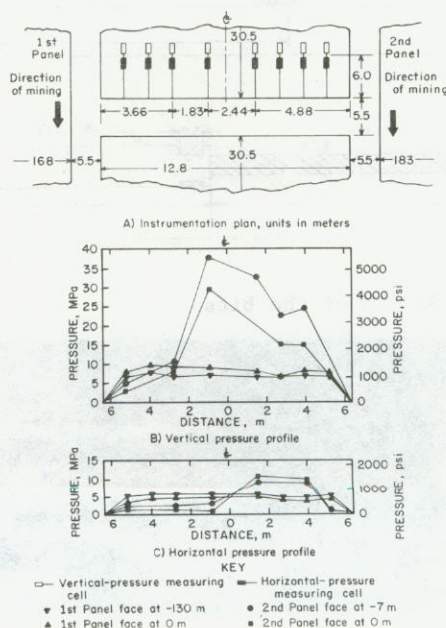


Fig. 5. Mining-induced pressure profiles across the pillar section.

entries were 5.5 m wide. All cell pressure readings were recorded with pressure recorders.

Using equation 5, and the value of BPC response ratio $K = 0.93$ (determined from the previous test conducted at the same mine), the individual values of N_V and N_H were calculated from the corresponding cell pressure readings, P_{BV} and P_{BH} . From these calculations, the pressure profiles at four selected stages of mining progress were obtained. These are plotted on figures 5b and 5c.

These profiles reveal that the stress distributions across the pillar section at those particular stages of mining progress are realistic. They also clearly indicate the load transfer from the pillar edges (yielded zones) to the pillar core in accordance with mining progress.

CONCLUSIONS

The proposed method is simple and practical. The equipment required is nonsophisticated and inexpensive. It does not require the value for deformation modulus of the rock for back calculation of stresses. The method directly measures the ground stress in situ. The two examples using directly measured mining-induced pillar stresses prove the applicability and practicality of the technique. This technique may have many other applications, such as monitoring a stress buildup that may lead to mine roof failure or to rock and coal outbursts.

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12

MINING GROUND CONTROL

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Chapter 54

INSTRUMENTATION AND MONITORING FOR PILLAR EXTRACTION IN A DEEP, FAULTED URANIUM MINE

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ABSTRACT

A rock mechanics instrumentation and monitoring program was implemented during pillar extraction at Gulf Mineral Resources' Mt. Taylor Mine, the deepest uranium mine in the U.S. Three types of monitoring were employed: (1) drift convergence around stopes (using a portable tube extensometer), (2) stress changes in pillars (using vibrating wire stressmeters in horizontal boreholes), and (3) load changes in haulageways under stopes (using vibrating wire load cells in jack stands).

Results include convergence-time graphs, convergence contour maps, stress-time graphs, a stress increase contour map, load-time graphs, and abutment load limit maps. Major factors influencing results include (1) size of pillars around the extraction area, and (2) presence and orientation of a major fault adjacent to one extraction area. These factors, combined with pillar extraction, produced evidence of load transfer over 120 m from a stope measuring only 55 m x 45 m. Vertical stress increases of 7 to 17 MPa (1000 to 2500 psi) within 60 m of the stope were obtained using the stress-meter manufacturer's method. The fault apparently prevented load transfer in the down-dip direction but increased load transfer in the up-dip direction. Convergence rates of up to 1.3 cm/day (0.5 in/day) were measured at two stations. Instruments were generally reliable despite adverse underground conditions.

The instrumentation and monitoring program at Mt. Taylor provided a warning system against ground control problems and a characterization of ground behavior for future development and extraction of pillars. Recommendations are made for future mining and instrumentation practices.

INTRODUCTION

Initial development of the Mt. Taylor Mine, owned by Gulf Mineral Resources Company, was started during the uranium boom of the early to mid-1970's. The mine is located about 30 km northeast of Grants, New Mexico, and just southeast of the Ambrosia Lake uranium district. Mt. Taylor differs from the Ambrosia Lake mines in its greater depth, harder rock, higher ore grade, and hot, wet underground conditions. Important background data on the mine is summarized in Table 1. Mining was discontinued at Mt. Taylor in December 1982 due to poor market conditions, but not before a number of interesting and challenging mining conditions were observed.

Although operations at Mt. Taylor were still in a developmental stage, a trial program for pillar extraction was started to provide a basis for evaluation of future mining costs and profits. The three pillar extraction areas are shown in Fig. 1. Because of the unpredictable consequences of pillar extraction at 950 m depth, a rock mechanics instrumentation and monitoring program was considered essential. The general mining method and instrumentation scheme are illustrated in Fig. 2.

This paper summarizes the objectives, design, and results of the instrumentation program. Problems with various types of instrumentation are also mentioned. Results provide a basis for characterizing rock mass behavior and for recommendations on mining practices during future development and extraction of pillars.

OBJECTIVES AND DESIGN OF INSTRUMENTATION PROGRAM

Three types of instrumentation were employed: (1) convergence of drifts (shortening of pillars) around extraction areas, (2) stress changes in pillars, and (3) load changes in haulageways under extraction areas (stopes).

Convergence monitoring has been recognized as an economical and reliable basis for evaluating the stability of underground openings. Convergence measurements on the ore level at Mt. Taylor provided a warning system against ground control problems and an indication of the horizontal extent of ground affected by extraction.

Most of the convergence stations at Mt. Taylor consisted of Swellex bolts point-anchored at about .75 and 1.5 m into the roof and a 1.8 m fully anchored bolt in the floor. These points were monitored with a portable tube extensometer. Roof to floor tape measurements on 5 cm bolts provided adequate monitoring data in a rapidly converging area just north of the Zone 15 stope (Fig. 1). Most stations were placed within .5 m of pillar sides and away from intersections in order to minimize roof sag and floor heave effects.

Stress monitoring is a valuable tool for evaluating the effects of changing the configuration of a mine. The spatial distribution, time change patterns, and estimated magnitudes of stress can be used in characterizing ground behavior. Stress changes were monitored with vibrating wire stressmeters in horizontal boreholes of various lengths in pillars of various sizes. Holes were drilled with a skid-mounted longhole machine to ensure accurate hole diameters for

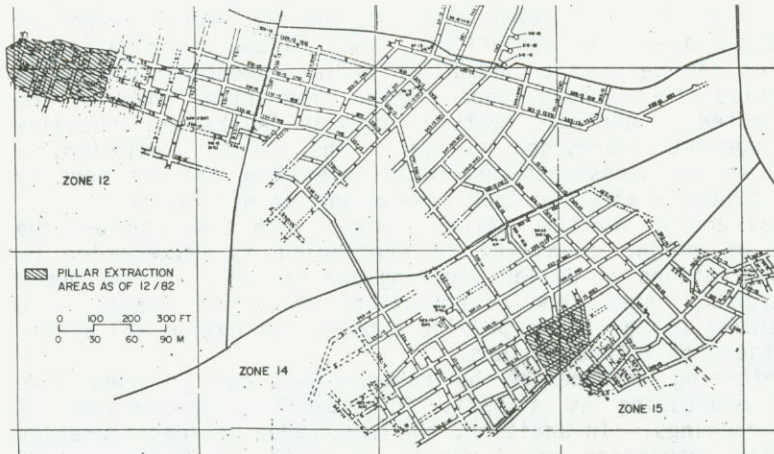


FIGURE 1 ORE LEVEL MAP SHOWING 3 PILLAR EXTRACTION AREAS

Table 1. Background Data on Mt. Taylor Mine

<u>Depth</u>	Ore level: 3100 ± 30 ft Haulage level: 3200 ft
<u>Mining Methods</u>	Drill & blast, modified room & pillar.
<u>Access</u>	Two vertical shafts, 14 ft & 24 ft diameter.
<u>Overburden Geology</u>	Flat-lying sandstones, siltstones, mudstones, and shales of varying thicknesses and strengths, including Mancos, Tres Hermanos, Dakota, and Brushy Basin formations at 2100 to 3000 ft depth.
<u>Rock Types at 3000 to 3250 ft</u>	Sandstones & mudstones of the Westwater Formation
<u>In Situ Stresses</u>	$\sigma_1 = \sigma_2 = 3400$ psi (estimated) $\sigma_3 = \sigma_h = 2400$ psi (from hydrofracturing)
<u>Intact Rock Compressive Strength (Sandstone, 3100 Level Ore Zone)</u>	Ore: 3800 to 9000 psi, average 6300 psi Waste: 1300 to 2500 psi, average 1800 psi
<u>Approximate Average Pillar Sizes</u>	Barrier pillars: 10 ft high, 80 ft x 80 ft, variable Stope pillars: 10 ft high, 90 ft x 50 ft and 100 ft x 50 ft, variable.
<u>Nominal Drift Size</u>	10 ft x 10 ft
<u>Ground Support</u>	Split sets & wire mesh throughout, square timber sets (3100 Level) or steel sets (3200 Level) in fault zones.

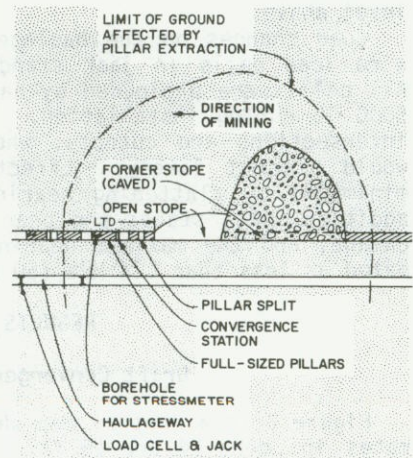


FIGURE 2 PROFILE OF GENERAL MINING METHOD & INSTRUMENTATION SCHEME & ILLUSTRATION OF LOAD TRANSFER DISTANCE

proper stressmeter anchorage. All stressmeters were preloaded during installation.

With an adequate number of stressmeters installed early, vertical stress changes in pillars can provide an indication of the maximum distance and distribution of load transfer. The load transfer distance (LTD, Fig. 2) is a measurement of the horizontal extent of ground affected by pillar extraction. The LTD and distribution of load transfer can be used to establish pillar dimensions and other mining practices for various stope widths. The actual shape of the arch is affected by geologic features such as faults and unusually weak or competent overburden strata. The Dakota Formation, a sequence of strong sandstones and shales overlying the ore zone by 45 to 60 m, probably affected arch development at Mt. Taylor.

Horizontal and/or vertical stress changes with time can provide indications of pillar stability if monitoring is accompanied by adequate observations of pillar yielding and failure. It was hoped that the trends, rates, and distributions of stress change in a typical yielding pillar could provide a basis for design and stability evaluations of future pillars.

Load monitoring accompanied by observations can be used like convergence monitoring as a basis for stability evaluations of underground openings. In addition, the magnitudes of load increases due to pillar extraction could provide a basis for future ground support design. Consistently measured load increases could be used as future design criteria for steel sets needed to keep haulageways open under abutment loading. At Mt. Taylor, load monitoring in 3200 Level haulageways was considered necessary because of the absence of steel support in most areas affected by pillar extraction on the ore level above.

Load changes on the haulage level were monitored with vibrating wire load cells in jack stands with 45 cm square bearing plates. All cells were preloaded by hand-tightening screw jacks to at least 5000 lb during installation. Most of these devices were placed at intersections and cutouts, where early indications of overloading would be most likely. Effects of localized ground failures were minimized by flattening bearing surfaces before installation. In addition, jack stands and bearing areas were repeatedly checked for damage, and the maximum bearing stress during monitoring was estimated as less than .34 MPa (50 psi).

RESULTS OF MONITORING

Drift Convergence/Pillar Shortening

Figure 3 is a contour map showing the distribution of convergence rates in Zones 14 and 15 for consecutive 8 and 6 week periods. A comparison of contour positions indicates significant increases in convergence between 20 and 60 m from the Zone 14 stope. This data suggests that the pressure arch above the Zone 14 stope was spreading outwards, while the stope dimensions did not increase significantly. Some spreading of the arch to the northwest may have been due to the added effects of load transfer from the Zone 15 stope.

The southern part of Zone 15 was the area of maximum convergence in the entire mine. This area experienced gradual convergence of up to .5 to 1 m in some places even before extraction began. Ground failures were characterized by heaving of floors, cracking of floors parallel to sides of pillars, spalling of pillars, and occasional fall of roof slabs near pillars. These conditions appeared to be induced by swelling of wet shale just beneath the floor of low-lying, poorly drained drifts. Thus, the high convergence rates in Zone 15 were apparently independent of pillar extraction in Zone 14.

The convergence contours in Fig. 3 are generally deflected away from apparently rigid pillars. ("Rigid" pillars are considered large enough to experience negligible deformations, i.e., deformations smaller than those due to load transfer.) The smallest of these rigid pillars measures approximately 24 m x 21 m. Smaller pillars, based on their appearance in the mine, were yielding. Convergence data also suggests that drifts not influenced by pillar extraction typically converges at a rate of .003 to .013 cm/day (.001 to .005 inch/day), while pillars lying within the zone of influence of extraction shorten at rates greater than .013 cm/day (.005 inch/day).

Graphs of convergence vs time led to the following observations on ground behavior:

1. The graphs show a general cyclic behavior of accelerating and decelerating trends.
2. Convergence graphs for most stations showed that slight separations developed between the .75 and 1.5 m roof anchors.
3. The end of accelerating periods at stations near the stope often coincided with caving of the stope.
4. A maximum convergence rate of about 1.3 cm/day (0.5 inch/day) was measured by extensometer readings at Station 12 (just before an adjacent pillar was extracted) and by tape measurements just northeast of the Zone 15 stope (Fig. 3).

The latter two observations indicate that trends and rates of convergence provide warnings of potential ground control problems.

Level surveys were initially used to monitor floor elevation changes, but damage by LHDs to a few floor points disrupted level loops on several occasions. From the available data it appears that pillar shortening in Zone 14 occurred by simultaneous upward displacement of the base and downward displacement of the top.

Stress Changes in Pillars

Interpretation of stressmeter data is complicated by the different installation times and monitoring periods of the gages. Some stressmeters were installed just before pillars were affected by load transfer, and other stressmeters were installed during periods of load transfer. Monitoring periods for five stressmeters were literally "cut short" by shear displacement across the borehole, breaking the cables. Despite these problems, stressmeter data provides valuable information on rigid and yielding pillar behavior and on load transfer and arch development away from the stope.

Figure 4 is a contour map showing an interpretation of the general distribution of vertical stress increase in Zone 14. Data is plotted as rates of stress increase in order to obtain comparable magnitudes from stressmeters which had different monitoring periods. Stress contours indicate that a stress concentrated zone developed west of the stope. The maximum stress concentration apparently developed at the boundary between the most rigid and the most yielding pillars. Observations and stress contours suggested that pillars yielded in an area measuring roughly 90 m x 90 m. The small pillars along the southwest and southeast edges of this area shed load to the adjacent larger, more rigid pillars.

The distribution of vertical stress change around the Zone 14 stope appears to have been influenced by the following factors: (1) transfer of overburden load from above the stope; (2) yielding of pillars west and southwest of the stope, resulting in greater load transfer in these directions; (3) presence and orientation (south-eastward dip) of the fault southeast of the stope, resulting in load transfer to the northwest, but preventing load transfer to the southeast; (4) presence of rigid pillars north of the stope and on three sides of the small pillar experiencing the maximum concentration of stress increase; and (5) possible variations in rock strength throughout the stope area, as indicated by compressive strength tests elsewhere in the ore zone.

The effect of the major fault (between Zones 14 and 15) on stress change distribution is an important factor. Both stressmeter data and visual observations suggested that the pressure arch above the Zone 14 stope was modified by the fault, as shown in Fig. 5. Stressmeter 5 showed no vertical stress increase as pillars were extracted and stopes were caving less than 60 m to the west. Stressmeter 6, in the core of the same pillar as Stressmeter 5, showed a steady decrease in horizontal stress which was apparently not related to load transfer. Observations before the beginning of pillar extraction suggested that the high convergence rates in the southern part of Zone 15 were primarily due to floor heave induced by swelling shale. Convergence in this area apparently occurred independently of pillar extraction in Zone 14.

Figure 6 is a graph of stress change vs time for a yielding pillar, about 55 m west of the stope. Data from Stressmeters 8 & 9 showed that the core of the pillar loaded more rapidly than the outer shell, but the first indication of failure (stress decrease) occurred at about the same time in the core and shell of the pillar.

Stressmeters 7 (Fig. 6) and 16, oriented horizontally, apparently measured a decrease in confining stress during pillar yielding, followed by localized increases in confinement as the pillars developed residual strength.

Load Changes on 3200 Level

Results of load monitoring in haulageways on the 3200 Level include (1) a map of apparent limits of pressure arch abutment loads and (2) load vs time data for each load cell. Load-time data indicated that cells loading at more than 30 lb/day were under abutment

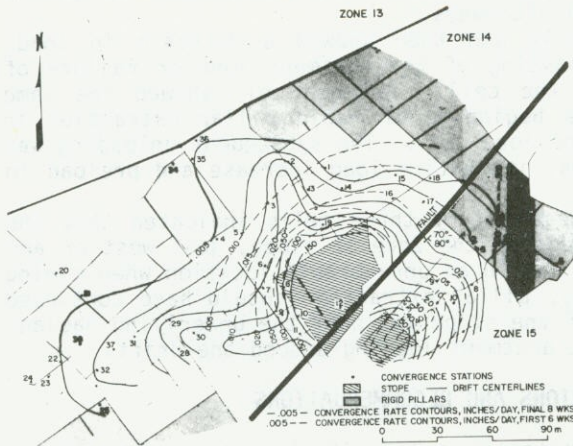


FIGURE 3 CONVERGENCE RATE CONTOUR MAP FOR FIRST 8 WEEKS & FINAL 6 WEEKS OF MONITORING

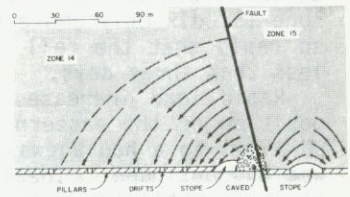


FIGURE 5 APPARENT EFFECT OF A MAJOR FAULT ON ARCH DEVELOPMENT & LOAD TRANSFER IN SECTION THROUGH ZONE 14 & 15 EXTRACTION AREA

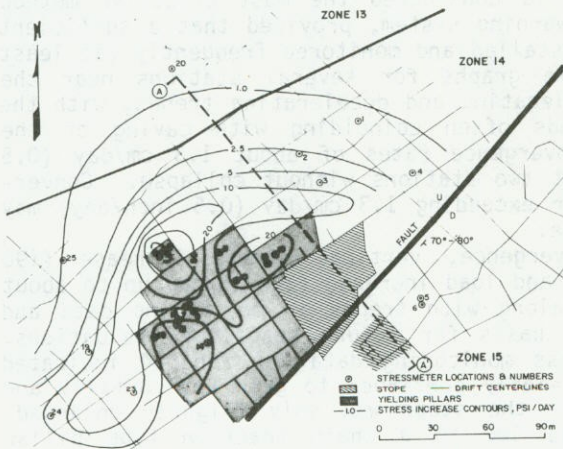


FIGURE 4 CONTOUR MAP OF RATE OF VERTICAL STRESS INCREASE IN PILLARS SECTION A-A' CORRESPONDS TO FIGURE 5

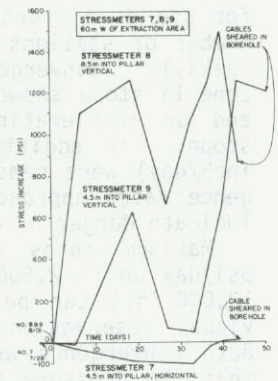


FIGURE 6 STRESS-TIME GRAPH FOR A YIELDING PILLAR WEST OF THE ZONE 14 EXTRACTION AREA

loads. Lesser loading rates were probably due to normal convergence of drifts lying beyond the influence of pillar yielding and extraction. A comparison of total load increases at three cells showed that a maximum load increase of up to about 12,000 lb can be expected in drifts lying near the middle of the abutment zone.

Graphs of load vs time for load cells in each of the three extraction areas were also used to study localized effects of pillar extraction on underlying haulageways.

One cell loaded steadily and then showed a decrease in load, probably indicating the passing of the abutment zone or failure of ground immediately above the cell. Another cell showed the same steady increase until the beginning of rapid pillar extraction in Zone 15, directly above the load cell. The subsequent unloading was so rapid that the cell lost its entire load increase and preload in less than three days.

Rapid load increases measured by three cells indicated that the position of the eastern edge of the Zone 12 stope, just west of and parallel to a haulageway, was in an unfavorable location when mining was discontinued. Ideally, pillar extraction should have continued at least until the edge of the stope was past the underlying haulage drift, in order to relieve abutment loading around the drift.

CONCLUSIONS AND RECOMMENDATIONS

Rock mechanics instrumentation and monitoring at the Mt. Taylor Mine was successful in providing a warning system for impending ground control problems and a characterization of ground behavior during development and extraction of pillars.

Convergence monitoring is considered the most effective method for use as a short term warning system, provided that a sufficient number of stations are installed and monitored frequently (at least weekly). Convergence-time graphs for several stations near the Zone 14 stope showed accelerating and decelerating trends, with the end of accelerating trends often coinciding with caving of the stope. In addition, convergence rates of about 1.3 cm/day (0.5 inch/day) were measured at two stations without collapse. Convergence rates approaching or exceeding 1.3 cm/day (0.5 inch/day) may indicate danger of collapse.

Maximum rates of convergence, vertical stress increase (190 psi/day up to 2,500 psi), and load increase (170 lb/day up to about 12,000 lb.) can be used along with trends in monitoring data and visual observations as a basis for future stability evaluations. Both convergence and stress monitoring data for Zone 14 indicated that the pressure arch steadily continued to grow horizontally and possibly vertically, while the stope was only slightly enlarged. Continued mining may have led to a chain reaction type pillar squeeze originating southwest of the stope, where yielding of several small pillars allowed spreading of the pressure arch. Such a squeeze could prevent economical recovery of up to 30,000 tons of high grade ore.

Table 2 is a summary of load transfer distance (LTD) determinations from convergence and stress increase contour maps (Figs. 3 and

4). A comparison of LTDs from convergence and stress data shows that stress changes in the central cores of pillars are a much more sensitive indicator than shortening of pillars. Also the LTDs for Zone 14 show that the influence of the small pillars west and southwest of the stope was greater than the influence of the fault. (LTDs measured to the west and southwest from the stope, toward small pillars, are greater than LTDs measured to the northwest of the stope, opposite the dip of the fault.)

Table 2. Load Transfer Distance (LTD) Determined from Convergence and Pillar Stress Monitoring

Extraction Area Location & Avg. Width During Monitoring	Direction from Stope	Approx. LTD (m)*	
		from Convergence	from Stressmeters
Zone 14 40 to 45 m	SE	**	0
	NE	45	60
	N	30	75
	NW	75 to 90	120
	W	120 to 140	140 to 150
	SW	120	150 to 180
Zone 12 30 to 45 m	E	<60	90
Zone 15 0 to 25 m	NE	**	<30

* Note that LTD is measured from the perimeter of the extraction area to the furthest point (convergence or stress contour, Fig. 3 or 4) showing the influence of pillar extraction.

** Insufficient data for LTD determination.

Instrumentation and monitoring, including visual observations, provided a basis for the following recommendations for future mining practices at Mt. Taylor and similar mines:

1. To minimize the possibility of loosening already yielded ground during pillar extraction, the general direction of mining should be away from the weakest ground (i.e., away from the fault and small pillars).
2. To avoid overloading pillars and control the spreading of the pressure arch, pillars should not be split more than one row ahead of the extraction area.
3. Pillars 3 m high and intended to be rigid (nonyielding) should have minimum dimensions of about 25 m x 25 m (or equivalent area) near stopes up to 45 m wide, where one edge of the stope coincides with a major fault.
4. Barrier pillars adjacent to stopes and up-dip of major faults should be designed to carry the overburden stress plus a stress increase due to the pressure arch above the stope.

5. If pillar extraction is to be discontinued, the eventual limits of extraction should ideally coincide with rigid pillars and should pass beyond any underlying haulageways.

Future underground instrumentation programs can take advantage of the experience gained at Mt. Taylor. In this respect, the following recommendations can be made:

1. Convergence points should be sited to avoid damage by mine equipment and to allow comparison of convergence measurements at all stations.
2. Vibrating wire stressmeter cables should be protected from shearing in boreholes by metal or hard plastic conduit, and instruments should be protected from curious miners by informing all crews of the purpose of these devices.
3. Convergence monitoring can provide an effective warning system for ground control problems, while stress and load monitoring can potentially provide sensitive means of determining parameters for future design purposes.
4. If possible, instrumentation should be installed well in advance of any changes in mine configuration, in order to allow measurement of the full effects of changing the configuration of the mine.

The results of rock mechanics instrumentation and monitoring at Mt. Taylor show that these techniques can potentially provide valuable guidelines in mine design and operation. Similar guidelines can be applied to deep room and pillar, shortwall, and longwall mining, especially in faulted ground. An understanding of ground behavior during extraction will eventually lead to improved ore recovery.

ACKNOWLEDGEMENTS

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Chapter 55

REINFORCEMENT OF LARGE PILLARS BY BOLTING

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ABSTRACT

An analysis of bolting reinforcement of several large [approximately 18 m (60 ft) cube] pillars was performed. The many overcoring stress profiles in pillars at the mine were used to produce generalized stress distributions at various stages in pre- and post-failure. A complete stress-deformation curve was estimated from this data. The effect of pillar bolting on the load-deformation behavior was extrapolated from the observed unbolted behavior. Bolting can increase pillar strength by up to 10 percent. The effect of bolting on general mine stability was examined with a short computer analysis. The computer model was calibrated against measurements performed at the mine, and produced good agreement between measured and modeled stresses. The analysis indicated that bolting increases the bolted pillar safety factors in the event of additional pillar failure by a small amount (approximately 5 percent). Progressive pillar failure is unlikely, because arching transfers most load from yielding pillars to large unmined abutments rather than to the smaller panel pillars.

INTRODUCTION

Pillar reinforcement by bolting was performed recently in the experimental mine at Exxon's Colony oil shale property in northwestern Colorado (Figure 1). Mining was performed at depths of 183 to 259 m (600 to 850 ft) in the Mahogany Zone, a kerogen rich horizon located in the Parachute Creek Member of the Green

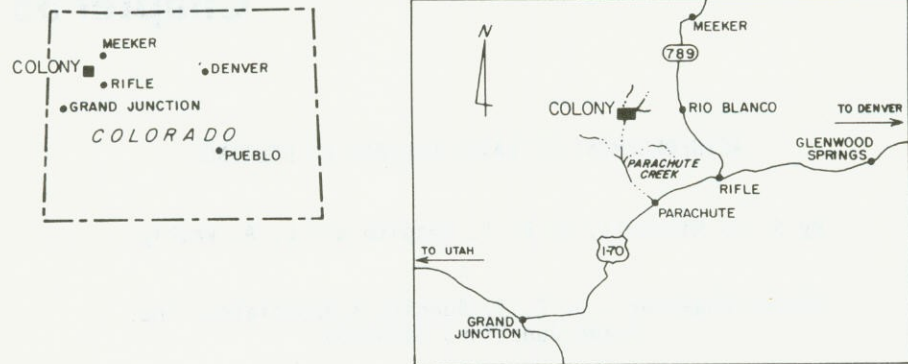


Figure 1 Location of the Colony Property

River Formation. The Mahogany Zone is a kerogen rich calcium magnesium marlstone with a uniaxial compressive strength of 90 MPa (13,000 psi) and generally good rock quality. Four pillars and a portion of a chain pillar located in the north corner of the mine (Figure 2) were reinforced with 36 bolts each entirely through the pillars. The bolts were tensioned to approximately half of their

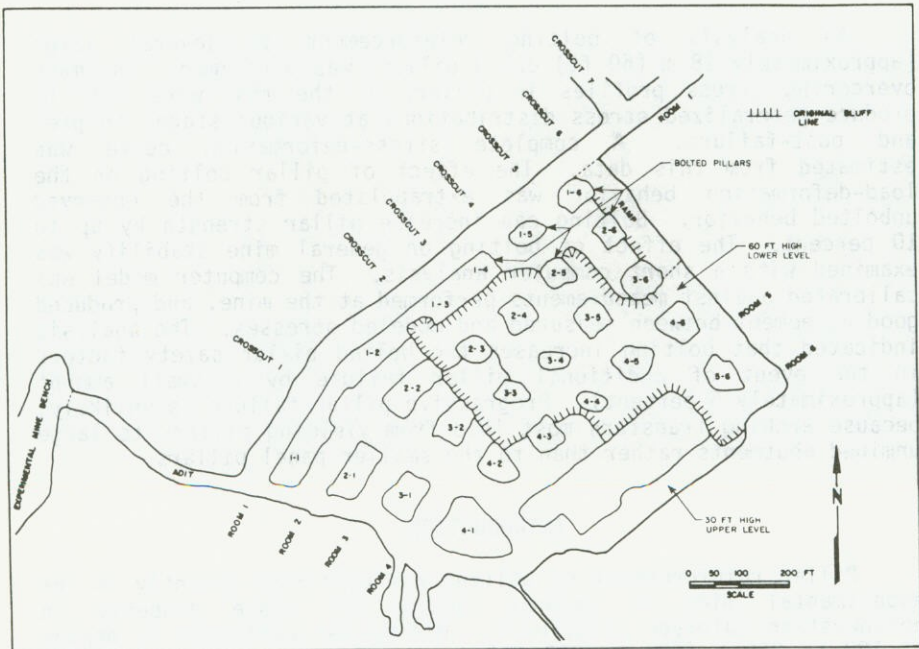


Figure 2 Plan View of the Colony Experimental Mine

yield strength of 834,000 n (187,500 lbs) and grouted. The purpose of this work was to provide additional load carrying capacity in these pillars in case of further pillar failure in the central part of the mine. Pillars 1-5 and 1-6 are in a residual or post-failure condition, while 2-5 and 2-6 have not yet reached peak stress.

In this paper, pillar failure is defined as pillar loading beyond peak strength. Pillar failure does not imply complete collapse; in oil shale, roof stiffness is sufficient to transfer much of the load from a yielded or failed pillar to nearby unfailed pillars and abutments, leaving the failed pillar standing in place carrying a significant residual stress.

PILLAR STRESS DISTRIBUTION

Thirty separate stress profiles have been generated by overcoring measurements in pillars of the Colony Pilot Mine (Agapito, 1972 and 1974; Agapito and Page, 1976). Some pillars were overcored at different times at various stages in pre- and post-failure. Half of the profiles are from pillars that were in an intact or pre-failure condition, and half from pillars in a residual or post-failure state. This data has been used to calculate average stress distributions in pillars at various loading stages.

The pillar stress profiles show consistent patterns for different stages of loading and failure. The measured profiles generally show very low stress for some distance near the pillar edge, and increase to a relatively constant stress in the pillar core area. The destressed pillar skin is due to blast damage and yielding related to low confinement. This region grows in size during the post-failure stage due to slabbing and more extensive failure of the edges. The constant pillar core stress is observed in most of the profiles as shown in Figures 3 and 4, and is

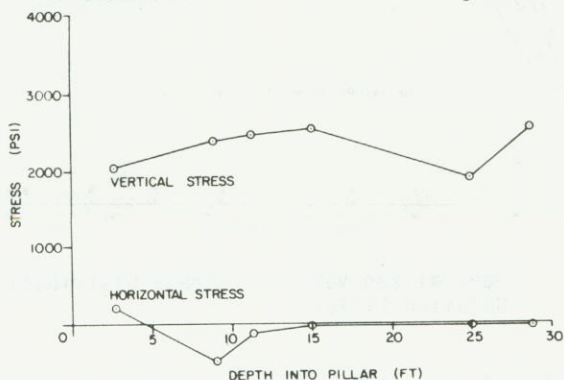


Figure 3 Stress Profile - Pillar 4-6, March, 1976

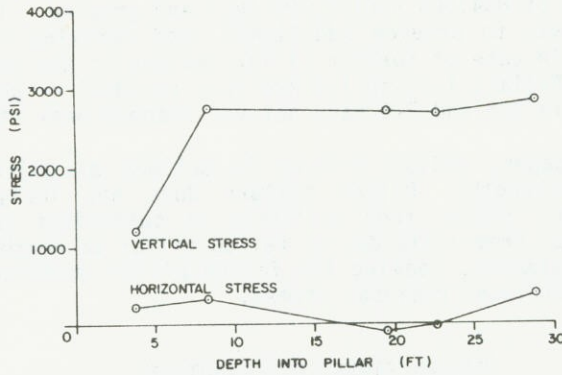


Figure 4 Stress Profile - Pillar 3-5, October, 1980

associated with the relatively constant horizontal stress in the core area.

Generalized average vertical stress profiles at various stages in pre- and post-failure are shown in Figures 5 and 6. These

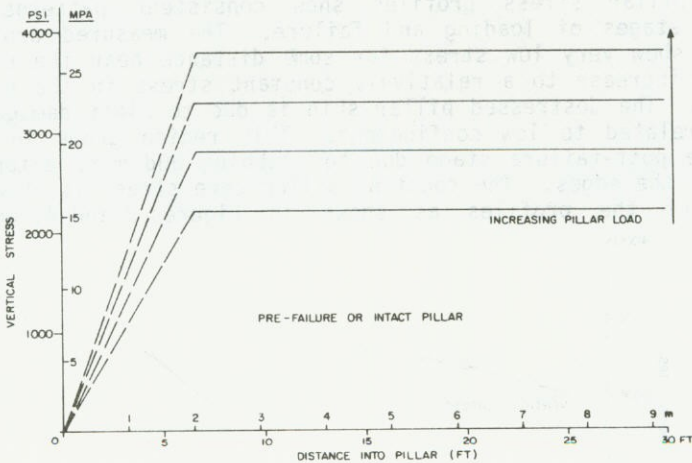


Figure 5 Generalized Vertical Stress Distributions, Unfailed Pillar

figures are based on the average measured pillar stress distributions. In the unfailed state, increasing pillar load is

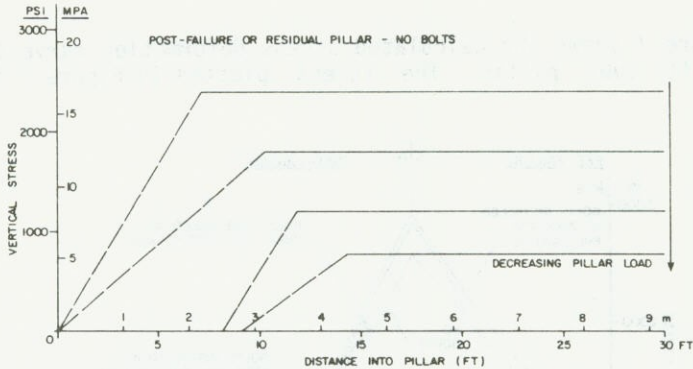


Figure 6 Generalized Vertical Stress Distributions, Failed Pillar Without Bolts

taken by an increase in the core stress level. The load carrying area of the pillar remains approximately constant up to failure. In a post-failure state, the pillar load carrying capacity decreases by both a reduction in core strength and an increase in the area of the destressed pillar edges. The latter effect is often observed in the field as sloughing or "hourglassing" of highly stressed pillars.

The pillar edges are assumed to carry a small load except in advanced stages of post-failure, when sloughing causes the edges to fall off (see Figure 6). The rock along the pillar skin is generally too broken up to obtain overcoring measurements, and so probably carries only small stresses.

PILLAR LOAD-DEFORMATION BEHAVIOR

The load-deformation behavior of unbolted pillars was examined to serve as a basis for evaluating the effect of bolt reinforcement. These calculations utilized pillar stress distributions shown in Figures 5 and 6, as generalized from overcoring measurements, and elastic moduli values obtained from measurements at the mine.

Both pre- and post-failure in-situ modulus values have been obtained by Agapito (1972) at the Colony Pilot Mine. Measurements in three pillars indicate an average pre-failure modulus of 4.5 GPa (0.65×10^6 psi). Post-peak measurements of two pillars have produced a slope of the stress-strain curve of -6.9 GPa (-1.0×10^6 psi).

Figure 7 shows the calculated stress deformation curve for an 18 m (60 ft) cube pillar. The stress plotted in Figure 7 is the

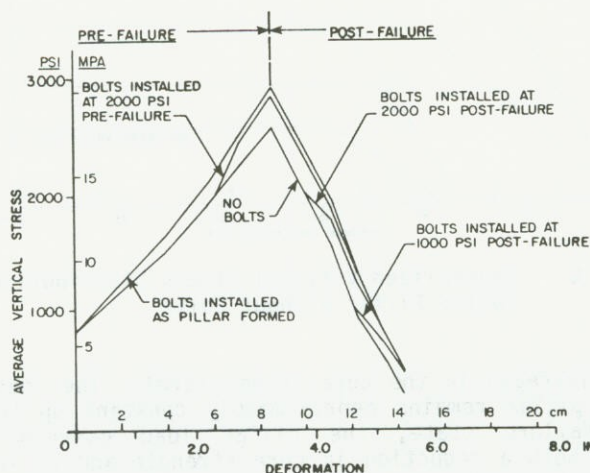


Figure 7 Bolted Pillar Stress-Deformation Behavior

load calculated from the stress distributions of Figures 5 and 6 divided by the entire original 18 x 18 m (60 x 60 ft) area. Pillar deformation is related to the core stress by the assumed modulus.

EFFECT OF PILLAR BOLTING ON LOAD-DEFORMATION BEHAVIOR

The effect of pillar bolting on the pillar load-deformation behavior was extrapolated from the observed unbolted behavior, and is also shown in Figure 7. Bolting increases pillar strength by:

- o providing confinement, and so increasing strength as observed in triaxial laboratory tests; and
- o preventing or limiting spalling of the pillar edges, retaining the pillar's cross-sectional area.

The generalized stress distributions and pillar load-deformation behavior were estimated based on these two effects. The first mechanism affects both the pre- and post-failure pillar strengths. The second effect adds strength mainly to pillars in a post-failure state, as pillars in a pre-failure mode should not spall.

The effect of pillar bolting is dependent on when the bolts are installed. The bolts have their maximum reinforcing effect

when stressed to their maximum load; this point is reached only after additional pillar deformation has increased the bolt tension beyond its installed value. For this reason, bolts provide the most reinforcement when installed early in the pillar load-deformation history. At maximum bolt tension, bolted pillars will carry 1.4 to 2.1 MPa (200 to 300 psi) more stress than unbolted pillars at the same deformation. This is an increase in peak strength of approximately 10 percent.

EFFECT OF PILLAR BOLTING ON MINE STABILITY

The effect on general mine stability of bolting four pillars in the Colony Pilot Mine was examined with a computer analysis. This analysis utilized EXPAREA, a boundary element program that calculates stresses and displacements perpendicular to a tabular orebody (St. John, 1978). One and two additional pillar failures in the central part of the mine were simulated, and the effect on nearby pillars, both with and without bolting, was calculated.

Verification of Model

A calibration run was made with the computer model to verify that it can adequately represent the mine. Pillars that are known to be failed were constrained to carry their measured loads, and the many overcoring measurements in unfailed pillars were compared to the computed results.

Table 1 indicates good agreement in the average vertical stress as measured by overcoring and computed by modeling.

Table 1 Comparison of Measured and Modeled Stresses

Pillar	<u>Measured Stress</u>		<u>Modeled Stress</u>		Difference
	MPa	Psi	Mpa	Psi	
2-6	18.0	2605	16.9	2447	-6%
2-5	19.9	2892	16.8	2437	-16%
3-5	16.5	2386	16.9	2450	+3%
4-5	15.9	2306	17.0	2471	+7%

Figure 8 also shows good agreement between a measured and modeled stress profile. These comparisons indicate that the computer is

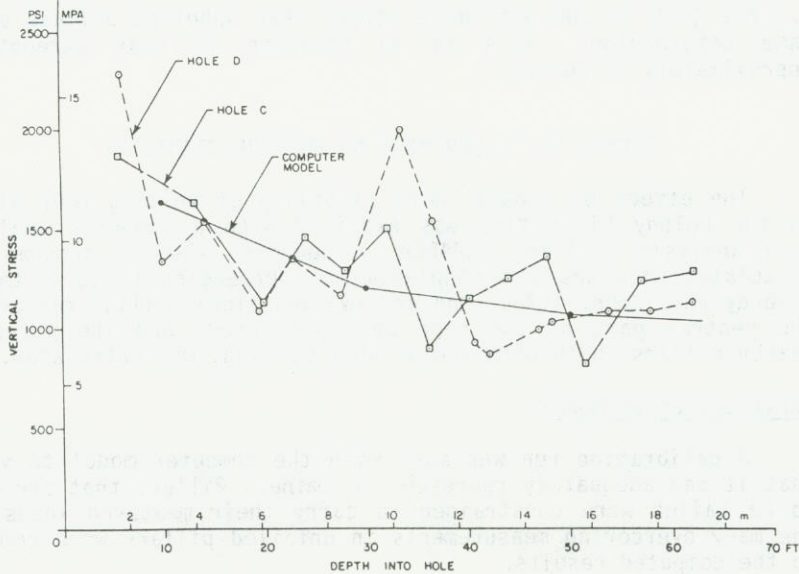


Figure 8 Comparison of Measured and Modeled Stresses, Rib Wall Southeast of Pillar 5-2

accurately modeling the average stress distribution around the pilot mine, particularly the stress arching occurring from the central area of the mine to the abutments.

Effect of Bolting on Mine Stability

The computer model was used to simulate one and two additional pillar failures in the central part of the mine, both with and without the current program of pillar bolting. The pillars assumed to yield are presently unfailed, and are close to the center of the mine, where their failure would have the greatest effect on other pillars because of the lack of nearby unmined abutments to which they can shed load.

The effect of pillar bolting was modeled by changing the modulus and peak strength in accordance with the bolted pillar behavior described above. A peak unbolted strength of 20.7 MPa (3000 psi) was assumed (Agapito, 1972).

Table 2 contains a comparison of the average stress and safety factor of the unfailed pillars at the present time and after the

Table 2 Stresses and Safety Factors of Unfailed Pillars

Pillar	Present Case			After Failure of Pillar 3-5 to 10.3 MPa (1500 psi)			
	Measured Stress (MPa)	Modeled Stress (MPa)	Safety Factor*	No Bolts		Bolts	
				Stress (Mpa)	Safety Factor	Stress (MPa)	Safety Factor
2-6	18.0	16.9	1.23	18.6	1.11	18.6	1.16
2-5	19.9	16.8	1.23	19.3	1.07	19.6	1.10
3-6	-	16.4	1.26	19.0	1.09	19.0	1.09
3-5	16.4	16.9	1.22	10.3	-	10.3	-
4-5	15.9	17.0	1.21	19.8	1.05	19.8	1.05

* Based on the modeled stress and a strength of 20.7 MPa (3000 psi).

assumed failure of Pillar 3-5, both with and without bolts. The safety factor is the ratio of strength to stress.

The assumed failure does not cause failure of any of the other currently unfailed pillars in the area, although stresses are increased and safety factors reduced significantly. The many previously failed pillars in the mine undergo significantly more fracturing, deformation, and unloading as a result of the additional pillar failure. If any of the unfailed pillars have a strength significantly less than the 20.7 MPa (3000 psi) as observed by Agapito (1972), additional failures could result. Alternatively, overcoring measurements show that Pillars 2-5 and 2-6 are carrying more load than indicated by the computer model (see Table 1). The 10 - 15 percent stress increase caused by the failure of 3-5 would probably be sufficient to push these pillars over their peak. Bolting of Pillars 2-5 and 2-6 increases their safety factors slightly over the unbolted case following the failure.

Subsequent analysis of an additional assumed failure, that of Pillar 4-5, indicates that the additional failure of Pillar 4-5 has almost no effect on the stresses and safety factors of the intact

bolted pillars. This is because Pillar 4-5 is far from the bolted pillars (see Figure 2); arching transfers most of the load from the failed pillars to the large unmined abutments to the east and the large pillar to the south, rather than to the bolted pillars.

Analysis indicates that pillar bolting has only a small effect on the safety factors of the bolted pillars following one or two additional pillar failures. The bolted pillars are less likely to fail when bolted than when not bolted, as shown by the relative safety factors after the failure of Pillar 3-5. Progressive pillar failure of all remaining unfailed pillars in the mine following a single additional failure appears unlikely; the competent and stiff roof strata causes much of the load from yielding pillars to be transferred by arching to large unmined abutments rather than to the smaller panel pillars.

CONCLUSIONS

Examination of the many pillar stress profiles indicates consistent patterns for different stages of loading and failure. A complete pillar stress-deformation curve was estimated from this data, and extrapolated to the case of pillars reinforced by bolting. Bolting can increase peak pillar strength by up to 10 percent, depending on the timing of bolt installation. Computer analysis indicates that pillar bolting has a small effect on the safety factors of the bolted pillars following one or two additional pillar failures. Progressive pillar failure and collapse of the north portion of the mine are less likely to occur because of the pillar bolting program. Most of the load from failed pillars is transferred to large unmined abutments by the stiff and competent roof strata.

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Chapter 56

PRESSURE MEASUREMENTS IN THE GOB

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ABSTRACT

Gob pressure measurements were made in a Western U. S. coal mine as part of a long-term program to evaluate cave progress and to determine the influence of geological discontinuities on caving conditions, load transfer, and resulting instabilities.

Gloetzl cells were selected for the measurements due to their simple, robust construction and a history of being able to monitor pressures in a broken medium. A methodology was developed for the successful installation and protection of the cells and hydraulic lines in the gob as the face retreated.

The measurements indicated a cave progress controlled by the frequency of major faults. The pressure-mining progress profile was compared to those observed in other parts of the world. It was concluded that the significant differences in profiles was caused by the thick-bedded strata, the existence of high lateral stresses, and the spacing between faults.

Recommendations are given for future applications of Gloetzl cells for gob pressure measurements.

INTRODUCTION

Background

When the overburden is undercut through removal of the coal seam, a redistribution of stresses takes place, transferring the weight of the undermined rocks to the panel boundaries. As the undercut dimensions increase, bed separation and roof falls

develop. The falls (or the cave) progress up into the roof until the mined space is filled with broken rock. At this time, compaction of the caved rocks occurs, permitting a partial transfer of overburden load to the floor.

The amount of load transferred through the gob and the distance beyond the face where full recompaction of caved rocks occurs are of great importance for determination of the stress redistribution to panel boundaries. The load transfer through the gob is a function of large scale geology (i.e., existence of major faults, lateral stress fields, and roof rock formations), as well as small scale structures (i.e., fractures) which affect the size distribution and stiffness of the caved rocks. Thus, the nature and degree of load transfer could vary from site to site.

Evaluation of cave progress by in-panel measurements (in the gob, under the panel, or over the gob) is believed to be essential for evaluation of caving conditions and load transfer mechanisms. Such measurements, however, have been very limited due to the difficulties and costs of obtaining in-panel measurements on a large scale. Alternatively, researchers have preferred to perform measurements at panel boundaries, and then speculate as to what is happening in the gob. Difficulties with the latter approach are that the measurements should be obtained on a very large scale since the caving methods influence the strata much beyond the panel boundaries. Problems exist with the reliability of most instruments and the meaning of measurements because of the nonlinear behavior of coal at the face.

Review of In-Panel Measurements

In-panel measurements have been performed for evaluation of cave progress. Generally, they can be grouped into measurements obtained above the gob in the roof, in the gob, and under the panel. A brief description of these measurements follows.

Measurements above the gob generally have been made in boreholes drilled from the surface or other openings located above the panel. These measurements determine the bed separation in the roof, which are not directly related to the load transfer through the gob. The techniques include installation of multiple position wire extensometers, geophysical techniques such as Time Domain Reflectometry (TDR), and the installation of radioactive markers in boreholes. The application of wire extensometers located in boreholes drilled from an upper level is discussed by Farmer (1980). The extensometer data is transferred into apparent vertical strain contours, thereby revealing the distribution of tensile and compression zones in the roof.

The TDR application involves grouting a 2.2 cm (7/8 in.) diameter coaxial cable in a borehole and monitoring the cable

condition (continuity or faults) by a tester (Wade and Conroy, 1980). Another method involves marking locations on the boreholes with a radioactive material and surveying the hole to detect the relative position of the markers as caving progresses.

The measurements in the gob are the most useful for the determination of load transfer through the gob. Few have been made due to difficulties of installing and protecting the load cells against the impact of caved rocks. Most measurements have involved the use of hydraulic flat jacks in packs located adjacent to longwall access road entries. The differences in the stiffness of the cells, fill materials of the packs, and the naturally caved material introduces difficulties in interpretation. In addition, pressure monitoring in the gob at the panel centerline is required for evaluation of cave progress and load transfer.

Oyanguren (1972) installed "goaf load cells" in the gob for determination of stress profile versus mining face position. Although information regarding the nature of these cells is missing, the results appear reasonable. The use of Gloetzl cells for direct measurement of the pressure in the gob (Maleki, 1981) is the topic of this paper.

Measurements under the panel have involved the installation of vibrating wire stressmeters (Wade, 1980) or rigid inclusion stressmeters (Oyanguren, 1972) in holes drilled from gateroad entries. There are questions regarding the meaning of the measurements if nonlinear behavior occurs at cell-hole interface.

Description of the Test Site

A large scale instrumentation program was initiated in a Western U.S. coal mine in order to evaluate the relationship between geological discontinuities, cave progress and roof instabilities. As a part of the program, gob pressure measurements were undertaken. These results will be discussed in this paper.

The coal mine utilizes a semi-caving panel mining method in which both development and retreat is performed with continuous miners. Shuttle cars are used for face transportation and belt-haulage for secondary transportation.

The development starts by driving three entries about 550 m (1800 ft) along the panel length, and the outlining of a barrier pillar 152 m (500 ft) long. In retreat, two rows of pillars are created along the panel width and then the first row, including one of the chain pillars, is retreated back to the panel entries. This is shown in Figure 1. This sequence continues until the panel is extracted. A barrier pillar is left between the cave and the submains. The second chain pillar will be mined during the retreat of the next panel, and thus no pillar is left between the panels.

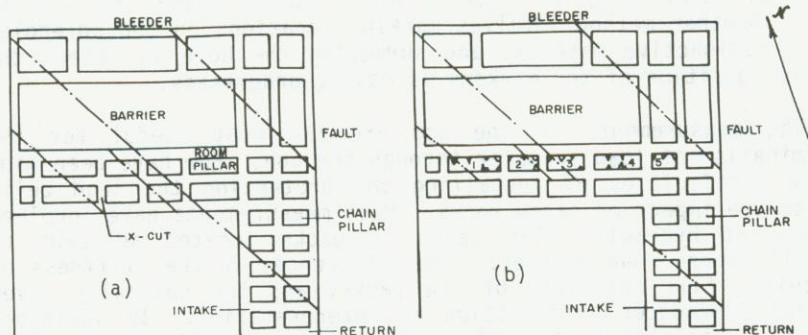


Figure 1 The Mining System

a) Retreat

b) Pillar Robbing Sequence

The topography over the instrumented panel is a plateau with an overburden depth ranging from 300 to 365 m (1000-1200 ft). The coal seam, which averages 4.5 m (15 ft) thick is overlain by a series of thick-bedded sandstones, siltstones and shales.

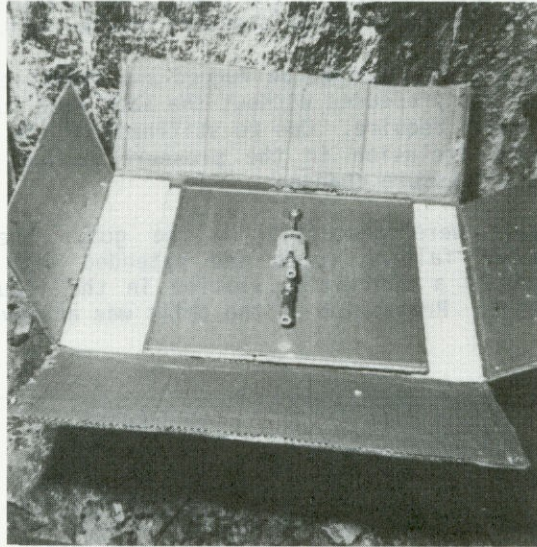
A series of parallel faults, having apparent vertical displacement of 0.15 to 4.5 m (0.5 to 15 ft) intersect the mining panels at 45 degrees. The faults dip 85 degrees to the southwest. Two sets of joints exist in the roof rocks. These are parallel and perpendicular to the direction of the faults.

The in-situ horizontal stresses are oriented approximately parallel and perpendicular to the faults with a magnitude of 6.9 and 4.1 MPa (1000 and 600 psi), respectively.

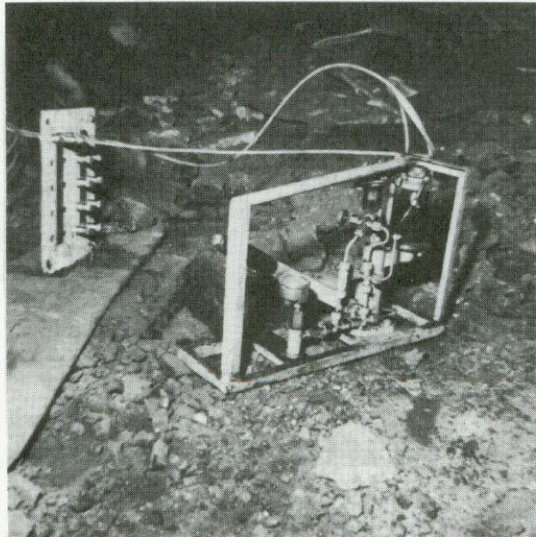
GOB PRESSURE MEASUREMENTS

Gloetzl cells were used for the measurement of the pressure exerted by caved rocks on the floor. The cells consist of a thin mercury filled pad 0.4 x 0.4 m (16 x 16 in.) in area, a bypass valve and two intake and return hydraulic lines filled with a combination of diesel fuel and oil for low compressability and viscosity. A cell and the hand pump used for pressurizing the lines are shown in Pictures 1 and 2.

To determine the external pressure on the mercury pad, the pressure in the intake line is increased until the bypass valve opens, allowing the fluid to flow in the return line. This occurs when the external pressure and the intake line pressures are equal. At this time, a sudden drop in the intake pressure line is



Picture 1 A Gloetzl Cell Used for Gob Measurements



Picture 2 The Gloetzl Cell Pump

observed on the gage mounted on the pump. The stable reading after the initial drop is the external pressure on the cell.

The cells are designed to be rugged and reliable. They yield measurements of gob pressures without the same need for calibration as some other cells require. Due to the stiffness of the pad, the cell acts as a rigid inclusion in the pressure medium, and thus is sensitive to any pressure changes.

Eight cells were installed in the gob. The lines were protected against falling rocks and extended with the retreat operations back to a monitoring station in the return entry, as shown in Figure 2. Protection of the cells was achieved by placing

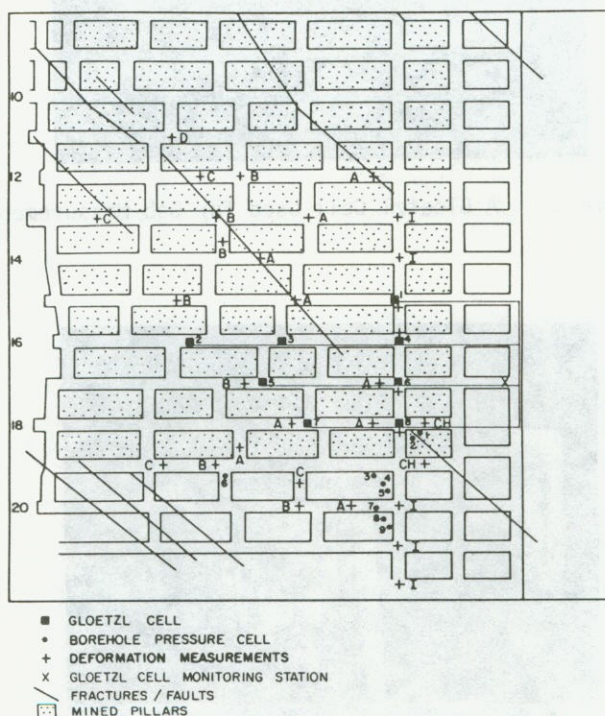


Figure 2 Map Showing the Location of the Instruments

approximately two feet of fine gravel and a piece of old conveyor belt 2.5 x 2.5 m (8 x 8 ft) in size over them, as illustrated in Figure 3. The pressure lines were protected by passing them

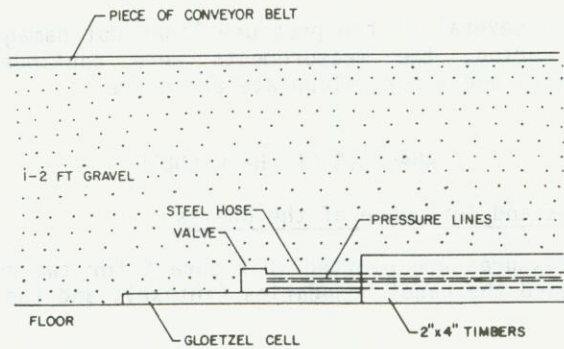


Figure 3 Protection of the Cells and the Lines Against Caving Rocks

through two slots 1.3×1.3 cm (0.5×0.5 in.) in size made in 5×10 cm (2×4 in.) timbers. The lines were extended every 3 m (10 ft) during the retreat. The lines were passed through the slot of a timber and temporarily kept within the slot by taping. Another timber was put over the bottom timber for completely containing the lines. The timbers were finally nailed together. For additional protection, end pieces of posts were put adjacent to the timbers and some dirt was shoveled on the top, as illustrated in Picture 3.



Picture 3 Protection of Pressure Lines by 5×10 cm (2×4 in.) Timbers, End Pieces of Posts and Dirt

Although several of the pressure lines got damaged during the monitoring period, the measurements were continued using the undamaged lines and a more elaborate procedure.

ANALYSIS OF THE RESULTS

Cave Progress and Influence of the Faults

Cell pressures are plotted in Figure 4 for two cells. One is located toward the panel boundaries (intake), and the other on the

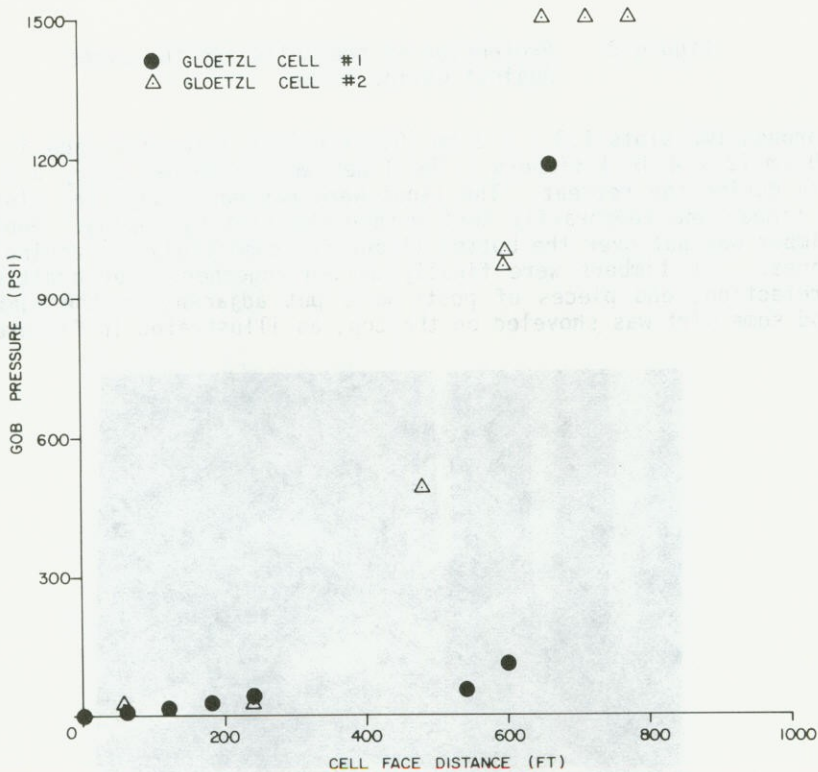


Figure 4 Cell Pressure Versus Face to Cell Distance

centerline to demonstrate the difference in the pressure pattern between these locations. In addition, successive pressure measurements for all of the cells versus the position of the retreat in the panel and adjacent panel are illustrated in Figures 5, 6 and 7.

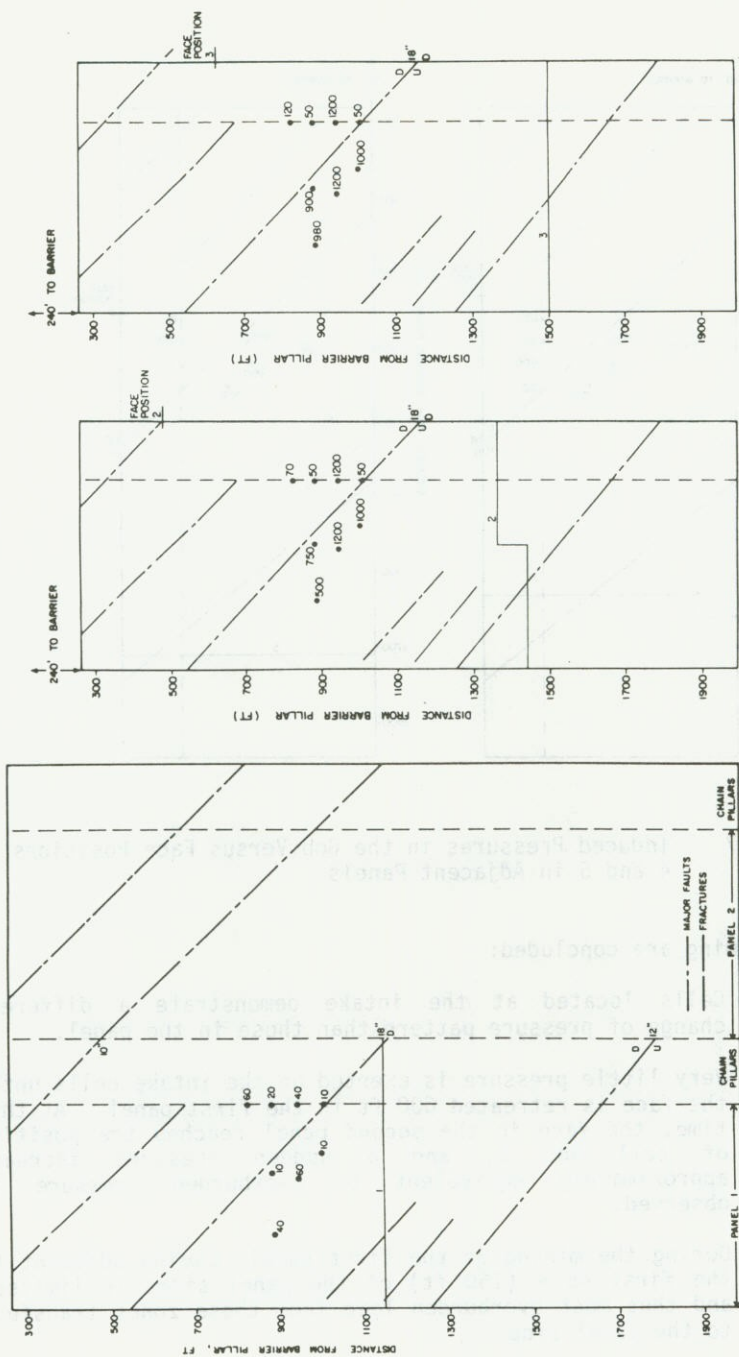


Figure 5 Pressure Measurement at Face Position 1

Figure 6 Induced Pressures in the Gob Versus Face Positions 2 and 3 in Adjacent Panels

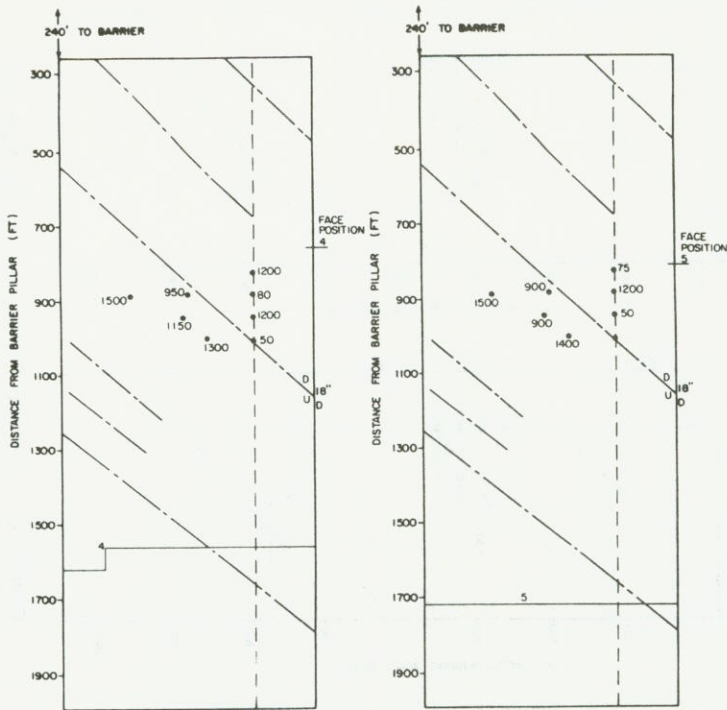


Figure 7 Induced Pressures in the Gob Versus Face Positions 4 and 5 in Adjacent Panels

The following are concluded:

- o Cells located at the intake demonstrate a different change of pressure pattern than those in the panel.
- o Very little pressure is exerted on the intake cells until the face is retreated 600 ft in the first panel. At this time, the face in the second panel reaches the position of cell No. 1, and a sudden pressure increase approximately equivalent to overburden pressure is observed.
- o During the mining of the first panel, caving adjacent to the first 45 m (150 ft) of the panel sides is limited, and thus most overburden load from these zones transfers to the panel sides.

- o Measurements from the cells located in the panel reveal a very limited cave height, 15 m (50 ft), developed even after the face has retreated 91 m (300 ft) from the cells.
- o The maximum pressure increase occurs suddenly at face positions 2 and 3, approximately 182 m (600 ft) away from the cells. At this time, a second major fault with a displacement of 0.3 m (1 ft) is undermined.
- o The complete breakage of roof rocks at the mine is dependent on the frequency of major faults. The greater the distance between the major faults, the stronger are the beams outlined by the faults, both due to strength and the existence of lateral stresses parallel to the direction of faults.
- o Fractures with less than .25 m (10 in.) displacement do not affect the breakage of main roof rocks.
- o Prior to complete breakage of roof rocks, load transfer is favored parallel to the direction of the faults, i.e., to the panel sides.
- o The optimum distance between the face position in adjacent panels was determined to be approximately 244 m (800 ft) for current panel orientations. At this spacing, the interaction between adjacent caves is minimal.

The equivalent rock column weight along a section parallel to the direction of the faults for different face positions is illustrated in Figure 8. Most measurements at face positions 4 and 5 demonstrate the existence of pressures higher than the overburden. Whether this is true or not cannot be confirmed, due to large leaks in the pressure lines at the time of these measurements.

Comparison With Other Measurements

Direct pressure measurements in the gob have been few. A large number of observations, however, have been made in Britain in order to determine the distance in gob where full overburden pressure is resumed. The British results, pressure measurements made in a South African potash mine (Oyanguren, 1972), and the measurements made in the Western U. S. mine are compared in Figure 9 to demonstrate the variations in the distance and load profiles. The British curve is estimated behavior for a section perpendicular to the face (Wilson, 1982).

From this comparison, it is concluded that the strata caving, and thus, load transfer to panel boundaries for the thick bedded formation of the U. S. mine, is significantly different from the

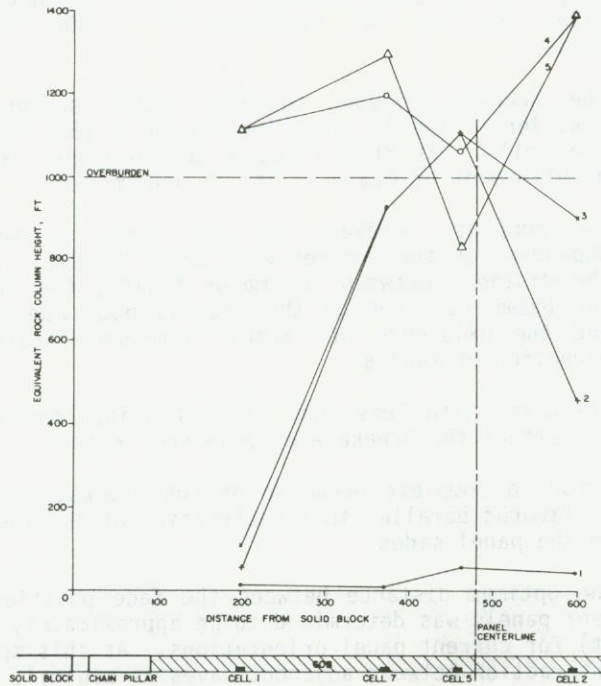


Figure 8 Equivalent Rock Column Height for a Section Parallel to a Major Fault

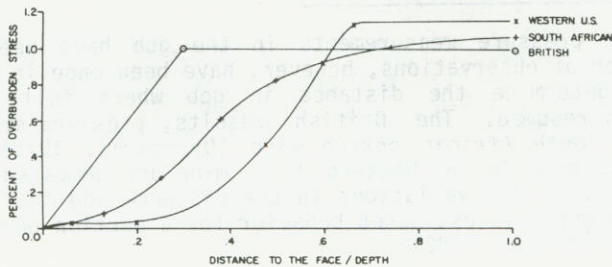


Figure 9 Comparison of Load Profiles for Different Strata Conditions

British mines, but is closer to the behavior of the sandstone/marlstone strata of the South African mine. For the U. S. mine, the existence of high lateral stresses and the large spacing between the major faults are other factors which could have produced the difference.

RECOMMENDATIONS FOR FURTHER WORK

The importance of gob pressure measurements in the evaluation of cave progress for site specific conditions was demonstrated. However, these measurements are, by nature, costly, and require installation and monitoring adjacent to gob lines. Careful attention to good safety practices is mandatory.

Gloetzel cells generally performed satisfactorily, but the protection of the lines was critical, and installation was very labor intensive. For other applications, the use of cells of special design in which either the number of lines is minimized or preferably eliminated is recommended. This will add to the capital cost of the measurements, but will result in better accuracy and lower manpower requirements.

ACKNOWLEDGEMENT

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Chapter 57

DETERMINATION OF SOLUTION CAVERN SPACING IN DEEP SALT DEPOSIT

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ABSTRACT

An engineering procedure for designing the minimum separation between solution mined caverns in bedded salt deposit is described. The procedure is based on data obtained by in-situ and laboratory test methods. The formulae utilize the concept that salt rock under high pressure in a deep salt rock deposit would behave as a perfectly plastic substance, and that the active opening surrounding a typical cavern could be described by a circular ellipsoid. The paper describes the calculations required for the safe design of a solution mined salt cavern field.

INTRODUCTION

The salt cavern LPG storage facility at Fort Saskatchewan, Alberta, Canada, is located at the southwestern edge of the upper Lotsberg Salt evaporites. Rock salt of the upper Lotsberg in this area consists primarily of clear, very coarse, crystalline halite. The salt bed is approximately 55 m thick and is found at a depth of about 1860 m from surface. It is overlain by a 15 m thick formation consisting of a lower shale, a middle limestone and an upper anhydrite. Approximately, the next 40 m consists of salt, followed by 80 m of dolomite and shale overlain by 180 m of salt interbedded with shale. The interval of 30 m below the mining horizon contains thin bands of shale interbedded with salt.

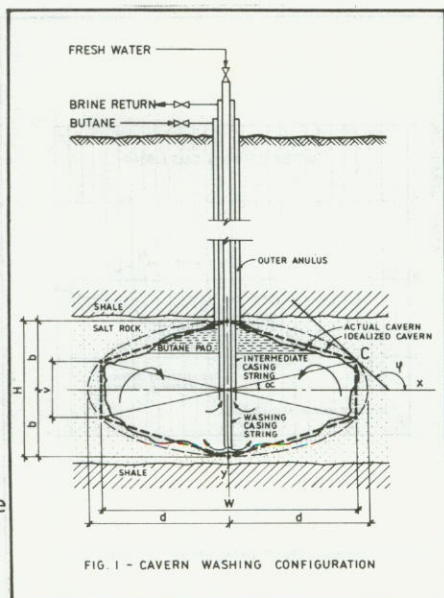
The method for solution mining the ten caverns presently in use has varied since the facility was established in 1971. The first methods involved washing short vertical sections of the salt bed in stages until completion. The present method positions the washing and intermediate casing strings as shown in Fig. 1, and injects

butane through the outer annulus to protect the roof from being washed. Earliest methods produced unstable cavern shapes with uneven walls and flat roofs. Using these methods limited typical diameters to 60 m and volumes to 70,000 m³. But as experience was gained, caverns with regular and predictable shapes, as shown in Figure 1, were attained that reached 100 m diameters and volumes of over 170,000 m³.

In storage service for the propane, or butane, the caverns are kept liquid full at all times using a brine displacement system. Although product must be pumped into the storage caverns, the difference in hydrostatic head between the brine and the propane, butane, is sufficient to deliver the product to the pipeline at high rates without pumping. The brine, however, is only about 90% saturated and when used to displace product will wash a cavern slowly larger. Radii increases of up to 0.6 m per year were observed at cavern bottoms, while increases of only 0.2 m per year were typical higher in the cavern.

DESIGN CRITERIA

The forecasting of safe dimensions for cavern development posed a rock mechanics design problem. The design criteria required that maximum sized caverns be spaced at their minimal safe distances to reach a fifty year life span without failure. Initial study determined that, minimal spacing could be accomplished with a triangular pitched drilling pattern and that the cavern life span would depend on the rate of volume increase due to displacing with unsaturated brine. Based on sonar measurements, the average diameter increase was considered at 0.5 m per year. The engineering theory, developed by Mraz, utilizing the concept that the salt rock under high pressure in deep salt rock deposit would behave as a perfectly plastic substance, was adopted. In order to determine rheological properties of the salt rock, the laboratory method proposed and developed by Mraz and Dusseault (ref. 2 and 6) was used. The testing was done by Dusseault at the University of Waterloo, Ontario, Canada.



LABORATORY TESTING

Stage-loading triaxial creep tests were carried out to determine creep rates under different constant stress conditions. Specimens 54 mm in diameter and 108 mm long were dry diamond cored from the 101 mm diameter drill cores provided from a drill hole at Fort Saskatchewan in Alberta. The ends were flat-ground (dry) to exacting tolerances, coated with a low friction dry lubricant, and mounted in a high pressure triaxial cell having a ram of the same diameter as the specimen. A confining stress was applied using hydraulic oil acting against a sealed flexible Neoprene membrane and the specimen remained under this confining stress for at least 48 hours in an attempt to overcome stress-relief creep and crack creation during sampling. The deviatoric stress was then applied axially, held constant, and the creep rate allowed to equilibrate. Steady-state creep was usually reached within 72 hours and the test was continued until three successive days of constant creep rate were observed. The axial and lateral load were then adjusted to the next set of stresses, and the process repeated. Testing was carried out under constant temperature ($\pm 0.1^\circ\text{C}$) to eliminate environmental effects. The use of hydraulic oil throughout the system eliminates any effects arising through specimen moisture changes by diffusion through membranes and seals. Careful attention to end conditions results in relatively uniform cylindrical strain with a minimum of "barrelling". The techniques used are repeatable and reliable, and the stage-loading can be carried out up to 7-8% of axial strain. To ensure quality data, accurate

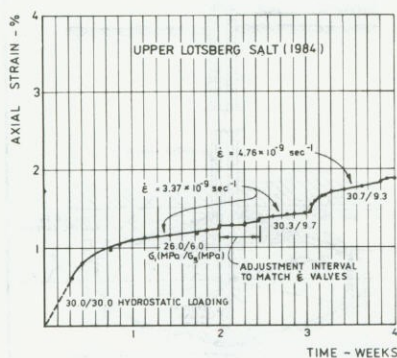
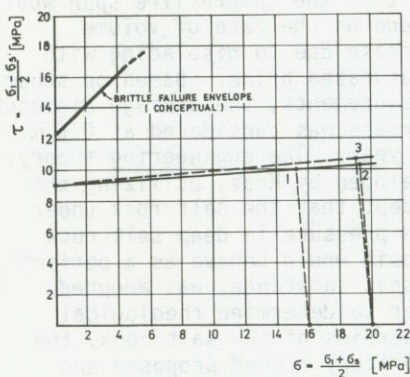


FIG. 2 - TYPICAL CREEP DATA, UPPER LOTSBERG SALT



TEST NO.	σ_1 [MPa]	σ_3 [MPa]	σ [MPa]	τ [MPa]	$\dot{\epsilon}$ [sec^{-1}]
1	26.0	6.0	16.0	10.0	3.37×10^{-9}
2	30.3	9.7	20.0	10.3	3.48×10^{-9}
3	30.7	9.3	20.0	10.7	4.76×10^{-9}

FIG. 3 - PLOT OF CREEP DATA IN STRESS SPACE

reliable stress control is necessary. Intensified accumulators using air/oil, rolling membranes, and Teflon-impregnated seals of extremely low friction have been designed to maintain stresses to within 50 kPa at stress levels greater than 5000 kPa. The pressurizing system is analogue controlled with an option for microcomputer control, and uses two pairs of highly sensitive solenoid control valves to give upwards and downwards stress control within an adjustable, very narrow range (less than 50 kPa). It has also been found that the entire loading system and sensing transducer systems must be protected from temperature changes as these can affect the constancy of stress conditions. Figure 2 is a linear plot of time versus strain for two after some time. A series of these test gives information as to the creep rates of the material at various stress states, and contours of equal creep rate can be drawn (Figure 3). In general, for the cavern problem, we are more interested in the lower stress limit for the dislocation creep phenomenon which leads to closure. Of less interest to the long-term stability analysis is the strain-weakening envelope (failure envelope), because once the caverns are excavated and reach equilibrium closure conditions, strain-weakened zones are stable and generalized plastic flow can be assumed to be constant if the opening geometry, as approximated by the equivalent opening concept, remains sensibly constant. This is a reasonable assumption, confirmed by field observations (Ref. 1).

GEOMETRY OF ACTIVE CAVITY AROUND SOLUTION CAVERN

In a deep salt rock continuum, a cavity will act in a way that conforms best to geohydrostatic conditions. As shown in Ref. (1) and (2), the active opening around a rectangular section is elliptical. A cross-section of a typical solution cavern can be simplified into an irregular hexagon, symmetrical about vertical and horizontal axes. Such a cavity will act as a circular ellipsoid with the following geometry (Figure 1):

$$H = 2b \quad (1)$$

$$b = \frac{H}{2}$$

Equation of Ellipse:

$$\frac{x^2}{d^2} + \frac{y^2}{b^2} = 1 \quad (2)$$

At Point "C":

$$x_c = \frac{w}{2}$$

$$y_c = \frac{v}{2}$$

From Equations (1) and (2):

$$d = \frac{HW}{2\sqrt{H^2 - v^2}} \quad (3)$$

Vertical Radius of Curvature at Point "A":

$$a = \frac{b^2}{d}$$

From Equations (1) and (3):

$$a = \frac{H \sqrt{H^2 - V^2}}{2W} \quad (4)$$

H Height of Cavern
W Width of Cavern
V Length of Vertical Vertex
b Vertical Semiaxis of Ellipsoid
d Horizontal Semiaxis of Ellipsoid

STRESS DISTRIBUTION AROUND THE CAVERN

Let us consider the horizontal axis of the ellipsoid as sufficiently large to neglect its influence on the mean curvature at Point "A". In that case, we can consider a plane geometry in any vertical section. Figure 4 shows two sets of stress conditions:

- (1) Single cavern at geohydrostatic pressure σ_0 , with plastic zone extended to the Point "P" (Ref. 2, 3):

$$\sigma_{r1} = \sigma_c + 2\mathcal{K}_p \ln \frac{r}{a} \quad (5)$$

$$\sigma_{l1} = \sigma_c + 2\mathcal{K}_p \left(\frac{1}{2} + \ln \frac{r}{a} \right) \quad (6)$$

$$\sigma_{t1} = \sigma_c + 2\mathcal{K}_p \left(1 + \ln \frac{r}{a} \right) \quad (7)$$

Beyond the plastic zone, the stresses are distributed according to equations for infinite elastic solid (Ref. 4):

$$\sigma_{r2} = \sigma_0 + \mathcal{K}_p \frac{r_p^2}{r^2} \quad (8)$$

$$\sigma_{l2} = \sigma_0 \quad (9)$$

$$\sigma_{t2} = \sigma_0 - \mathcal{K}_p \frac{r_p^2}{r^2} \quad (10)$$

$\sigma_r, \sigma_l, \sigma_t$ Principal Stresses
 σ_0 Geohydrostatic Pressure
 $\mathcal{K}_p$ Parameter of Plasticity in Virgin Ground
 σ_c Cavern Pressure
a Radius of Curvature
r Distance from Centre of Curvature
 r_p Extent of Plastic Zone in Virgin Ground

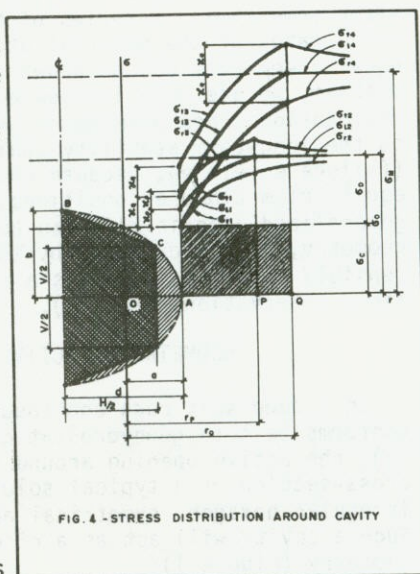


FIG. 4 - STRESS DISTRIBUTION AROUND CAVITY

At Point "P", $r = r_p$ and $\sigma_{l1} = \sigma_0$. Hence from equation (6):

$$r_p = a e^{\frac{\sigma_0 - \sigma_c - \gamma_p}{2 \gamma_p}} \quad (11)$$

As shown in (1) and (2), the velocity of flow will be distributed as follows:

$$\dot{\epsilon}_r = \dot{\epsilon}_A \frac{a}{r} \quad (12)$$

$\dot{\epsilon}_A$ Velocity at Wall
 $\dot{\epsilon}_r$ Velocity at Distance "r"

(2) Multitude of caverns at mining pressure σ_M , with plastic zone extended to the Point "Q":

$$\sigma_{r3} = \sigma_c + 2 \gamma_Q \ln \frac{r}{a} \quad (13)$$

$$\sigma_{t3} = \sigma_c + 2 \gamma_Q \left(\frac{1}{2} + \ln \frac{r}{a} \right) \quad (14)$$

$$\sigma_{t3} = \sigma_c + 2 \gamma_Q \left(1 + \ln \frac{r}{a} \right) \quad (15)$$

$$\sigma_{r4} = \sigma_M - \gamma_Q \frac{r_Q^2}{r^2} \quad (16)$$

$$\sigma_{l4} = \sigma_M \quad (17)$$

$$\sigma_{t4} = \sigma_M + \gamma_Q \frac{r_Q^2}{r^2} \quad (18)$$

$$r_Q = a e^{\frac{\sigma_M - \sigma_c - \gamma_Q}{2 \gamma_Q}} \quad (19)$$

σ_M Mining Pressure
 γ_Q Parameter of Plasticity in Pillar
 r_Q Extent of Plastic Zone in Pillar

Figure 5 shows a typical unit of triangular pattern of solution caverns. It is assumed that the extracted area is very large and that no arching has developed in the overlying strata. Reloading of the pillars will result in a change of average pressure at the core of the pillar from σ_0 to σ_M . A load equilibrium will be satisfied if:

$$L_0 = L_M$$

L_0 . . . Original Load over Area
 L_M . . . Load after Reloading over Area

$$L_0 = \sigma_0 A_0 = \frac{\sqrt{3}}{4} m^2 \sigma_0 \quad (20)$$

Utilizing 180° rotation about vertical axis of cavern:

$$L_M = \frac{\sqrt{3}}{4} m^2 (\sigma_M + \gamma_Q) - \frac{\pi}{2} d^2 (\sigma_c + 2 \gamma_Q) - \frac{\pi}{2} \int_a^{(r_Q + d - a)} (r + d - a)^2 \sigma'_{t3} dr \quad (21)$$

The number of caverns developed in the area, with respect to their depth, will not be sufficient to cause a total caving and subsequent reloading of pillars. A parabolic arching (stress envelope) will develop and the pillars will be only partially loaded. As shown in Reference (5), the maximum caving height " h_c " is given by (Figure 6):

$$h_c = \frac{1 - \nu_A}{\nu_A \tan \gamma_c} \cdot \frac{w}{2} \quad (24)$$

$$\gamma_c = \frac{\psi_A}{2} + 45^\circ \quad (25)$$

$\nu_A = 0.2$. . . Average Poisson's ratio of overburden

$\psi_A = 55^\circ$. . . Average angle of Internal Friction of Overburden

w Width of Mining Area (m)

Maximum pressure from caved overburden:

$$\sigma_w = \frac{D_A}{100} h_c \quad (26)$$

$D_A = 2.3 \text{ t/m}^3$. . . Average Density of Overburden

If caving occurs within the stress envelope, the pillars are loaded by the weight of the caved rock. In order to maintain the virgin ground stress distribution equation (23) must be satisfied letting $\sigma_M = \sigma_o = 43.0 \text{ MPa}$, $r_o = r_p = 23.4 \text{ m}$, $\alpha_Q = K = 9.0 \text{ MPa}$, $\sigma_o = \sigma_w$ (to be determined). The value of pressure satisfying these conditions is $\sigma_w = 33.9 \text{ MPa}$, and from Equations (26) and (24), $h_c = 1470 \text{ m}$ and $w = 2340 \text{ m}$. In other words, the virgin ground conditions would exist at proposed spacing $m = 160 \text{ m}$ if the total width of mining area is less than 2340 m , provided that the size of caverns is maintained. However, as described earlier, the average increase in cavern diameter will be 0.5 m/year and the required useful life is 50 years. Hence, the dimensions at the end of 50 years will be: $H^* = 50.0 \text{ m}$, $W^* = 115.0 \text{ m}$, $V^* = 25.6 \text{ m}$, $d^* = 66.9 \text{ m}$, $a^* = 9.3 \text{ m}$. Let us assume that the plastic zone at the end of the useful life will reach the centre point between caverns:

$$r_Q = \frac{m}{2} - d + a \quad (27)$$

By letting $r = r_o$ and $\sigma_{L3} = \sigma_M$ in Equation (14) and combining it with equations (23) and (27), we have:

$$\alpha_Q = \frac{\frac{1}{2} \sigma_o + \sigma_c \left(\frac{\pi d^2}{m^2 \sqrt{3}} - \frac{1}{2} \right)}{1 + \ln \frac{m-2(d-a)}{2a} - \frac{\pi}{m^2 \sqrt{3}} \left[\frac{m^2 + d^2 + 2(d-a)^2}{4} \left(\frac{m-2d}{d-a} + \ln \frac{m}{2d} \right) \right]} \quad (28)$$

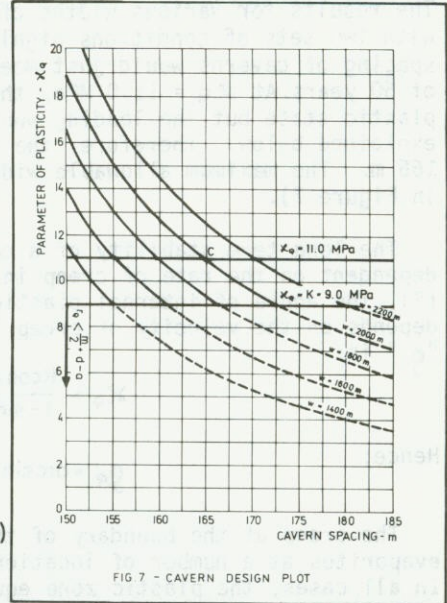


FIG 7 - CAVERN DESIGN PLOT

The results for various widths of mining area are plotted in Figure 7, with two sets of conditions highlighted. At $\alpha_Q = K = 9.0$ MPa, the spacing of caverns would just meet virgin ground conditions at the end of 50 years. At $\alpha_Q = 11.0$ MPa, the pillars would be totally in a plastic state but the loading would still be acceptable, as will be explained below. Therefore, the recommended cavern spacing is $m = 165$ m. The maximum allowable width of mining area is 2000 m (Point "C" in Figure 7).

The long term stability of a cavern in salt rock is primarily dependent on the rate of creep in the pillars. As shown in Reference (2), the angle of internal plastic friction " ϕ " is variable and depends on the velocity of creep " $\dot{\epsilon}$ ". The relationship of " α_Q " and " ϕ " is:

$$\alpha_Q = \frac{K \cos \phi_Q}{1 - \sin \phi_Q} \quad (29)$$

Hence:

$$\phi_Q = \arcsin \frac{\alpha_Q^2 - K^2}{\alpha_Q^2 + K^2} \quad (30)$$

The creep at the boundary of the plastic zone has been measured in evaporites at a number of locations in Saskatchewan and New Brunswick. In all cases, the plastic zone equilibrates in such a way that the elastic/plastic boundary creep velocity ϵ_P is about 20 μ m/day, although this figure was expected to be different in different materials. It appears that the depth of plastic zone varies with loading and the geometry but the steady state creep rate at its boundary does not. In the virgin ground cavern $\alpha_Q = K$, $r_Q = r_P = 23.4$ m, $a = 12.7$ m. Thus, total closure rate at Point "A" from Equation (12):

$$C_A = 2\dot{\epsilon}_A = 2\dot{\epsilon}_P \frac{r_P}{a} \quad (31)$$

$$C_A = 74 \mu\text{m/day} = 27 \text{ mm/year.}$$

The rate of creep at any distance " r " is directly proportional to the first derivative of " σ_r ":

$$\dot{\epsilon}_P \sim \sigma'_{rP} = \frac{2\alpha_P}{r_P} \quad (32)$$

$$\dot{\epsilon}_Q \sim \sigma'_{rQ} = \frac{2\alpha_Q}{r_Q} \quad (33)$$

Let $\alpha_P = K$, $r_P = r_Q$. Then:

$$\dot{\epsilon}_Q = \dot{\epsilon}_P \frac{\alpha_Q}{K} \quad (34)$$

And since: $\dot{\epsilon}_A = \dot{\epsilon}_Q \frac{r_Q}{a}$ (35)

Then: $\dot{\epsilon}_A = \dot{\epsilon}_P \frac{r_P \alpha_Q}{aK}$ (36)

At the end of 50 years, $r_P^* = \frac{m}{2} - d^* + a^* = 24.9$ m and $\alpha_Q = 11.0$ MPa.

Thus, total closure rate:

$$C_A = 2 \dot{\epsilon}_P \frac{r_P^* \alpha_Q}{a^* K} \quad (37)$$

$$C_A = 131 \mu\text{m/day} = 48 \text{ mm/year}$$

$\dot{\epsilon}_A$ Creep Rate at Point "A" at Wall

$\dot{\epsilon}_Q$ Creep Rate at Point "Q"

$\dot{\epsilon}_P$ Creep Rate at Plastic/Elastic Boundary

The total cavern closure at the end of 50 years is about twice the virgin ground rate, which is acceptable.

CONCLUSION

The equations developed in this paper permit the safe design of solution mined salt caverns under the specified conditions. The additional washing of the cavern walls by the unsaturated brine during cavern operation and the resulting enlargement of the cavern over time in the described case was shown to substantially exceed closure effects due to creep. The proposed cavern spacing will ensure stable pillars over the design period of 50 years.

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Chapter 58

LEACHED SALT CAVERN DESIGN USING A FRACTURE CRITERION FOR ROCK SALT

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INTRODUCTION

In 1975 Congress passed the Energy Conservation Act to establish a U. S. Strategic Petroleum Reserve (SPR) with a capacity of 750 million barrels of crude oil. The most economic storage medium was determined to be salt caverns leached in salt domes in Louisiana and Texas. Salt caverns existed at several sites when the reserve was created. These were obtained by the U. S. Department of Energy (DOE) and used to initiate SPR oil storage. In order to meet the storage capacity approved by Congress, new caverns also had to be leached. To support the resulting design effort, finite element computer programs have been used to determine the creep closure and structural stability of salt caverns. Using site specific material properties including creep models, elastic moduli and fracture data, the finite element analyses have replaced earlier empirical approaches to cavern design. This report presents results of such finite element analyses to determine the best cavern roof shape and the minimum pillar to diameter ratio, P/D. These numerical predictions indicate that the current cavern design is safe.

FINITE ELEMENT AND MATERIAL MODELING

The finite element program used in this study has existed for approximately four years. Its theoretical basis (Key, et al, 1980) and application are well documented. It was originally developed to predict the creep around mined openings in bedded rock salt for the Waste Isolation Pilot Plant (WIPP). The program was tested by modeling the creep of salt around a drift and by comparing the results with the predictions of eight other structural computer codes (Morgan, et al, 1981). Since then the program has had continual use for calculations to support the WIPP (Miller, et al, 1982). For about two years the

program has also been used to calculate the creep and structural stability of SPR salt caverns (Preece and Stone, 1982). Other applications include the analysis of laboratory creep tests that are used to develop creep models for salt (Branstetter and Preece, 1983).

The program numerically simulates the creep of rock salt and the resulting stresses, strains and displacements as functions of time. The constitutive model used has elastic and creep components where strain rate is a power function of effective stress as given below.

$$\dot{\epsilon} = A \bar{\sigma}^n \quad (1)$$

where

$$\dot{\epsilon} = \sqrt{(2/3) \dot{\epsilon}_{ij} \dot{\epsilon}_{ij}}$$

$\dot{\epsilon}_{ij}$ = plastic strain rate tensor

$$\bar{\sigma} = \sqrt{(3/2) S_{ij} S_{ij}}$$

S_{ij} = deviatoric cauchy stress tensor

A = laboratory determined constant

n = laboratory determined stress exponent

The elastic constants and the creep parameters used are listed in Table I. They are based on extensive creep data on rock salt from the Salado formation in New Mexico and on limited but site specific measurements on salt samples from West Hackberry and Bryan Mound (Wawersik, et al, 1980a and 1980b). The parameters in Table I either match or overpredict the creep rates that are recorded in laboratory tests. Within the context of this study, overpredictions are considered conservative and acceptable.

TABLE I

Material Properties of Regions in Figure 3

Elastic Material	Youngs Modulus (Pa)*	Poisson's Ratio
Shale	7.09 E 8	.288
Caprock	7.09 E 8	.288
Overburden	9.58 E 5	.330
Salt	A [(Pa) ^{4.9} days] ⁻¹ N	Shear Modulus (Pa) Bulk Modulus (Pa)
	4.82 E -39	4.9 1.75 E 10 1.55 E 10

* Intact modulus (Winterkorn, 1975) divided by ten.

Rock salt fracture was treated in two ways. As usual, tensile failure was described by means of a cut-off stress. In this case, the cut-off stress was set equal to zero. Compressive fracture was modeled by means of a limiting inelastic strain criterion. The limiting strain, in turn, is a measure of damage before the rock ruptures in the sense that it loses some or all of its load bearing ability. At this time, the criterion does not consider the effect of fracture healing. The compressive fracture criterion used is based on empirical observations in triaxial compression tests [Wawersik, 1974]. Accordingly, the axial strain to rupture, e_1^r , is given by the sum of instantaneous strains and time-dependent strains, $e_1^r = e_1^i + e_1^c$. Referring to Fig. 1, e_1^i is the inelastic strain upon loading to any stress $\tau = \sigma_1 - \sigma_3$ in the pre-failure regime. e_1^c is the time-dependent strain past e_1^i as observed, for example, in a creep test. Measurements indicate that e_1^c of salt is between the ultimate strain, e_1^u , in quasi-static tests and the quasi-static post-failure strain at the same strain rate and at the same value of σ_3 . Laboratory tests on salt further suggest that the post-failure behavior of rock salt may be approximated by a line of slope β (Figure 1) that increases rapidly with the smallest principal compressive stress, σ_3 , and temperature, T . Combining these observations, it was possible to write

$$e_1^r = e_1^i + e_1^c \leq e_1^u + \frac{\tau - \tau_u}{\beta} \quad (2)$$

with, for example, $\frac{1}{\beta} = -\exp \frac{U}{T} (a\sigma_3^m + b)$. The parameters, U , a , m and b are constant. τ_u is the ultimate stress (Figure 1).

To bound rock salt failure in general, equation (2) was rewritten by expressing the least compressive stress in terms of the mean stress, σ and the effective stress, $\bar{\sigma}$, and by replacing the e_1^i by $\bar{e}$, the effective strain, and τ by $\bar{\sigma}$. Thus, setting $\frac{1}{\beta} = \alpha$,

$$\bar{e}^r = \bar{e}^i + \bar{e}^c \leq \bar{e}^u + \alpha(\bar{\sigma} - \bar{\sigma}_u) \quad (3)$$

where $\alpha = \alpha(\sigma, \bar{\sigma}, T)$. Equation (3) may be recast in terms of strain rate by substitution of τ using the rate Eqn (1).

To evaluate the stability of caverns, it was important to be conservative. This was attempted by assuming $\alpha = 0$ and using only ultimate strains that were measured at ambient temperature. Under these conditions, early laboratory compression data at strain rates $\dot{e} \approx 10^{-5} \text{ s}^{-1}$ were fitted by a second order polynomial to yield limiting strain

$$\bar{e}^r \approx \bar{e}^u = 2.30 + (4.57 - 0.357 \sigma_3) \sigma_3 \text{ for } \sigma_3 \leq 3.45 \text{ MPa}$$

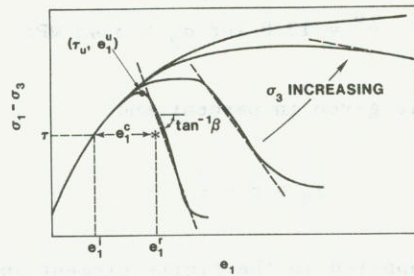


Figure 1. Schematic of Complete Quasi-Static Stress-Strain Curves, $(\sigma_1 - \sigma_3)$ vs e_1 , and Empirical Creep Rupture Parameters for Rock Salt.

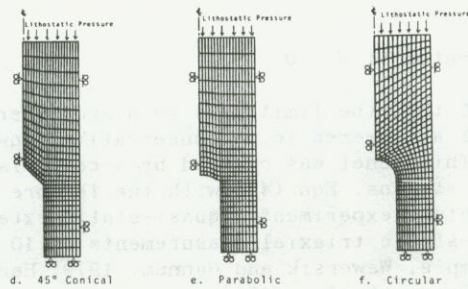


Figure 2. Finite Element Models of Roof Configurations

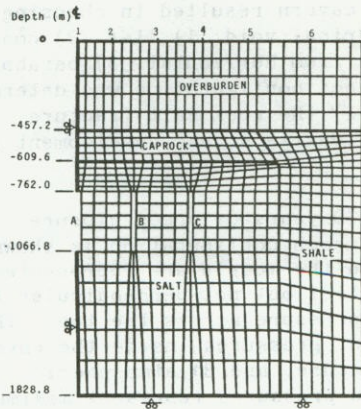


Figure 3a. Axisymmetric Finite Element Model of West Hackberry Dome with New Caverns at P/D of 1.8.

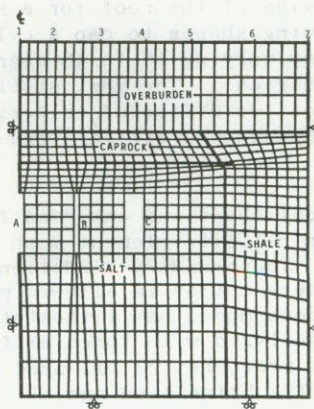


Figure 3b. Axisymmetric Finite Element Model of West Hackberry Dome with New Caverns at P/D of 1.0.

$$\bar{\epsilon}^u = 13.8 \text{ for } \sigma_3 > 3.45 \text{ MPa} \quad (4)$$

where the strain $\bar{\epsilon}^u$ is given in percent and

$$\sigma_3 = \sigma - \frac{\bar{\sigma}}{3}.$$

Equation (4) was applied in the finite element analyses by comparing the calculated effective strains, $\bar{\epsilon}$, with the predicted limiting strain, $\bar{\epsilon}^u$. This comparison was done formally by means of the failure function

$$F = \bar{\epsilon} - \bar{\epsilon}^u. \quad (5)$$

Stability is indicated if $F < 0$.

It is important that the limiting strain and, therefore, the failure function F are deemed to be conservative, lower bound approximations. This tenet was checked by a comparison of the predicted failure strains, Eqn (4), with the failure strains of triaxial creep rupture experiments, quasi-static extension tests and more recent quasi-static triaxial measurements to 10.3 MPa confining pressure [for example, Wawersik and Hannum, 1979; Hansen and Mellegard, 1979; Price et al., 1981].

ROOF SHAPE STUDIES

The design of the roof for a new cavern resulted in choosing from the following shapes to cap a cylindrical void: 1) flat, 2) conical with angles varying 15 to 45 degrees from horizontal, 3) parabolic, and 4) spherical. The most stable roof configuration was determined by means of two different criteria: 1) the rock salt fracture criterion presented earlier, and 2) tensile stress development in the roof.

Each roof shape was analysed for 30 years of creep closure. The axisymmetric finite element mesh for each configuration is shown in Figure 2. The boundary conditions on the models are represented by rollers where motion is allowed parallel but not perpendicular to each roller. Each model has lithostatic pressure across the top. The flat roof was analysed with three different pressures inside the cavern 1) brine head pressure, 2) oil head pressure, and 3) atmospheric pressure. The brine head and oil head pressures represent maximum and minimum pressures that could occur during normal cavern operations. The atmospheric pressure case represents the most severe loading condition the cavern could ever be subjected to. The models with

conical roof angles were analysed with brine head and atmospheric pressure inside the cavern. The parabolic model was analysed with brine head pressure.

Numerical values for the two stability criterion are listed in Table II for each configuration. When compressive failure alone is considered, the flat roof appears to be the most stable roof shape since it yields the lowest value of the fracture function. This is due to a bridging effect that shifts some of the load away from the cavern roof to produce a lower creep rate. However, when the flat roof is subjected to atmospheric pressure, the minimum stress becomes tensile which makes the flat roof least desirable. The 15 and 30 degree conical configurations had compressive stresses in the roof even at atmospheric pressure. The 45 degree conical configuration at atmospheric pressure would not converge past the first several time steps. At that point, however, the stresses in the roof were well into the compressive range. In examining the fracture function values in Table II, it is noted that all are less than zero. Hence, according to this criterion, no slabbing will occur and stability at this depth is ensured.

TABLE II: SUMMARY OF ROOF SHAPE STUDIES

Configuration	Type of Internal Pressure	Creep Strain At 30 Yrs (%)	Mean Stress in Roof (MPa)	Fracture Function %	Minimum Principle Stress in Roof (+ Compression MPa)
Flat	Brine Head	0.23	10.7	-13.57	8.92
"	Oil Head	0.50	9.98	-13.3	8.79
"	Atmosphere	8.7	8.26	-7.1	-0.128
15° Conical	Brine Head	0.31	11.7	-13.49	8.92
"	Atmospheric	9.80	8.09	-4.0	0.0163
Parabolic	Brine Head	0.35	11.7	-13.45	8.988
30° Conical	"	0.43	16.7	-13.37	9.01
"	Atmospheric	14.0	8.86	-0.20	0.427
45° Conical	Brine Head	0.65	11.7	-13.15	9.07
"	Atmospheric	- - -	- - -	- - -	8.20
Circular	Brine Head	6.75	11.8	-7.05	8.41

PILLAR TO DIAMETER RATIO

The ratio of pillar to cavern diameters (P/D) was evaluated by means of two criteria: (1) the absence of fracture in tension or compression, and (2) the percent volume loss of the cavern due to creep closure. These criteria were applied in the analysis of a new cavern array that was adopted for several SPR facilities currently under construction.

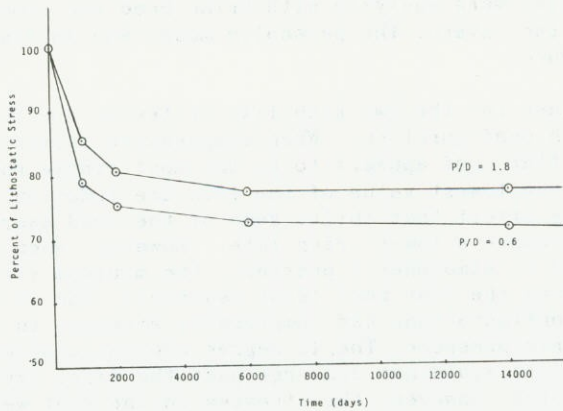


Figure 4. Vertical Stress History 152 Meters Above Torroid B.

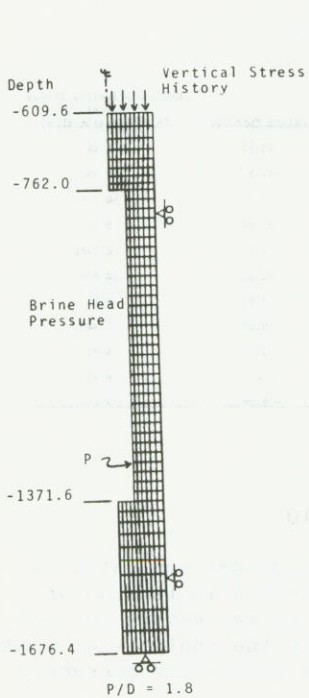


Figure 5. Detailed Axisymmetric Finite Element Model of a Cylindrical Cavern.

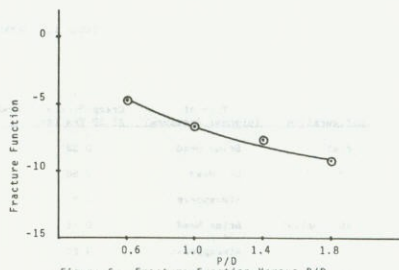


Figure 6. Fracture Function Versus P/D .

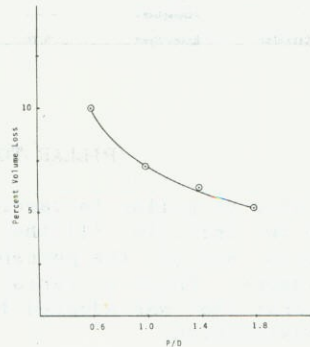


Figure 7. Percent Volume Loss versus P/D

Two analyses were carried out. The first was a simplified but large scale axisymmetric model of the entire dome to investigate coupling between the structural response of individual caverns and the entire salt dome. It showed that, due to creep relief, the stresses around the caverns are reduced from the original lithostatic stress as some of the overburden load is shifted to the caprock or transferred to the shale and sand layers along the sides of the dome. The computed stresses from the large-scale model were then used to set the boundary conditions for a more detailed analysis of the actual cavern complex. The large scale finite element model is shown in Figure 3a. Both sides of the model were allowed to move vertically but not horizontally. The bottom is allowed to move horizontally but not vertically. The top, which represents the ground surface, is unconstrained. The four different materials included in the model are labeled on the mesh and their corresponding properties are given in Table I. The low moduli for shale, caprock and overburden accounted for discontinuities in the rock, consistent with some subsidence predictions using linear elastic finite elements (Dahl and Choi, 1973). The caverns were modeled as a central cylinder, "A" surrounded by two toroids "B" and "C". The cylinder and toroids are configured to approximate the volume and volumetric distribution of the actual West Hackberry SPR site.

The model also approximated the entire history of the cavern complex from the creation of the first caverns in 1940 to the caverns currently being leached. Starting with lithostatic stresses everywhere, the pressure in cylinder "A" and toroid "B" was lowered gradually to brine head pressure over 4000 days (11 years) to simulate the first leaching phase. The computation was continued at constant brinehead pressure within caverns "A" and "B" until, after 11,000 days (30 years) leaching of the new caverns was simulated by reducing the pressure in toroid "C" to simulate leaching the new caverns. Gradual depressurization of "C" occurs over 3000 days (8 years) and is followed by constant pressure conditions for 30 years, the projected life of the SPR.

Because of creep relief, the vertical stress in the region immediately above the caverns decreases as some of the overburden load shifts to the shale surrounding the dome. The vertical stress in the shale increases as a result of this load transfer. Cavern analysis which uses the original lithostatic stress above the cavern as a boundary condition will overpredict the closure of the cavern. The vertical stress history 152 meters above toroid "B" represents an average for the region above the caverns. This history, shown in Figure 4, was applied across the top of the detailed axisymmetric finite element models of the new caverns shown in Figure 5. To evaluate the pillar to diameter ratio, P/D was varied from 1.8 to 0.6 by changing the mesh width while the cavern radius remained constant. Each model had the vertical stress history from the previous analysis across the top and brine head pressure inside the cavern. These models were analysed for 30 years and the results were post-processed for stability and volume change.

The cavern stability was evaluated by means of the fracture function, "F" Eqn (5). Values of "F" were calculated by post-processing the creep strains and stresses from each analysis. Accordingly, "F" varies vertically along the cavern wall and is at a maximum at approximately point "P" shown in Figure 5. Values of the fracture function at "P" in each model after 30 years of creep is shown in Figure 6 as a function of P/D. The plot indicates that all pillar widths are predicted to be stable.

The cavern volume decreases with time due to creep closure. The calculated volume loss for each cavern after 30 years of closure is shown in Figure 7 as a function of P/D. Figure 7 indicates that the volume loss from the caverns is within acceptable limits when the P/D is reduced.

CONCLUSIONS

The results of several finite element analyses of salt creep and fracture around SPR caverns are consistent with the pillar to diameter ratio of 1.8 that is currently applied to the design of SPR caverns. The calculations further indicate that this spacing has an adequate but not overly conservative safety factor of approximately two. Moderate reductions of P/D from 1.8 should not result in cavern collapse. To ensure cavern stability along the roof, conical shapes with roof angles between 15 and 30 degrees appear to be optimal. In the future, it is planned to also evaluate the effect of P/D = 1.8 on the magnitude of surface subsidence at each SPR site.

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Chapter 59

NUMERICAL MODELING OF "STIFF" BACKFILL IN THE COEUR D'ALENE MINING DISTRICT

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ABSTRACT

As part of a U.S. Bureau of Mines-sponsored project on the utilization of "stiff" backfill to minimize potential rock burst hazards, a finite element study was undertaken to predict stresses and displacements around a stope mined up from the 5400 level of the Sunshine mine, Kellogg, Idaho. This stope was to be backfilled using a high-modulus fill. In preparation for that configuration, certain instrumentation was installed near the 5400 level to monitor stress changes. Due to untenable economics, only 2 cuts of the stope were ever made. This paper presents the results of the instrument readings taken during the first cut and compares them with the numerical predictions, as well as presents the modeling results of closure and fill stresses in the stope if it had been mined.

INTRODUCTION

As mining depths have increased, greater and greater problems have been encountered with rockbursts. One mining district that has had such problems is the Coeur d'Alene silver mining district of northern Idaho. Various researchers have proposed different schemes for alleviating the pressures leading to rock bursts, and each has had its degree of success. Most companies in the district use some form of layout control and/or destressing. Perhaps the best overview of the rock mechanics problems in the district is summarized by Corp (1981).

One idea that has been proposed for alleviating rock burst problems is to utilize stiffer than normal (i.e., high-modulus) backfill. The basis for the concept is that if a high-modulus fill were used, it would absorb more load than regular fill (as the stope converged),

thus alleviating some of the pressure which would normally go on the abutments, which causes rockbursts. Stout and Friel (1980) investigated methods for increasing the stiffness of fills, and found that it could be done by vibrating or "double-placing" fill, which essentially decreases the void volume. The concept of decreasing void volume, thus stiffening the fill, was expanded upon by Hustrulid and Moreno (1981). Essentially, stiffer fill could be used, but will it help?

That question is a critical one. In order to answer it, arrangements were made with the Sunshine Mine, of the Coeur d'Alene mining district, to monitor one of their stopes in terms of fill pressures and closures, as well as abutment pressures. The stope to be used was one in which high-modulus fill was to be used, and it was to be compared to another with no such high-modulus fill. That work was funded by the U.S. Bureau of Mines out of the Spokane office (Stout et al., 1983), and is the basis for this paper. The fill configuration was to be similar in nature to that proposed by Stout and Friel (1980, p. 804)--that is, a double-placed center surrounded by vibrated material. The instrumentation program to monitor the mining was quite detailed, but is not the principal subject of this paper. However, some vibrating wire stressmeter field results were obtained, and will be presented here.

COMPARISON OF FIELD MEASUREMENTS TO NUMERICAL PREDICTIONS

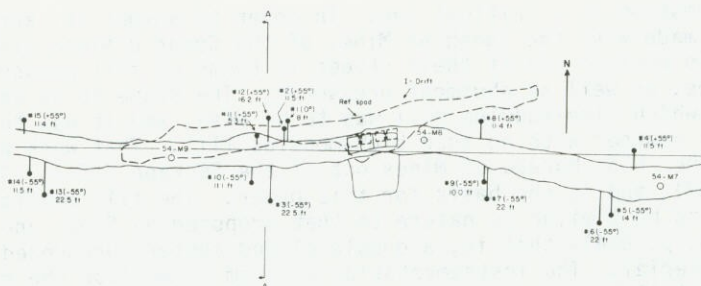
Early in 1981, fifteen vibrating wire stressmeters (VWS) were installed in the vicinity of the 5400 level of the Sunshine Mine. These fifteen cells were part of an overall instrumentation plan to monitor the stope. Installation was under the direction of the author, from Montana Tech, and D. Rankin, at the site. A location map of all cells installed is presented as Fig. 1. Note this figure is a plan view, but all cells but one were installed at 55° from horizontal, to correspond to the dip of the vein. The cells were oriented to measure stress changes perpendicular to the vein.

No particular problems were noted during installation, but one of the cells (number 7) was lost early on. Fourteen others survived the installation and gave consistent readings from March of 1981 through March 1982. This is the period during which the first I-drift above the 5400 level was cut. Another cut above that was partially taken before the mine was closed.

Measurements obtained were used to calculate stress changes observed (as the I-drift was advanced) as described in IRAD GAGE instruction manual (1977, pp. 26-30). The hard rock formula used was:

$$\sigma_r = \frac{\left(\frac{422400}{T_o} \right) \left\{ 1 - \left(\frac{T_o}{T} \right)^2 \right\}}{9.4 - 0.5 \times 10^{-6} E_r}$$

where σ = stress change calculated,
 T_o = initial stabilized VWS reading,
 T = VWS reading on date of interest,
 E_r = rock modulus of elasticity.



Scale: 1 in = 60 ft

Fig. 1 - Plan view of instrumentation installed near the 5400 level.

The first concern in utilizing this equation is the question of the proper modulus of elasticity to be used. The modulus of elasticity representing intact specimens of the surrounding rock can vary from 6 to 10 times 10^6 psi (Beus and Chan, 1980, pp. 27-28, 39). Pariseau (1981) uses a value of 1×10^6 psi in his finite element method (FEM) simulations, but that value was obtained by derating the intact modulus in a manner similar to that cited in Lama and Vutukuri (1978, pp. 287-292). Derating is due primarily to structural features that need to be accounted for in the larger picture. Since the modulus of elasticity desired for the equation above is that immediately surrounding the borehole, a value of 8×10^6 psi was chosen--half-way between the extremes cited in Beus and Chan (1980). Using this value, one can calculate the stress changes each VWS underwent. Fig. 2 shows how those calculated values changed with time for the 14 cells monitored.

From Fig. 2, it is obvious that cell nos. 2 and 12 show the highest calculated stress changes, about 1700 and 2800 psi respectively (remember, using $E_r = 8 \times 10^6$ psi in the formula). The rest of the cells were somewhat less active. From Fig. 1 one can ascertain that cell nos. 2 and 12 should record high values, since they are located very closed to the I-drift, and are in a thinner pillar than cell no. 8 is, as a comparison. A detailed profile view is needed, but not enough surveying information was received from the field to compile

one. Additionally, it appears that a first cut (above the I-drift) had been partially completed when the last reading was taken (Rankin, 1982), explaining the sudden jump in calculated stress change in cell no. 12. Unfortunately, no access to the site for surveying or monitoring purposes since March of 1982 has been possible to verify this sudden change observed.

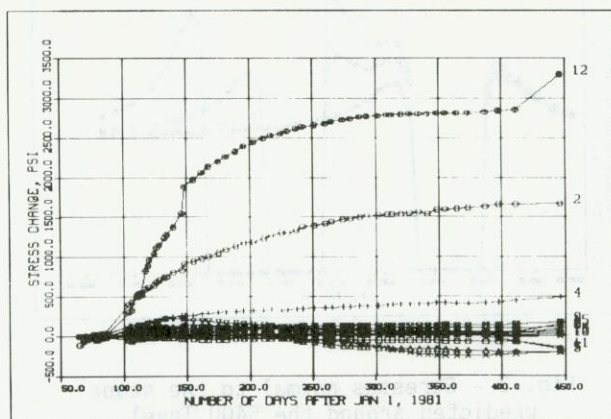


Fig. 2 - Calculated stress changes for all cells monitored.

Finite Element Predictions

A strictly linearly elastic FEM mesh was created to model the 5400 level, subsequent I-drift and first cut. Hydrostatic pressures of 6,000 psi, a modulus of elasticity of 1×10^6 psi and a Poissons ratio of .25 were used. Field measurements of in-situ stresses show a somewhat higher value in the area of interest (Corp, 1981), but 6,000 psi was arrived at by multiplying 1.1 times the overburden. Also, 1×10^6 psi was used for the modulus of elasticity simply as a benchmark figure (i.e., the same as used by Pariseau, 1981). Actually, the value of this modulus will not affect the stresses predicted here.

The "vertical" (i.e., σ_{yy}) stresses predicted plotted along the x-axis from this simulation are shown in Fig. 3. Note that in-situ stresses stabilize to 6,000 psi at the margins of the plot, as they should. Three calculated stress situations are shown: just the 5400 level drift, that drift plus the I-drift, and that situation plus one cut. This figure is quite interesting, and stresses predicted are about what one would expect for each situation. In the field, the VWS cells observe changes, not absolute stresses as this figure shows. A more useful plot would be one which directly plots those

predicted differences. Fig. 4 is such a plot. Two stress change curves are shown: the lower one shows the effect of the I-drift, the higher one shows the effect of the I-drift plus one cut.

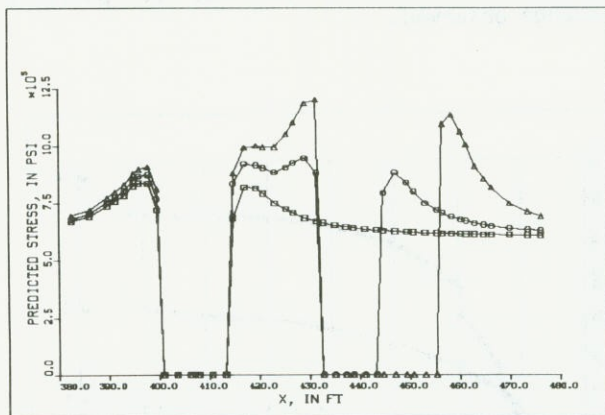


Fig. 3 - Stresses normal to the stope predicted around the 5400 level.

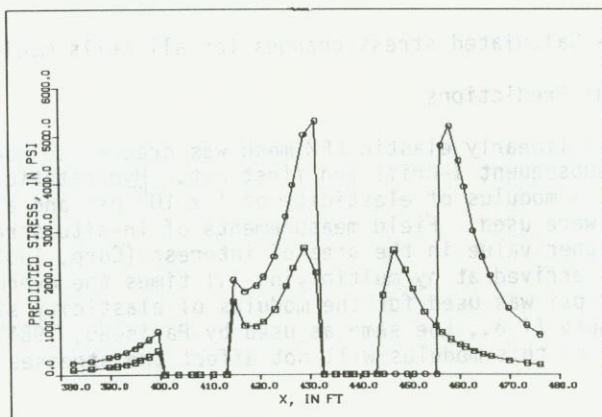


Fig. 4 - Increase of stresses predicted due to the I-drift and one additional cut.

It is of interest to note that in Fig. 4 the location of cell no. 12 (i.e., 16.2 ft into the pillar, towards the stope) is predicted to have a stress increase of about 2500 psi, cell no. 2 (at 11.5 ft into the pillar) 1000 psi, cell no. 10 (11.1 ft into the vein away from

the stope) 10 psi, and cell no. 3 (22.5 ft into the vein, away from the stope) practically zero (actually, it's off this particular figure).

Discussion

A direct comparison of FEM predicted versus observed (but calculated) stress changes in the profile view section A-A (see Fig. 1) is given in Table 1.

Table 1 - Stress changes predicted versus those observed near the 5400 level.

<u>VWS Cell No.</u>	<u>Predicted Stress Change, psi</u>	<u>Observed Stress Change, psi</u>
3	~0	~0
10	100	~0
11	1000	~0
2	1500	1700
12	2500	2800

Cell no. 1 was ignored since it was installed horizontally, not at 55°.

Cell no. 11 is really the only cell not comparing well. It is remarkable (perhaps fortuitous?) that cell nos. 2 and 12 agree so well. Any differences in Table 1 could be zeroed out easily by an in-situ stress increase in the model, a slight change in the geometry or dimensions of the model, or a change in the modulus used in the VWS formula. More observations of these cells would certainly be interesting.

MODELING OF STOPING USING HIGH-MODULUS BACKFILL

As the stope in question was to be mined, additional instrumentation was to be installed in the fill and in the wall rock. Instrumentation results were then to be compared to predictions made using FEM simulation. This would allow some "retrofitting" of the FEM model. Since the stope was not mined, no input was received from field monitoring. However, the entire sequence of mining and backfilling was simulated on the computer. That simulation is the subject of this section, and gives some insight into what might be expected in terms of convergence and stress changes if the stope had been mined.

Description of Stope Model

This study is not the first to look at FEM simulation of cut and fill mining (see, e.g., Pariseau, 1981), but is intended to study how different fill configurations will affect convergence and abutment pressures using a somewhat different approach. This particular

effort uses a FEM code entitled ADINA (ADINA Engr'g., 1981), which was chosen for its nonlinear characteristics (Stout et al., 1983). The wall rock can be modeled as linearly elastic, but the fill needed to be nonlinear in some fashion. This particular code was installed on a DEC VAX 11/780 at Montana Tech, and compiled the first time--an impressive feat.

First of all, Fig. 5 shows the overall 2-dimensional mesh used in the problem. This represents a profile view of the stope looking west. The center bottom area is where the stope is being mined. One must imagine the stope would be tilted 55° counterclockwise in the field, but since in-situ stresses of 6,000 psi were used (in both the x and y directions), the angle of the simulation is unimportant. The overall mesh size is 2240 by 1000 feet. The model was made so that the 5400 level drift was 10 feet wide and 5 feet high (since the problem is symmetric along the stope axis, only one-half the problem need be modeled). A 20 foot pillar is left (1010 to 1030) on the x-axis, and the stope is subsequently mined, left to right. The 5200 level would be at the far right (1230 to 1240). A similar pillar is left at the right side. In all, 18 separate cuts and fills are made. These may not correspond exactly to the measurements of the stope under question, but certainly are close enough for the purposes of this work.

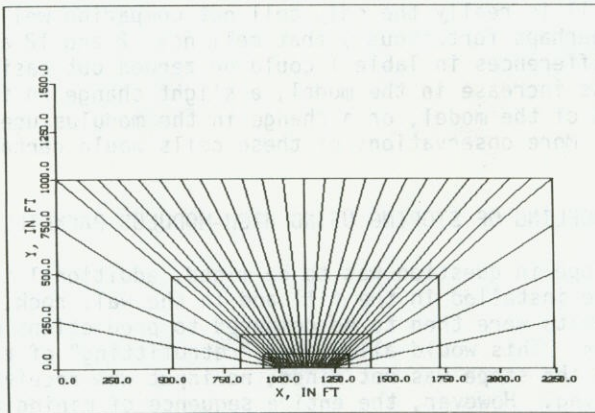


Fig. 5 - View of 2-dimensional ADINA mesh used.

In the sequence of the model, a cut (4 elements) is mined (using the ADINA "death" option), and the cut mined one step previously is backfilled by the "birth" option using elements with different material properties than the in-situ rock. Death and birth sequences are controlled by a time function that is input by the operator. As a case in point, the 5400 and 5200 level are "mined out" at 1.2 sec, the I-drift at 2.2 sec, the first cut at 3.2 sec, the second at

5.2 sec, the third at 7.2 sec, etc. These are not "real" times, only imaginary timesteps performed by the run. Backfilling begins at 4.4 sec (there is nothing magic about 4.4, only that it is between 4 and 5) with the I-drift, 6.4 sec with the first cut, etc. A solution cycle during mining is one cut and fill (that is, 2 sec). A printout at 10, 20, 30, and 40 sec time-step solutions during each simulation run--the 4 printouts are simply "snapshots" along that route. The total solution time for each simulation is 10.0 hours on Tech's VAX 11/780.

In terms of material properties and models used, the material surrounding the stope was assumed to have a modulus of elasticity of 1×10^6 psi, or 144×10^6 psf (since ft are simpler to use here), and a Poissons ratio of .25. It also was assumed to be isotropic thermo-elastic (model "3" - see p. XII.15 of ADINA Engr'g., 1981). The "thermo" portion was used for loading by changing the temperature between time 0 and time 1 sec. Thus the structure was loaded without moving the nodes.

The fill material was a bit more complicated. ADINA's material model "4" (ADINA Engr'g., 1981, pp. XII.17 - XII.22) corresponded most closely to that of fill material, since it was a curve description model using volumetric strain, bulk modulus, and shear modulus. The volumetric strain and bulk modulus were measured by Pope in a previous work (Stout et al., 1979, p. 92), so it was convenient to utilize that data as input into the model. The article by Hustrulid and Moreno (1981) supports this approach also. Tangent properties to the curves describing the material properties used are summarized in Table 2. The properties were taken from p. 92 of Stout et al. (1979).

TABLE 2 - Fill properties used
in the FEM model

Volumetric Strain	Bulk Modulus, $\times 10^6$ psf		
	Regular Fill	Vibrated Fill	Double Placed Fill
0.0	.0954	.331	.920
.025	.2708	.553	1.228
.05	.5644	.924	1.792
.075	1.029	1.452	3.109
.125	1.768	2.445	4.778
.3	2.895	3.154	5.00

The next parameter of interest to be input is G, the shear modulus. According to Jaeger and Cook (1979, p. 111), the bulk modulus K relates to G as follows:

$$K = \frac{2(1+\nu)G}{3(1-2\nu)}$$

Consequently, the parameter ν (Poisson's ratio) is important for estimating G . There seems to be some disagreement on its value for fill, so a value of .25 was assumed initially. Actually, it probably varies similar to K in some manner.

To summarize, the stope is modeled using the ADINA FEM code by using time-steps from 0 to 40 sec. Load is applied by heating, using the appropriate coefficient of thermal expansion to yield 6,000 psi in-situ stresses. Cuts are made at various time steps by using ADINA'S "death" option. Fills are made at various time steps by using ADINA'S "birth" option. The fill is a nonlinear model using parameters discussed previously. Output of stresses and displacements are then obtained and processed for ease of plotting. Of greatest interest are the normal stresses along the x-axis (the stresses perpendicular to the stope) and displacements of the stope wall.

Results Obtained

A series of 6 different computer runs were executed for this study. Each of these runs yielded 4 "time-shots" of the stresses and displacements surrounding the stope in question. These runs used the following different scenarios:

1. no fill,
2. "regular" fill ($\nu=.25$),
3. vibrated fill ($\nu=.25$),
4. double-placed fill ($\nu=.25$),
5. double-placed fill ($\nu=0.0$), and
6. mixed, with the center portion (4 cuts) double-placed fill ($\nu=.25$), and the rest vibrated fill ($\nu=.25$).

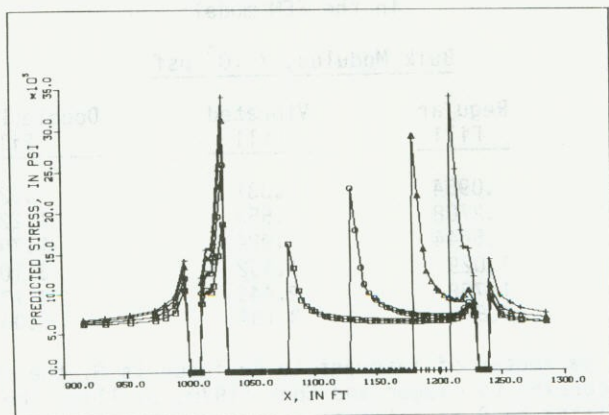


Fig. 6 - Comparison of stresses predicted at 4 levels of stope completions using no fill.

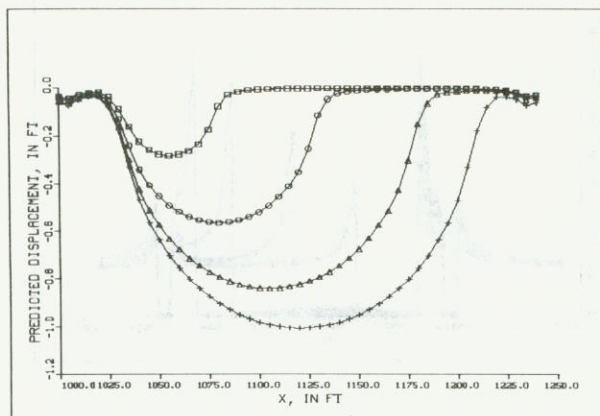


Fig. 7 - Comparison of stope displacements predicted at 4 levels of stope completion using no fill.

The first run made is an obvious one--a stope with no fill. Figures 6 and 7 show output results from that run. Fig. 6 shows the stresses (those stresses normal to the stope in a profile view) after 28, 56, 83, and 100 percent of the stope had been mined. All results appear reasonable, with no real surprises. The maximum abutment stresses predicted after 18 cuts was about 34,500 psi. Fig. 7 shows the displacements across the stope calculated for each time period. The maximum displacements predicted after 18 cuts was about 1.0 foot. Remember, this is only one-half the convergence expected for a full 10-foot wide stope, since the model used only 5-foot wide openings and a symmetric result is expected if a mirror analysis were performed.

The second run, the one using "regular" fill (see Table 2 for the properties used), yielded results shown in Figs. 8 and 9. As one views Fig. 8, it becomes apparent that the fill is not taking a great deal of load. The maximum stress observed in the fill is about 1,000 psi, but if one integrates the area under the curve (by planimeter or computer) of fill pressures, the fill has absorbed about 7% of the original stresses normal to the stope. The abutment pressure has decreased to about 32,000 psi (a 7% reduction from the first run). Fig. 9 shows the maximum displacement is now about .92 foot, an 8% reduction from the first run. Consequently, "regular" fill does assist somewhat in ground stability.

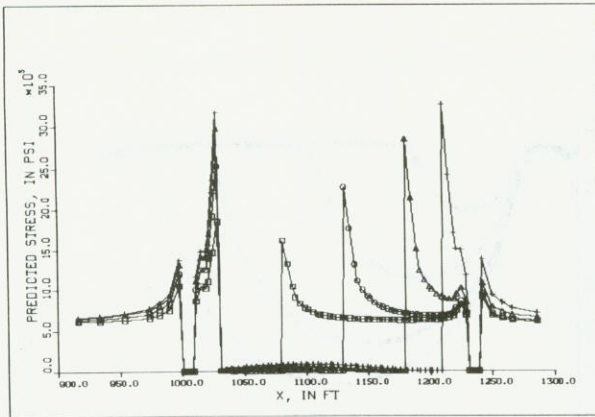


Fig. 8 - Comparison of stresses predicted at 4 levels of stope completion using "regular" fill.

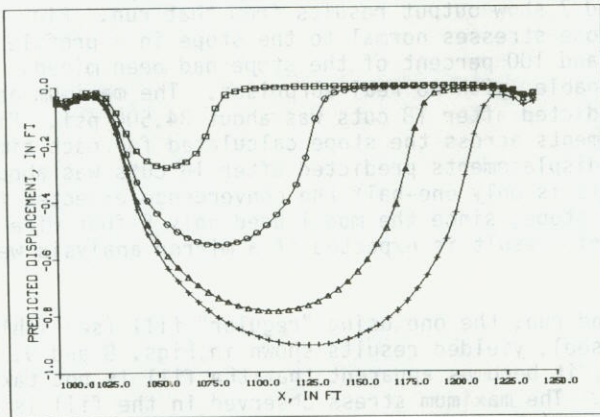


Fig. 9 - Comparison of stope displacements predicted at 4 levels of stope completion using "regular" fill.

The results from the final 4 runs are tabulated in Table 3. The figures showing stress and convergences from each run look similar, so are not repeated here. The final run is perhaps interesting to look at, and the results are shown in Figs. 10 and 11. The mixed fill situation shows a high percent of reloading on the center core,

but does not do as well on the sides, as might be expected given the center stiff versus the side softer portion. However, the results are much better than the vibrated fill and almost as good as the doubleplaced fill, which seems surprising. This reaffirms the initial design concept of a center stiff core as being a good compromise situation.

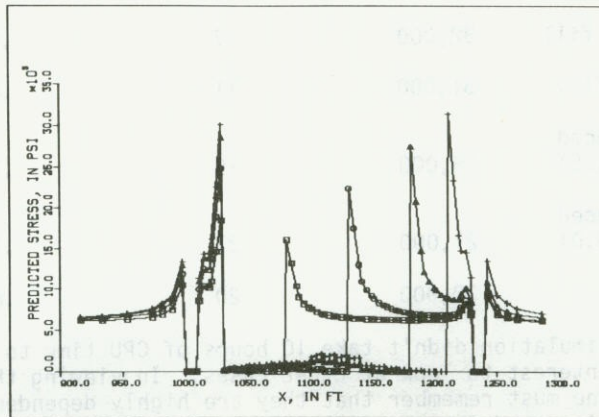


Fig. 10 - Comparison of stresses predicted at 4 levels of stope completion using mixed fill.

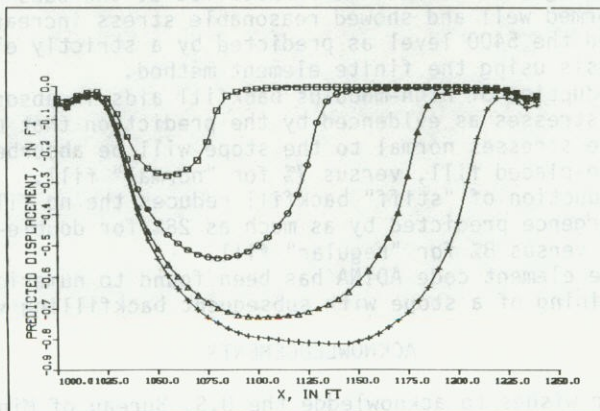


Fig. 11 - Comparison of displacements predicted at 4 levels of stope completion using mixed fill.

TABLE 3 - Summary of stresses and convergences predicted.

Type of Run	Maximum Abutment Pressure, psi	Percent Reloading of fill	Maximum Convergence, ft (one-half of total)
• no fill	34,500	0	1.0
• "regular" fill	32,000	7	.92
• vibrated fill	31,000	11	.87
• double placed fill ($\nu=.25$)	28,000	19	.78
• double placed fill ($\nu=0.0$)	27,000	27	.72
• mixed	30,000	20	.82

If each simulation didn't take 10 hours of CPU time to run, it would be of interest to look at other cases. In viewing the results of Table 3, one must remember that they are highly dependent on the input properties used for the fill, which are presented in Table 2 of this report.

CONCLUSIONS

One can arrive at the following initial conclusions:

- The vibrating wire stressmeters installed at the Sunshine Mine performed well and showed reasonable stress increase values around the 5400 level as predicted by a strictly elastic analysis using the finite element method.
- The introduction of high-modulus backfill aids in absorbing in-situ stresses as evidenced by the prediction that up to 27% of the stresses normal to the stope will be absorbed by double-placed fill, versus 7% for "normal" fill.
- The introduction of "stiff" backfill reduces the no fill maximum convergence predicted by as much as 28% for double-placed fill, versus 8% for "regular" fill.
- The finite element code ADINA has been found to numerically model the mining of a stope with subsequent backfilling very well.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the U.S. Bureau of Mines for support of this work, and in particular Robert Dorman, the Contract Technical Officer from the Spokane Mining Research Center, for his assistance and support. Also, without the help of D. Rankin, at the Sunshine Mine, and their personnel, the readings presented in this report would not be possible.

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GROUND CONTROL IN COAL MINING

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GROUND CONTROL IN COAL MINING

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ABSTRACT

The ground control problem in coal mining is discussed in terms of the various types of ground control problems that can occur in coal mining. The various types of ground control problems are discussed in terms of the various types of ground control problems that can occur in coal mining.

Chapter 60

STRESS CONTROL METHOD APPLIED TO STABILIZATION OF UNDERGROUND COAL MINE OPENINGS

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ABSTRACT

Serious floor heave of up to 2.4 m in a 2.4-m high mine entry was eliminated by applying the stress control method of mining, as a last resort, at the No. 5 coal mine of Jim Walter Resources, Inc., in the Black Warrior coal basin near Birmingham, Alabama. Underground observation of the first 3-room entry created using the stress control method is discussed here. The behavior of the test entry, which eliminated the heave problem, is analyzed in relation to similar studies conducted in a salt mine under similar ground conditions by utilizing finite element analysis.

INTRODUCTION

SIGNIFICANCE OF TRIAL APPLICATION OF STRESS CONTROL METHOD

Failures of roof and floor present serious rock mechanics problems in many underground coal mines. These problems have been dealt with mainly by using artificial supports, such as roof bolts, wooden cribs, and concrete linings. These conventional means of ground control become increasingly difficult to use as the ground instability intensifies. Such instability encountered at the No. 5 mine of Jim Walter Resources, Inc., operating in the Black Warrior coal basin in Alabama, was so intense that none of the conventional means of ground control seemed feasible.

The stress control method of mining was applied as a last resort. This method in fact proved to be successful in eliminating the intense ground failure problems without requiring any artificial means of ground control. The results of this trial demonstrate that the

stress control method is not only a powerful and scientific method but also that it is economical and practical for solving ground failure problems in underground mines.

The scientific significance of this development lies in the field verification of the rock mechanics principles of complex ground upon which the stress control method is based, demonstrating that the method is applicable to most underground mining, regardless of the type of ore body to be mined.

HISTORICAL BACKGROUND AND THEORETICAL ISSUES

The stress control method has a long history. The origin of the method may be traced back to coal mining in northern England in the early 1950s, when Charles Holland first introduced the concept of the stress arch and yield pillar (1). But his method did not receive wide acceptance at that time, despite his penetrating vision. In the meantime, the yield pillar was successfully adapted in underground trona mining in Greenville, Wyoming, through the creative work of Bill Fischer of FMC (2). Fischer utilized the yield pillar to sustain temporary stability of failing roof strata in his total extraction retreat mining. The use of yield pillars to provide long-term stability of an advancing new entry system was not developed until the late 1960s in potash, where the stress control method originated. Many potash mines at that time were suffering from extensive floor heave and roof failure problems. The conventional use of yield pillars and stress arches was found to be inadequate to solve these intensive ground failure problems. It was at this time that the stress control method was first introduced, successfully stabilizing the ground and eliminating the saturation roof bolting requirements which had been believed to be essential. In this new method, the yield pillars were used to control formation of a final protective stress envelope surrounding a group of openings separated only by thin yield pillars. The formation of the envelope was controlled by following a certain time sequence of creating the yield pillars (3).

Results of the experiments conducted on the stress control method at the JWR No. 5 mine are described below. Discussion here is focused upon the most basic underlying issues in rock mechanics principles of the stress control method--namely, constitutive modelling of complex grounds by finite element simulation of the stress control method.

MINE TESTING

MINE AND GROUND FAILURE

The JWR No. 5 mine is located in the Black Warrior coal basin about 20 miles southwest of Birmingham, Alabama. The mining seam is high quality anthracite up to 6.4 m thick at a depth of about 680 m. The lithological profile of the strata around the seam is shown in Fig. 1. The mine is in the process of successfully expanding its longwall mining. The room and pillar method provides a vital function

both for development entries of longwall mining and for conventional production. The room and pillar method uses 100 ft (30.5 m) square pillars with 20-ft (6.1-m) rooms in a checkerboard pattern.

The entries created by the room and pillar method were suffering floor heave in one section of the mine, to the extent that the floor touched the ceiling in some extreme cases, as shown in Fig. 2. The floor heave was due mainly to the buckling failure of the heavily stratified shale underlying the floor. Shale in the roof was found to be relatively competent, maintaining a stable roof. No conventional means of ground control was found to be useful for eliminating the floor heave. Under this situation, the stress control method was introduced as a last resort to solve the problem.

STRESS CONTROL DESIGN AND TEST AREA RESULTS

The first of 2 test entries, the 3-room entry, was cut by the stress control method, i.e., the outside rooms first, the center room second, as shown in Fig. 3. The 4-room entry cutting sequence was identical to that of the 3-room entry, except that the center protected room opening consists of 2 subrooms cut at the same time. Regardless of the number of rooms in one multiple-room entry, the stress control method requires that all the yield pillars be sufficiently small that final excavation of the center protected rooms results in immediate formation of the final intended stress envelope. This is accomplished by making all the yield pillars in one entry fail uniformly. Details of this process have been described previously (3).

The stress control method succeeded immediately upon its application, visibly eliminating the floor heave problems without any artificial means of ground control. This was so obvious that no rock mechanics measurement of any sort was needed to verify the successful result. This success is shown in the photograph of the protected room in the 3-room entry (Fig. 4). Here the floor heave of Fig. 2 has virtually vanished. The impact of the stress control method in the

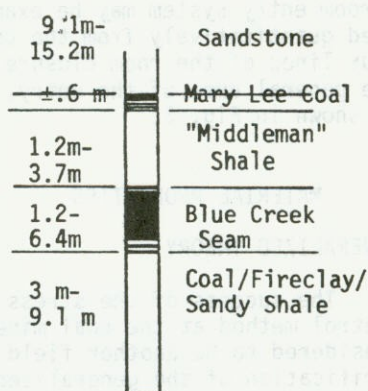


FIG. 1. Lithological Profile of Strata Around Coal Seam.



FIG. 2. Floor Heave in Conventional Room.

3-room entry system may be examined quantitatively from the contour lines of the room closure in the general area of the entry, as shown in Fig. 3.

MATERIAL PROPERTIES

GENERALIZED THEORY

The success of the stress control method at the coal mine is considered to be another field verification of the generalized theory of material properties upon which the stress control method is based. The theory uses the failing characteristics of geological materials as critical components of the entry design. The most important aspect of the theory is the constitutive modelling of material properties, which is especially developed for finite element simulation of the stress control method.

The properties of geological materials are considered basically non-elastic. The elastic part of the material properties constitutes only a minor portion of the generalized material behavior. The generalized theory places its emphasis on material properties of yielding, failure, and post-failure behavior in relation to time. This is accomplished by the development of the modular concept of the constitutive model of geological materials.

Local failures are utilized in achieving ultimate stability of the multiple-room entry system designed by the stress control method. Therefore, such failures are not only tolerated but quantitatively controlled in both intensity and extent. For example, yielding failure is induced controllably in the yield pillars

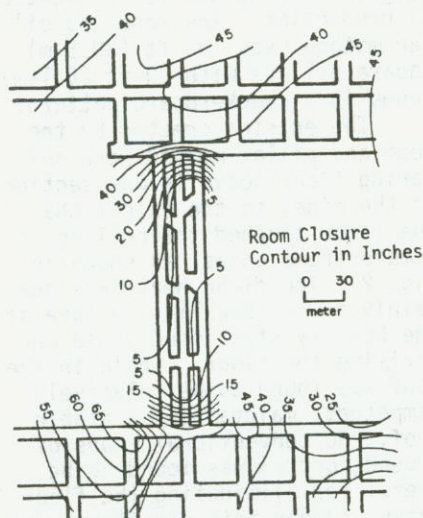


FIG. 3. Experimental Layout of 3-Room Entry in Coal Mine and Room Closure Contour Lines Demonstrating the Effectiveness of Floor Heave Elimination by Stress Control Method (Without Any Artificial Means of Ground Control).

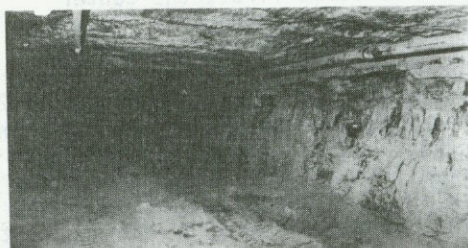


FIG. 4. Elimination of Floor Heave in 3-Room Entry.

inside the multiple-room entry. Without such local failure, the stress control method is not possible.

MODELLING OF GEOLOGICAL MATERIALS

The modular concept of material properties has provided a physical basis upon which the constitutive models of geological materials are built. This concept is a key to the practical application of the finite element method in complex grounds. The material model was constructed by assembling basic behavioral components in a certain modular system, closely representing the physical behavior of generalized geological materials. The model may be represented in a rheological form, but it is not limited to rheological properties. It is designed to apply to a broad spectrum of material properties ranging from soft/ductile to hard/brittle materials. Because of the modular concept, clay seams and fractured strata can be physically simulated by mathematical selection of the material property coefficient values to adequately represent complex grounds such as those in coal and potash mines.

The generalized constitutive equations based on the modular concept are found to be fundamentally different from those conventional constitutive equations which are generated by curve-fitting of experimental data. Such examples are creep laws and hyperbolic or non-linear stress-strain formulations. In contrast to these conventional approaches, the modular approach considers the ground behavior to be an integration of all the basic behavioral components of the material. The modular approach is found to be direct in mathematical representation and far superior in field application. Details of the modular approach were reported previously (4).

FINITE ELEMENT MODEL OF 3-ROOM ENTRY

COAL VS. SALT

The REM computer model of the 3-room entry, described below, was created for a salt mine. In fact, the coal mine entry was found to be very similar to that of the salt mine in fundamental geomechanical behavior and conditions. This may be attributed to the principle of generalized material properties on which the stress control method is based; that is, all materials fail and yield. Because of this similarity, the salt mine model may be used to evaluate the conditions of the coal mine.

REM MODEL

A computer simulation model of a 3-room entry in a bedded salt formation was constructed to analyze the stress control method. The model uses the REM computer program, which is based on the generalized theory of geological materials. The REM model is designed to simulate the entire time-dependent process of room excavation, yield pillar failure, and stress envelope formation by the stress control method.

The finite element mesh of the 3-room entry of the mine and the material properties of the ground were previously reported (4). The two outside rooms are excavated first, then one month later, in the simulation, the center protected room is excavated. Using the results of the REM simulation, the stress control method is analyzed as follows:

GLOBAL STRESS AND STRAIN DISTRIBUTION

The stress and strain distribution patterns around the 3-room entry system after 10 years of creep deformation are shown in Fig. 5. Here the stress distribution pattern illustrates how the protective stress envelope is formed around the entire 3-room entry. The major thrust of the stress envelope goes into the hard shale formation, leaving the immediate salt roof formation well stress-relieved. The center room is highly stabilized, as is evidenced by the small amount of stress remaining in the roof and floor. It is the stress relief effect which makes the roof and floor stable without any artificial means of support.

However, the corresponding strain distribution pattern shows the damage created by the stress control method. The outside room suffers a large accumulation of strain, especially around the upper outside corner and in the overlying clay seams. In contrast, the center room is virtually free from such creep strain accumulation, even after 10 years of creep deformation. The area of the maximum damage is found in the first clay seam above the upper outward corner of the entry, indicated by the concentrated strain exceeding

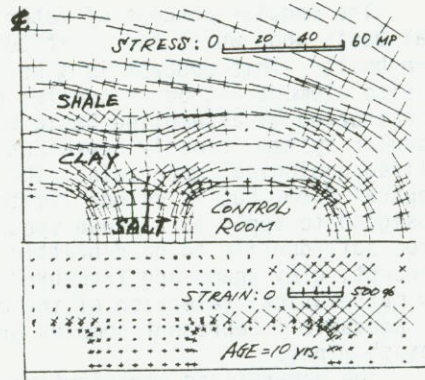


FIG. 5. Global Distribution of Principal Stresses and Strains Around 3-Room Entry in Salt Bed Overlain by Shale and Clay Seams, Illustrating Mechanism of Stability Achieved by Stress Control Method of Mining.

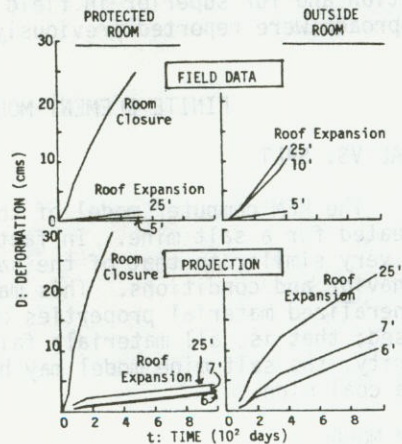


FIG. 6. Comparison of Computer Model Simulation and Field Measurement in Creep Deformation of Inside (Protected) and Outside (Control) Rooms of the 3-Room Entry in the Salt Bed.

several hundred percent. This agrees with observations in the mine.

CREEP DEFORMATION

The long-term closure of the protected room simulated by the REM model is compared with the actual 3-room test entry in the mine in Fig. 6. The computer simulation is found to be in general agreement with the basic nature of the mine room closure, although the simulation is found to be a little higher than measured. The creep expansion of the immediate roof strata over both protected and outside rooms are also plotted in the figure. Here the model compares very well with measurements in both rooms, the protected (inside) and the outside. The difference between the inside and outside rooms is well-simulated. The comparison shows that the large difference is coming not from the immediate roof, but rather from the creep deformation of the salt pillars and abutments. It is important to note that the expansion of the first 25 ft of the roof formation over the protected room is less than 3 cm in the first 500-day period in both simulation and measurement, while the room closure during the same period exceeds 20 cm in both simulation and measurement. The good match of the computer simulation may be considered as a field and computer verification of the formation of the protective stress envelope by the stress control method.

DISCUSSION

4-ROOM ENTRY

This presentation is limited to the REM computer model analysis of the 3-room entry. Actually, a 4-room entry was also tested in the coal mine, very successfully, further demonstrating the effectiveness of the stress control method in the coal mine. Because of the close resemblance of ground behaviors between salt and coal, a wide range of design methods already developed in salt and potash mines to meet their diverse ground conditions may be directly applicable to solve similar problems in coal mines. This remains to be investigated.

PRELIMINARY MODEL OF UNKNOWN GROUND

Ground conditions are often unknown when a first computer model analysis of mine entries is needed. It has been shown through experience that most material properties may be assumed adequately for a preliminary simulation analysis. The reasons are not certain, but it is probable that failure behavior, which dictates the stress control method of design, does not require high precision in material property coefficient values, as long as the fundamental failure mechanism is recognized. This further indicates that there is significant room for future refinement of the REM model of coal mine openings for further optimization of safe and economic mining. The preliminary model thus will be used as a basis for further improvement as actual field data

accumulates.

FIELD INSTRUMENTATION SYSTEM

A computerized instrumentation system has been developed for measuring the three fundamental design parameters--stress state, material properties, and deformation rate--in situ. The system utilizes a field data acquisition system which operates three different probes for respective measurement of the three parameters. The system provides collection, processing and analysis of data for evaluation on-site (5). The stressmeter probe is used for in situ validation of the stress control method by directly observing initiation, transformation, and settlement of stress envelopes created around the mine entries. The propertymeter probe is applied to determine in situ material properties of complex ground, such as weak seams, soft rock, and fractured ground, where core specimens are not usually available, to obtain critical data for the computer model. The creep rate probe is applied to determine the dynamic state of microcreep deformation of the ground to monitor ground stability.

CONCLUSIONS

1. The stress control method of mining is highly effective in controlling ground instability in underground coal mines.
2. The use of the stress control method in coal mines can improve safety and extraction ratio and reduce mining costs.
3. The finite element method, using the REM code, discloses the fundamental mechanism of the stress control method using failure characteristics of geological materials.
4. In situ measurements of stress state, material properties and creep rates are useful for quantitative control in application of the stress control method.

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Chapter 61

OPTIMUM MINING PLAN FOR MULTIPLE SEAM MINING

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ABSTRACT

Multiple seam mining and its associated problems are very serious in Southern West Virginia where poor planning or lack of knowledge in seam interaction often results in complete loss of coal properties. Several measures have been proposed to alleviate the interaction problems under various multiple seam conditions, but few of them are specific in terms of mining plan. Two parallel approaches are adopted for this paper. One is field investigation which consists mainly of mine site visits and gathering of data. The other is the two-dimensional finite element simulation of mine workings using the data gathered as input. Two typical cases which represent the past and present multiple seam mining practice in West Virginia were investigated. Much attention is focused on the interaction problems experienced and the important parameters controlling interaction. Several key parameters controlling interaction in multiple seam mining were analyzed. Proper mining plans, by which the interaction problems can be minimized, are proposed for the individual cases after a better understanding of the interaction mechanism was realized through finite element analyses.

INTRODUCTION

Almost all coal seams in West Virginia are multiple seam, although in some areas it is not so obvious due to nonmining in the adjacent seams or the seam separation is large enough to be ignored. It is, however, becoming increasingly common to mine coal in an area where the working face will be affected by a previously mined-out area or a presently active mine through interaction effects. The adverse ground control problems associated with these interaction effects have increased the risk of mining coal and have attracted more attention

(Scurfield, 1970; Whittaker and Hodgkinson, 1971; Dunham and Stace, 1978; Szwilski, 1979; Peng and Chandra, 1980; Stansbury, 1981; Haycocks, et al., 1982; and Pariseau, 1983).

During the past decades, the seams selected for mining have been ones of easy accessibility with best physical conditions available and suitability for market desires, i.e. coal with low sulfur and ash, and high heating value. Selection of mining sequence is therefore seldom based on ground control considerations. Problems associated with multiple seam mining are further enhanced by poor planning and the lack of knowledge in seam interaction. As a result, a complete loss of coal properties often occurs. It has been pointed out (Peng and Chandra) that the seam interaction can be avoided, minimized, or on occasion utilized beneficially if careful mining plans are carried out. The development of an optimum mining plan for multiple seam mining should be based on a better understanding of the interaction mechanism, and all possible mining sequences and methods must be considered. Once the optimum mining plans for various multiple seam conditions are developed and made known to the coal operators, it will reduce many unnecessary hardships currently facing multiple-seam coal mining operators thereby greatly reducing the mining cost and increasing coal recovery. Safety and profitability of multiple seam mining will therefore be improved tremendously.

This paper presents the results of a recent investigation on two cases of multiple seam extraction and the study of the interaction mechanism. We hope that the results will lead to a better understanding of the interaction mechanism in multiple seam extraction, from which modified mine layout can be derived to substantially improve face and entry stability.

SIMULATION OF MULTIPLE SEAM MINE WORKINGS

The objective of this section is to identify possible causes of the ground control problems due to the interaction of multiple-seam mining by simulating the mine workings utilizing finite element analysis. The problem areas defined were then compared with those gathered underground to establish the validity of the finite element analysis. Improved mining plans are subsequently proposed to alleviate the interaction effects. Two typical cases which represent the past and present multiple seam mining practices in West Virginia were chosen for the study.

Case History

Case No. 1. The mine has panels in both Cedar Grove (1.37 m thick) and No. 2 Gas (1.27 m thick) seams with the former as the upper seam. The stratigraphic sequence is shown in Fig. 1a. Both seams are being mined simultaneously with development and pillaring operation being kept in advance in the upper seam. Square pillars, 15.2 m wide and entries, 6.1 m wide are kept superimposed in both seams. A panel in

the lower seam was extracted under the gob area of the upper seam (Fig. 1a and 1b). Serious roof control problem and water seeping down from the upper seam through the cracks in the strata between the seams were first reported when the pillar line in panel 2B approached vertically below the bleeder pillars in panel 2A. Even worse roof conditions and more serious water problem occurred when the pillaring operation approached the area below the sump area left behind in the middle of the gob of panel 2A in the upper seam.

Case No. 2. Two main seams, in descending order, No. 2 Gas (1.7 m thick) and Eagle seam (2.5 m thick) have been mined. The stratigraphic sequence for this mine is shown in Fig. 1c. The upper seam was extracted first. A five-entry mains was developed in 1971. About four years later the Eagle seam was mined. When the pillar extraction started in the Eagle seam below the main entries in the No. 2 Gas seam, heavy roof falls and floor heaving in the entries began to take place. Many portions of the entries and crosscuts were closed completely because of roof caving.

Finite Element Models

Two-dimensional finite element analysis using the NASTRAN program (McCormick, 1983) were performed for Cases No. 1 and No. 2. The mine models used in the analysis for both cases along cross-section AA (Fig. 1) are shown in Fig. 2. The stratigraphic sequences used are shown in Fig. 1. The innerburden thickness and the overburden depth for the upper seam are 23.8 m and 82.8 m for Case No. 1, respectively; 50.6 m and 163.8 m for Case No. 2, respectively. The strata in both cases was assumed to be isotropic, homogeneous and perfectly elastic. All faces of the model, except the top, were rollered. Isoparametric quadrilateral elements with double precision were used in the models. The element sizes used in the areas of interest were generally from 1.22 m x 1.52 m to 3.05 m x 1.52 m for Case No. 1 and 0.91 m x 1.22 m to 1.52 m x 1.52 m for Case No. 2. The total number of nodal points and elements were 4100 and 3912 for Case No. 1, 3774 and 3650 for Case No. 2. The mechanical properties of the strata used for both cases are listed in Table 1. Coulomb criterion was used to evaluate the yielding conditions and to determine the extent of caving. The uniaxial compressive strength (C_0), uniaxial tensile strength (T_0) and triaxial stress factor (q) for different rock types are listed in Table 2. One-fourth of the strength value in Table 2 was adopted in the finite element analysis due to the scale factor. In other words, the recommended safety factor is 4 for compression and tension. Once the element yielded, its Young's modulus was reduced to one-tenth of its original value. The Young's moduli of gob material range from 1/100 to 1/57 of that of intact rock (Price, 1929).

Results and Discussions

Case No. 1. The pillaring operation was carried out from left to right for Case No. 1 as shown in Fig. 2a. The face convergences and roof conditions above the face areas were calculated and evaluated

when the face of the lower seam was in the locations P1, P2, and P3 (Fig. 2a). The face was vertically below the bleeder pillars of the upper seam when it was at location P1; under the gob area when it was at location P2; and below the sump area when it was at location P3.

Figure 3 shows the caving and yield zones induced by the mining of both seams. When the face was at location P1, caving and yield zone A1 was developed. It clearly shows that this yield zone reached the upper seam. Since a sump area was constructed in the middle panel of the upper seam during the extraction, water will certainly drain down through the yield zone and cause problems at the face area of the lower seam. This confirms the problem observed underground. The water problem at this point, however, will not be as serious since the face is still far away from the sump above. When the face advances to location P2, yield zone A2 was generated. The yield zone was not high enough to connect the yield zone created by upper seam mining and, therefore, no water problem will be encountered which agrees with field observation. Severe yielding condition in the roof occurs when the face approaches the location P3. Yield zone, A3, extends upward and joins the yield zone, A4, induced by the upper seam mining. This triggers the severe water problem at the face area. Water problem is further aggravated by the fact that the face is below the sump area.

Figure 4 shows the face convergences at locations P1, P2, and P3. The convergence has its highest value (curve C) when the face approaches below the sump area and has its lowest value (curve B) when the face is under the gob area of the upper seam. Since bad roof conditions are commonly associated with high face convergence, the former case will experience more ground control problems than the latter case.

Comparisons of field observations with the results of finite element analysis strongly indicate that the simulation closely identify the ground control problems associated with the present mining plan and, to some extent, predicts the severity of the problems.

Case No. 2. The major problem for Case No. 2 is that the stability of the main entries in the No. 2 Gas seam is jeopardized by the mining of the Eagle seam below. The problem can be examined through the tensile stress distribution in the immediate roof of the main entries (Fig. 5). A sharp increase in tensile stress in the roof of entry 2 and 3 was experienced, while negligible increase in entry 1 was observed. Entry 5, which is located directly above the center of the gob, experienced a sharp decrease in tensile stress. The most critical roof condition occurs in entry 2 which is about 28° measured from the vertical line extended over the gob edge while a better roof condition is achieved in entry 5. The effect of undermining will diminish at a location 42° measured from the vertical line extended over the gob edge.

Improved Mining Plan. It is obviously a mistake for Case No. 2 to undermine a main entry system. The best alternative is to develop a

main entry system in the lower seam superimposed with that in the upper seam. The system in the lower seam, however, will still have certain effect on the stability of the entries in the upper seam. This will be discussed further in a latter section.

To improve the mining condition of the lower seam in Case No. 1, one needs to prevent the connection of the yield zones induced by the mining of both seams so as to alleviate the water problem. A revised mining plan for the lower seam is proposed as shown in Fig. 6. The plan suggests that a remnant pillar (F) about 6.1 m wide at the left side of location P1 should be left and only partial extraction for the pillars between P2 and P3 is recommended. According to the improved plan, the total extraction ratio will drop from the original 82% to 69%. However, the working condition will be greatly improved.

EFFECT OF UNDERMINING

The mining sequence adopted in Case No. 1 is that both seams have been mined simultaneously with pillaring operation being kept in advance in the upper seam. Although the working condition in the upper seam will not be affected by the undermining according to the arrangement, the coal ribs (or barrier pillar) and the underlying and overlying strata on both sides of the gob will be affected. One typical model studied (Fig. 7) indicates that a yield zone induced in the upper seam can sometimes be extended out as far as one panel width from the gob edge due to the mining of lower seam depending on its overburden depth, innerburden thickness, strength of innerburden, and panel width. Adverse roof condition will be expected when development and extraction in this area are proceeded or the stability of the barrier pillars will be seriously affected. The width of yield zone will increase with increasing innerburden thickness at a more or less constant influence angle α (Fig. 8). Basically, the influence angle decreases with decreasing overburden depth (curve a) and panel width (curve b) (Fig. 9). The influence angle will remain constant until the innerburden reaches a certain thickness and starts to decrease as the thickness of innerburden increases further (Fig. 10) and will finally become zero as the innerburden thickness reaches a critical thickness. Beyond that, the yield zone in the upper seam disappears. The critical thickness increases as the overburden depth and panel width increase.

EFFECT OF A SUPERIMPOSED MAIN ENTRY SYSTEM IN THE LOWER SEAM ON THE MAIN ENTRY STABILITY IN THE UPPER SEAM

Three-dimension models simulating a seven-entry mains were used in this study. Since the entry length is much longer than its width, a model cut across the entries and containing a half pillar and a half crosscut is adopted due to symmetry (Fig. 11). The coal seam thickness used for both seams is 1.83 m, while the entry and crosscut are 6.1 m wide. The ratio of overburden depth to pillar width was kept at

10 for all models. Square pillars were used in the models.

Figure 12 shows the effect of innerburden thickness on the entry stability in the upper seam. The stability of the outermost entry (solid line) and of the middle entry (dash line) was evaluated. Safety factors for both entry roofs decrease as the innerburden becomes thinner, i.e., the entries are less stable. The influence on the outermost entry, however, diminishes as the overburden increases.

LONGWALL LAYOUT TO MINIMIZE INTERACTION EFFECTS

In Southern West Virginia, multiple-seam mining is a rule rather than exception. Once a room-and-pillar or longwall system has been adopted it will affect the workings of all seams in the area. If a longwall system is adopted for one seam, two major ground control problems are frequently encountered when the overlying or the underlying seam is mined subsequently. Firstly, the best relative arrangement of chain pillars must be determined as it affects strongly the entry stability for the seam to be mined. Secondly, if a large barrier pillar or a solid rib was left in the preceeding seam what effect of the angle at which the rib edge of pillar is approached will have on the working face and panel entries in the succeeding seam. To answer these questions, we postulated several possible longwall layouts in the analysis. The stratigraphic sequences used are obtained from mine sites located in southern West Virginia.

Effect of Relative Arrangement of Chain Pillars

Six possible longwall layouts are postulated in the analysis (Fig. 13). Layouts (a), (b), and (c) represent the possible layouts when the lower seam is mined first, while layouts (d), (e), and (f) represent the possible layouts when the upper seam is mined first. The chain pillars used in cases (a), (c), (d), and (f) were 16.5 m wide, while those used in cases (b) and (e) were 33 m wide. Therefore, only three panel entries are present in cases (b) and (e). A model containing a half pillar and a half crosscut was cut across the panel. This three-dimensional finite element model was considered adequate due to the symmetric conditions. Linear isoparametric hexahedron elements were used in the model which contains 2640 grid points. Figure 14 shows a typical three-dimensional finite element mesh layout and the stratigraphic sequence used in the analysis. The physical and mechanical properties of rocks are listed in Tables 1 and 2. Table 3 shows the convergences as calculated at the intersections and points midway between two intersections for entries designated E1 and E2.

It clearly suggests that columnization of chain pillars will generally produce the most adverse conditions in the panel entries whether mining is proceeded in ascending or descending order. Widening of chain pillars (b & e) in the succeeding seam would not bring about significant improvement in the panel entries. However, by locating the entries under the gob areas of the preceeding seams (e & f) the entry stability of the succeeding seams are improved tremendously as

evidenced by the sharp decline in entry convergence in the respective cases.

Effect of the Angle at which the Rib Edge of Pillar is Approached

In this analysis, the upper seam was assumed to be mined out previously with a large barrier pillar left in place. The face of the lower seam was then assumed to approach the rib edge of the pillar at angles of 30°, 45°, 60°, and 90°. Three-dimensional finite element models similar to those used above were used to simulate the individual cases. Linear isoparametric hexahedron elements were used in the models which contain 2890 grid points. The stratigraphic sequence used was similar to that shown in Figure 14 except that the innerburden thickness is 25.6 m rather than 58 m. The material properties listed in Tables 1 and 2 were used in the analysis. Results of the analysis have shown that the average vertical stress across the face line as calculated are 23.37 MPa for 30°, 25.25 MPa for 45°, 28.19 MPa for 60°, and 29.83 MPa for 90°, respectively. It is apparent from these results that roof support problems at the face will generally be most severe when a rib edge of barrier pillar is approached at right angle. To examine the effect of the angle at which the rib edge of pillar is approached will have on the stability of the entries in the lower seam, two single entries were simulated in the lower seam as the only excavations. Figure 15 shows the calculated convergences along one of the two entries for different approaching angles. It shows that the angle at which the rib edge of pillar was approached does not affect significantly the magnitude of the interaction effect, although the 30° case will generally have less stable entry compared to the 90° case.

CONCLUSIONS

1. The ground control problems associated with multiple seam mining can be generally identified by finite element analysis and improved mining plans can therefore be proposed accordingly.
2. The arrangement of mining the seams with extraction in the upper seam preceding the lower seam might seriously affect the stability of barrier pillars left in the upper seam or cause substantial loss of coal reserve in the upper seam, depending on the overburden depth, innerburden thickness, and panel width.
3. The stability of a main entry system in the upper seam may deteriorate due to development of another main entry system underneath depending on the innerburden thickness.
4. Columnization of longwall panel entry chain pillars was found to produce the most adverse conditions in the panel entries whether mining is proceeded in ascending or descending order.
5. Widening of chain pillars in the succeeding seam would not bring about significant improvement in the panel entries.
6. Positioning the entries under the gob of the preceding seam would bring about significant improvement in the panel entries.
7. The angle at which the rib edge of pillar is approached was found

to have significant effect on the face below and minor influence on the panel entries below. Roof support problems at the face will generally be most severe when a rib edge of barrier pillar is approached at right angle. On the other hand, roof support problems in the panel entries will generally be most severe when the angle between the entry and the influencing rib edge is less.

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Table 1. Physical and Mechanical Properties of Coal and Coal Measure Rock

Material	Young's Modulus (MPa)	Poisson's Ratio	Density ³ MN/m
Gray Shale	2393	0.25	0.025
Sandy Shale	7662	0.3	0.025
Sandstone	9586	0.17	0.025
Coal	1379	0.3	0.014

Table 2. Laboratory Values of Uniaxial Compressive Strength, Uniaxial Tensile Strength and Triaxial Stress Factor for Each Type of Rock

Rock Type	C _o (MPa)	T _o (MPa)	q
Gray Shale	29.12	3	3.3
Sandy Shale	33.91	5.74	3.9
Sandstone	32.19	4.02	4.4
Coal	13.6	0.82	3.0

Table 3. Calculated Convergences at the Intersections and Points Midway Between Two Intersections

Cases	Convergence (cm)			
	Midway Between Intersections		Intersections	
	E1	E2	E1	E2
a	2.35	2.87	2.52	3.24
b	2.12	2.85	2.27	3.22
c	0.55	0.52	1.20	0.91
d	2.64	3.13	2.78	3.45
e	2.43	3.09	2.55	3.40
f	0.74	0.66	1.55	1.14

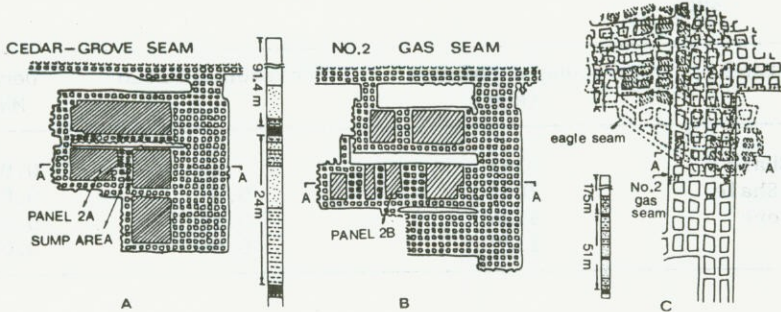


Fig. 1. Sectional layouts. A. Workings in the upper (Cedar Grove) seam for Case No. 1. B. Workings in the lower (No. 2 Gas) seam for Case No. 1. C. Dotted lines in the lower (Eagle) seam and solid lines in the upper (No. 2 Gas) seam for Case No. 2.

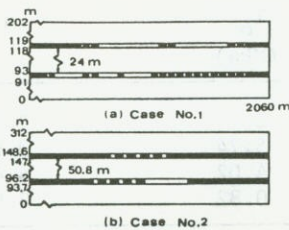


Fig. 2. Two dimensional finite element models.

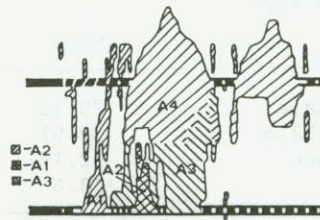


Fig. 3. Caving and yield zones induced by the mining of both seams.

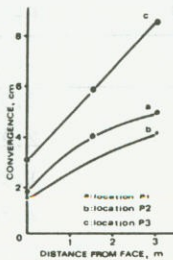


Fig. 4. Face convergence at locations P1, P2, and P3 of lower seam.

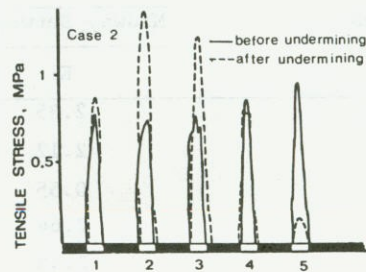


Fig. 5. Effect of undermining on the tensile stress distribution in the entry roof of the No. 2 Gas (upper) seam.

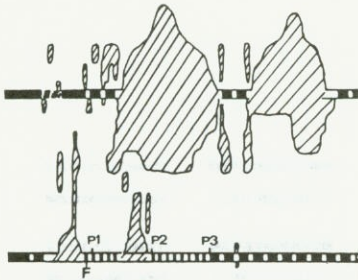


Fig. 6. Improved mining plan for the lower seam of Case No. 1.

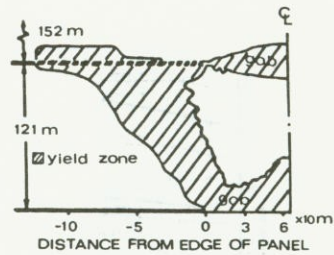


Fig. 7. Yield zone induced in the upper seam due to the undermining.

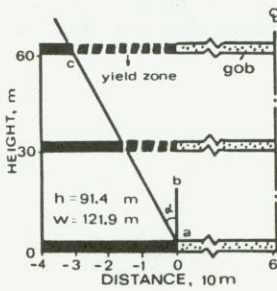


Fig. 8. Effect of innerburden thickness on the yield zone in the upper seam.

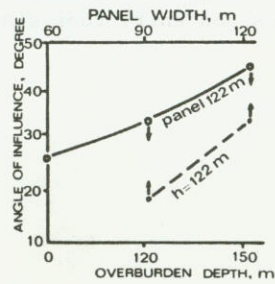


Fig. 9. Effect of overburden depth and panel width on the influence angle.

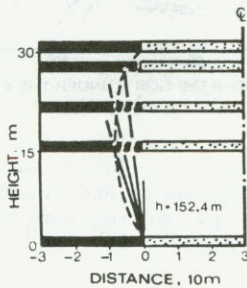


Fig. 10. Effect of innerburden thickness on the influence angle.

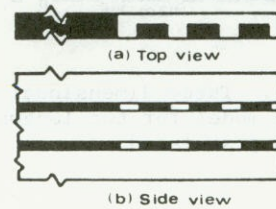


Fig. 11. Three dimensional finite element model for main entry system.

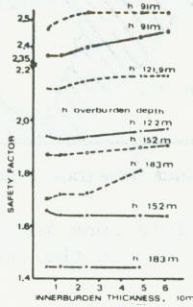


Fig. 12. Effect of innerburden thickness on entry roof stability.

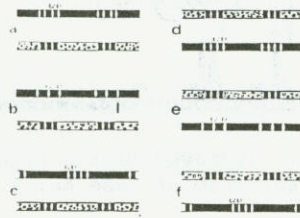


Fig. 13. Possible longwall layouts postulated for the analysis.

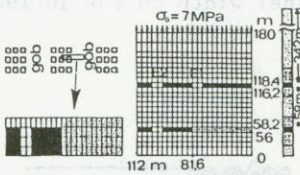


Fig. 14. Three dimensional finite element model for the longwall layouts postulated.

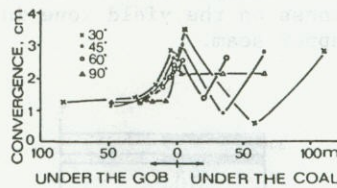


Fig. 15. Calculated convergences along the entry underneath the rib edge of pillar for approaching angles of 30°, 45°, 60°, and 90°.

Chapter 62

AN INVESTIGATION OF THE CAUSES OF CUTTER ROOF FAILURE IN A CENTRAL PENNSYLVANIA COAL MINE: A CASE STUDY

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ABSTRACT

Cutter roof failure is a specific type of ground control problem which frequently results in massive roof failure. It is a common occurrence in coal mines of the Northern Appalachian Coal Basin, causing delays in production and posing a safety hazard to mine personnel. The Bureau of Mines is conducting research on the causes of cutter roof failure to gain a basis from which to prevent its occurrence and to support such roof when failure does occur.

Research conducted in a coal mine of central Pennsylvania has revealed a correlation between the occurrence of clastic dikes and formation of cutter roof failure. In-mine mapping of ground conditions showed an increase in roof failure in areas of high frequencies of clastic dikes. Rock pressure monitoring around clastic dikes registered the greatest amount of roof loading near the intersection of dikes with the rib. Load cells measuring horizontal pressure changes in the roof indicated that the greatest pressure changes were occurring perpendicular to entry headings when clastic dikes were present. Analysis of rock pressure monitoring shows that the roof behaved as two cantilever beams when severed by a clastic dike. Additional roof supports such as trusses and cribbing were found to effectively support the roof in areas of clastic dikes and prevent cutter roof failure. However, these methods were only successful when employed shortly after mining.

INTRODUCTION

Occurrences of cutter roof failure during development mining have been observed in increasing numbers. In response to this, the Bureau of Mines is investigating the causes of this type of

roof failure, which can deteriorate to a major roof fall if additional support is not applied. The uniqueness of this problem has limited the techniques available for its treatment. The approach of trial and error treatment has proven unsatisfactory in many cases, as the causes are often different from mine to mine.

Cutter roof failure initially begins as a fracture along one or both roof-rib lines of an entry and propagates nearly vertically into the roof (fig. 1). When the fracture breaks to a height above the anchor horizon, or along a weak bedding plane, massive roof failure may occur. Figure 2 is an example of severe cutter failure and figure 3 illustrates the end result of such cutter development.

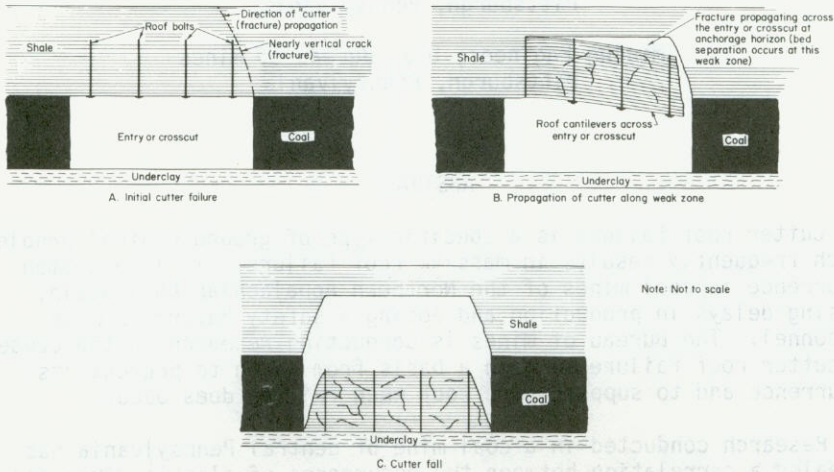


Fig. 1. Cutter roof failure diagram (modified from Kripakov, 1982; Nichols, 1978; Stefanko, 1983).

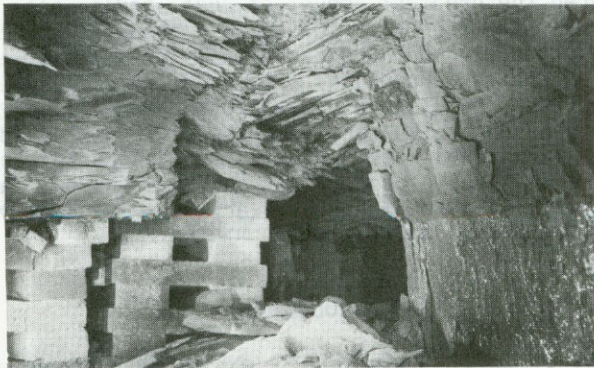


Fig. 2. Close view of cutter roof failure along rib. Crib blocks are approx. 0.1 m (4 in) thick.

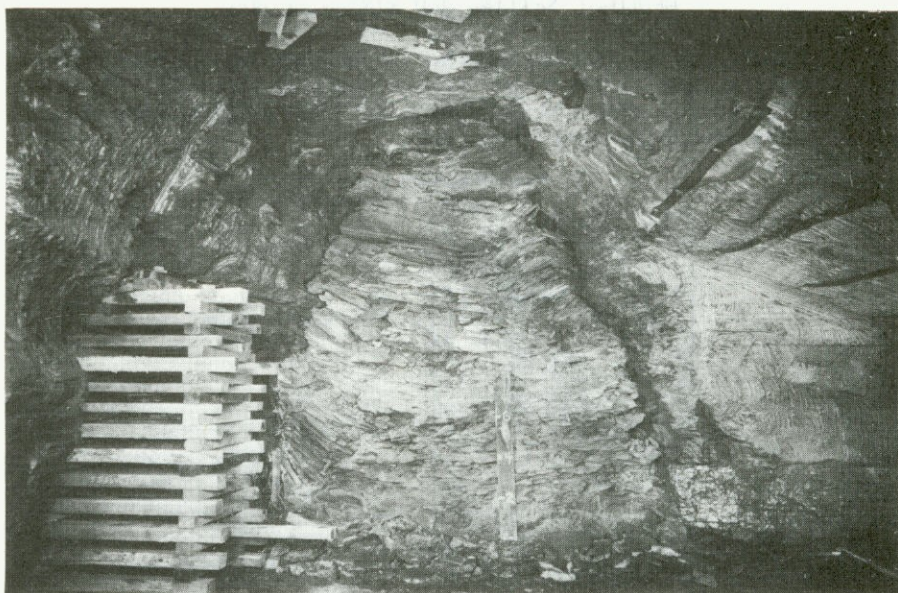


Fig. 3. Roof fall as result of cutter roof failure in study mine. Numbers on board represent feet above floor rock.

Previous research by the Bureau has shown a correlation between cutter roof failure and high horizontal stress fields. Aggson (1979) and Kripakov (1982) conducted studies addressing the particular stress state that would initiate cutter roof using finite element analysis. Research conducted by Thomas (1950) approached cutter development from a purely practical perspective, without any instrumentation. His work is believed to be the earliest attempt to understand cutter development. The influences of the direction of mining and occurrence of clastic dikes on cutter roof failure formation have been observed through detailed mapping by Iannacchione, et.al., (1984).

The occurrence of cutter roof has had a major effect on entry stability at the Greenwich Collieries North Mine of Indiana County, Pennsylvania. The investigation conducted there was comprised of two basic phases which contribute to an understanding of the pressure dynamics surrounding cutter development and the influence of geologic anomalies. The first phase consisted of in-mine mapping of all deformational and geologic features in the area studied. The second phase included the instrumentation of roof and pillars to determine pressure changes near and away from geologic anomalies as mining advanced. Improving entry stability through timing and placement of supports and design of mine configuration was found to effectively reduce cutter development.

GEOLOGIC SETTING AND MINING METHOD

The study area is located within the Appalachian Plateau Province along the axis of the Brush Valley Syncline (fig. 4). Structural relief on the mine property does not exceed 152 m (500 ft), and dips are generally less than 2.5° .

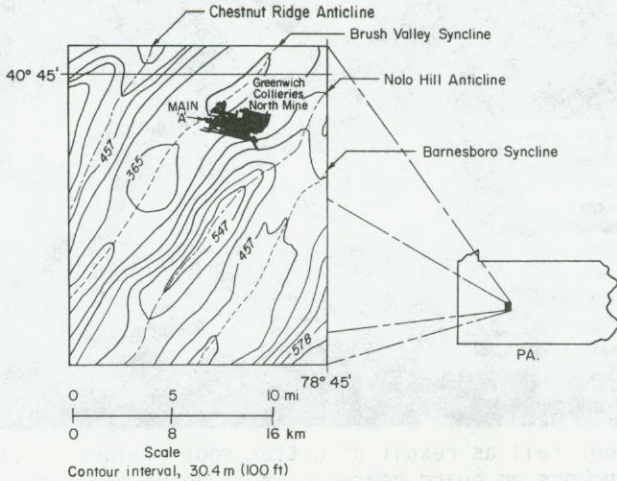


Fig. 4. Structure location map of Greenwich Collieries North Mine. Contours on base of Lower Freeport Coalbed.

The coalbed being mined is the Lower Freeport which is stratigraphically within the Pennsylvania age coal bearing rock of the Allegheny Group. The Lower Freeport Coalbed has an average thickness of 0.9 m (3 ft) and is directly overlain generally by 3 to 4.8 m (10 to 16 ft) of a competent dark gray shale (fig. 5). Above

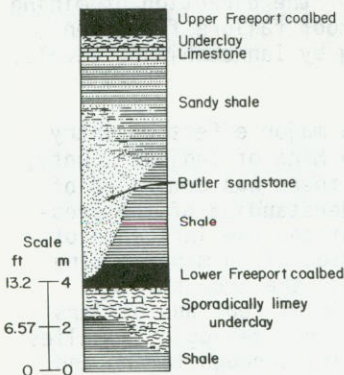


Fig. 5. Localized stratigraphic column.

the shale bed is a sandy shale interbedded with shale and sandstone; this unit ranges from 3 to 4 m (10 to 13 ft) in thickness. Intermittently the channel phase Butler Sandstone replaces the shale and sandy shale units. The next stratigraphically higher coalbed is the Upper Freeport with an average interburden of 9 m (30 ft). The maximum overburden above the Lower Freeport Coalbed is 200 m (655 ft). Immediately below the Lower Freeport is a sporadically limey underclay ranging from 1 to 3 m (3 to 10 ft); this, in turn, is underlain by shale.

In the area of the mine being studied coal is being extracted through the use of a drum-type continuous miner. The section employs a system of five to nine entries driven on 18.2 m (60 ft) centers. Entry and crosscut widths are approximately 5.5 m (18 ft) with crosscuts normally driven on 24.3 m (80 ft) centers.

CUTTER ROOF INVESTIGATION

Although cutter roof failure is a mine wide occurrence at the Greenwich Collieries North Mine, unusually severe cutter conditions were being experienced near the face of the Main A heading (fig. 4). This heading will ultimately be used to reach the remaining reserves available; thus, it is important to maintain entry stability for the life of the mine.

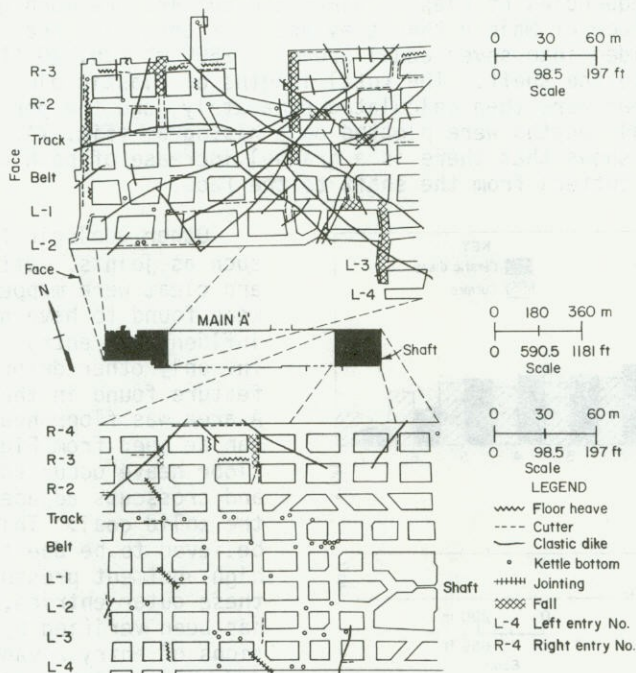


Fig. 6. Geologic structure and deformation map of Face and Shaft areas showing difference in clastic dike and cutter frequencies.

Assessment of Ground Conditions

Detailed mapping of geologic features and deformation due to pressure release phenomena was conducted in the Main A heading from the shaft to the face. Figure 6 is an example of the mapping and

illustrates the relation of geologic features to mining-induced deformation. Clastic dikes (commonly known in mining terms as clay veins) ranged in thickness from 30 cm (1 ft) of claystone matrix with fragments of shale and coal to as thin as a filmlike trace of calcite or clay. The dikes were observed (in roof falls) to extend as high as 8 m (26 ft) into the roof rock and would penetrate the coalbed at various angles. Few dikes reached the floor rock. Fractures and slickensides in the roof were associated with the presence of clastic dikes and slickensided faces were often marked with horizontal striations. Cutters normally formed adjacent to clastic dikes, with their deepest penetration (often greater than 60 cm (2 ft)) into the roof directly next to a dike and lesser penetration as distance from the dike increased. When cutter roof failure deteriorated enough to form roof falls, clastic dikes often formed their boundaries (fig. 6). It can also be seen from Figure 6 that frequencies of clastic dikes and cutters are much greater near the face of Main A than they are near the shaft area. Main A was divided into seven equal areas of 2500 m² (26,880 ft²) from the face to the shaft. The total lengths of clastic dikes and cutters in each area were then calculated separately, and the percents of their total lengths were plotted on a histogram (fig. 7). This histogram shows that there is a gradual increase of both clastic dikes and cutters from the shaft to the face.

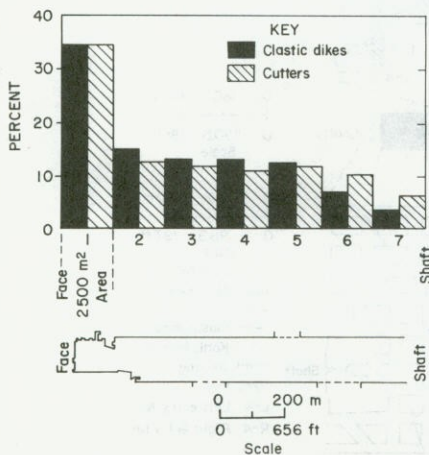


Fig. 7. Percents of total lengths of clastic dikes and cutters in equal areas of 2,500 m².

Other geologic features such as joints, kettlebottoms, and cleat were mapped but were found to have no major influence on entry stability. The only other deformational feature found in the Main A area was floor heave. As can be seen from Figure 6, floor heave occurred in entries and crosscuts adjacent to the solid coal. This is believed to be due to rather high abutment pressures in these outer entries, which has been verified by observations of entry advancement and will be investigated further through in situ stress measurements.

Rock Pressure Monitoring

Observations made through mapping revealed that cutters mainly formed adjacent to clastic dikes shortly after mining. This sug-

gested that the disruption of roof integrity, due to the presence of clastic dikes, was contributing to the propagation of cutters. The natural beam of the roof rock, spanning from pillar to pillar, is severed when a clastic dike is present, and this causes the roof to behave as two cantilever beams.

In an attempt to measure and quantify the cantilever beam effect, hydraulic diaphragms (load cells) were placed in the roof strata to measure pressure changes parallel and perpendicular to the entries as mining advanced. An individual load cell measures pressure changes in only one direction; thus two cells were used to measure mutually perpendicular horizontal pressure changes. The cells were set in the roof strata to a height of 1 m (3 ft) from the mine roof surface, which placed them approximately halfway between the anchor horizon and mine roof surface. The load cells had an initial setting pressure of approximately 1800 kPag (260 psig) to ensure a solid contact with the strata. The pressure was read as gage pressure because barometric pressure was not considered to cause significant change.

Later, hydraulic U-cells were placed on bolts in monitored areas along with the load cells. The U-cells measure bolt loading and were installed on the regular mining cycle as bolts were installed. The bolts being used in Main A are of the point anchor resin, tension rebar type. Since the U-cells were installed as the bolts were being installed, their setting pressure was determined by the amount

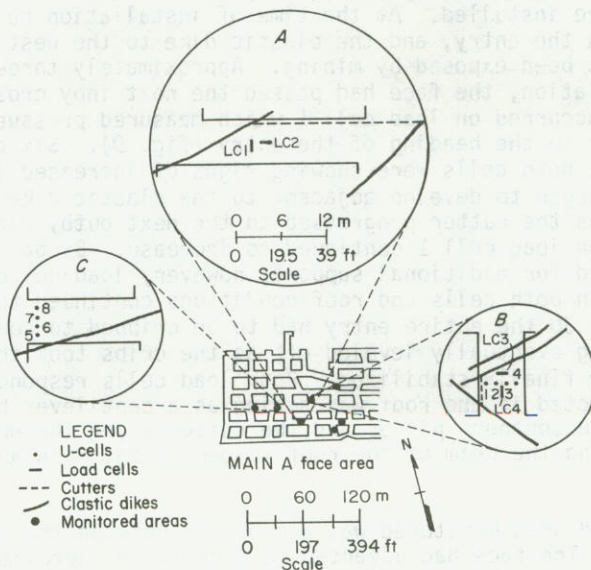


Fig. 8. Location map of monitored areas.

of torque applied by the bolting machine. This torque ranged from 271 to 406 Nm (200 to 300 ft/pounds), which resulted in a U-cell reading of 3960 to 5000 kPag (575 to 723 psig).

All instruments were installed in fresh cuts at the furthest advance of mining in that particular entry or crosscut.

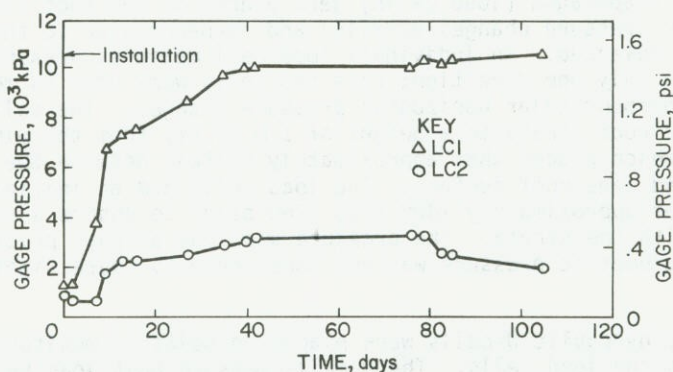


Fig. 9. Behavior of load cells 1 and 2.

The first area monitored was in the L-1 entry of Main A; two load cells were installed (fig. 8, A). The entry had been advanced to a position approximately 3 m (10 ft) west of load cell 1 when the cells were installed. At the time of installation no cutter had formed in the entry, and the clastic dike to the west of the cells had not been exposed by mining. Approximately three days after installation, the face had passed the next inby crosscut, and loading occurred on load cell 1 which measured pressure changes perpendicular to the heading of the entry (fig. 9). Six days after installation, both cells were showing signs of increased loading as a cutter began to develop adjacent to the clastic dike just inby the cells. As the cutter progressed to the next outby clastic dike, the loading on load cell 1 continued to increase. By day 10 posts were installed for additional support; however, loading continued to increase on both cells and roof conditions continued to deteriorate. On day 30 the entire entry had to be cribbed to control the roof. Loading eventually leveled off as the cribs took the load, and the entry finally stabilized. Both load cells responded as would be expected if the roof was acting as a cantilever beam with support on the southern pillar and the cutter along the northern pillar allowing the beam of the roof to bend (fig. 8, A and fig. 9).

The second area monitored was a crosscut between the belt and track entry. The face had advanced to a position approximately 3 m (10 ft) north of load cell 3 (fig. 8, B). Based on the first experience of monitoring cutter development, it was decided that

U-cells would be installed on a row of bolts across the entry to measure bolt loading in addition to using load cells. The clastic dikes just north of the cells had not yet been exposed by mining when all of the U-cells and load cells were installed (fig. 8, B).

Loading of U-cell 1 and load cell 3 occurred as soon as the next 6 m (20 ft) cut into the face was taken from the crosscut which exposed the two clastic dikes (fig. 10 and 11). Load cell 3 measures pressure changes perpendicular to the entry and showed a total increase of approximately 240 KPag (35 psig) in the first day while load cell 4 bled off. Some natural bleed off is expected as the hole deforms if there is no movement in the roof to apply a load to the cell or if extension occurs in the beam of the roof. By the second day the U-cell closest to the inby rib (U-cell 1) showed an increase in load of nearly 2000 KPag (290 psig) as cutter roof failure was beginning to develop along the inby rib between the clastic dikes intersecting the rib (fig. 8, B). From past experience with cutters developing this quickly, it was suggested that supplemental support be installed immediately. Trusses were installed between the rows of bolts, resulting in an immediate decrease in the rate of loading. The addition of trusses effectively reunited the natural beam of the roof and prevented the roof from deteriorating further. Finally loading leveled off (fig. 10 and 11) and the cutter did not become

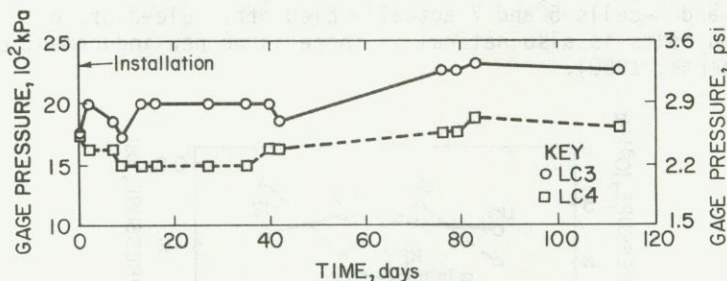


Fig. 10. Behavior of load cells 3 and 4.

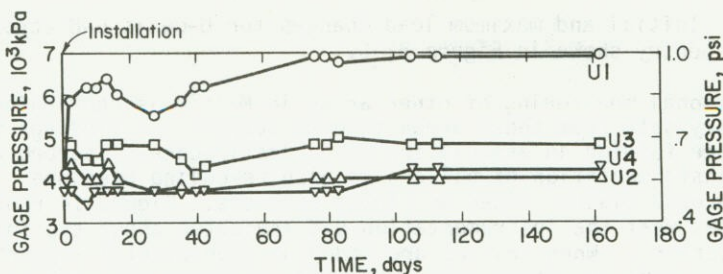


Fig. 11. Behavior of U-cells 1-4.

any more severe. Figure 12 shows the maximum load seen by each U-cell in the crosscut of Figure 8, B. Once again the loading roughly parallels that which would be expected from a cantilever beam (figs. 10, 11, and 12).

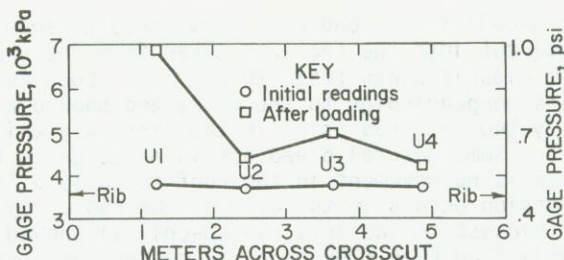


Fig. 12. Initial and maximum loads for U-cells 1-4 across crosscut shown in Figure 8, B.

In the next area (fig. 8, C) trusses were being used on a regular basis and had been successful in deterring the formation of cutters. Shortly after this cut was mined, some rock did fall from the clastic dike in the roof, but this activity ceased once trusses were installed. Figure 13 shows the total loading seen by each U-cell across the entry. Very little total loading occurred on U-cells 6 and 8, and U-cells 5 and 7 actually bled off. Bleed off of loading on bolts is also natural if there is no new inducement of load (Roberts, 1980).

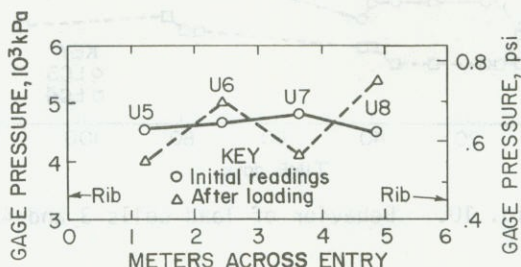


Fig. 13. Initial and maximum load changes for U-cells 5-8 across entry shown in Figure 8, C.

Additional monitoring of other areas in Main A is continuing. Preliminary data from these areas tend to support our findings that cutters are forming in association with clastic-dike-disturbed roof. Initial instrumentation of pillars is also revealing pressure changes around clastic dikes as mining advances. Figure 14 roughly illustrates what the instrumentation has indicated about the formation of cutters. When trusses are added as supplemental support, the clastic dike is effectively supported, allowing the roof to act as a solid beam.

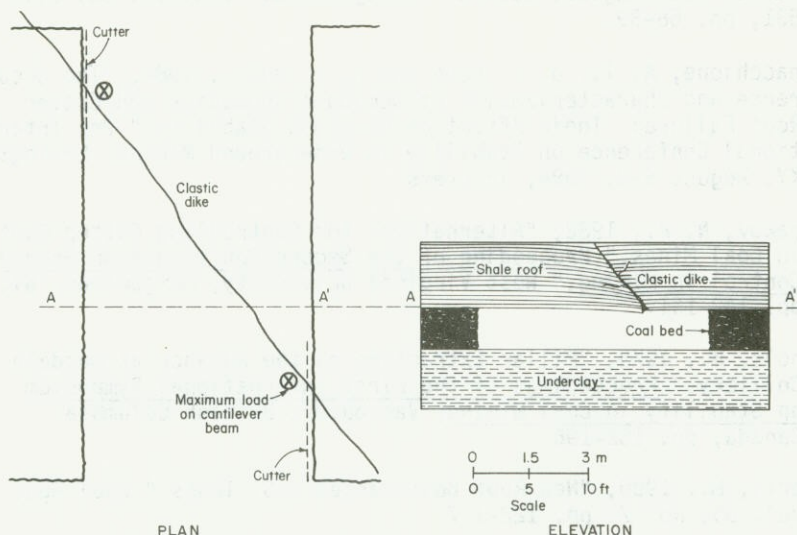


Fig. 14. Roof deflection at clastic dike due to weight of cantilever beam as would be expected from figure 12.

CONCLUSIONS

The increased occurrence of cutter roof failure, in the Main A heading of the study mine, was attributed to the high frequency of clastic dikes. Rock pressure monitoring near clastic dikes revealed that the roof strata were behaving as a cantilever beam. This cantilever beam would initiate cutter roof failure and develop along an entry or crosscut, often extending across several breaks. Staggering of crosscuts prevented the extension of cutters and deterred the occurrence of large falls extending over several breaks. Trusses and cribbing were found to be effective in stabilizing clastic dikes and inhibiting the development of cutter failure when employed immediately after mining.

In situ stress and rock property data will be collected at the mine and may give further insight into the development of cutter roof failure.

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Chapter 63

METHODS OF DESIGNING MECHANICAL ROOF BOLTING IN HORIZONTALLY BEDDED STRATA

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ABSTRACT

The reinforcement mechanism of suspension effect is analyzed based on beam-column theory. The equations of the maximum bending stress, deflection and transferred bolt load for the bolted roof strata are derived. In the analysis, the bolt load is assumed to be point load and the horizontal stress is uniformly applied to each stratum. The reinforcement mechanism of friction effect is also analyzed. The major function of roof bolting in this case is to create the frictional resistance by tensioning the roof bolts, thereby the individual thin strata are "welded" into one single thick stratum. Based on the results, equal spacing of roof bolts is used in the suspension effect while for friction effect, the bolts should be spaced based on equal shear force concept.

The design nomographs for the proper bolting pattern and bolt tension of mechanical roof bolting systems are presented for practical applications.

INTRODUCTION

Today, roof bolting is the primary support system in underground coal mines; more than 95% of the underground coal mined in the United States are mined under roof-bolted roofs. In spite of the widespread usage of mechanical roof bolts in the vast majority of underground coal mines, the mechanisms by which they reinforce the mine roof are still relatively unknown. In particular, there is no consensus regarding the design criteria about the bolting pattern (i.e., bolt length and bolt spacing both along and across the opening) and bolt tension at which they should be installed to reinforce the mine roof

safely and economically. The commonly used 1.22 x 1.22-m (4 x 4-ft) bolt spacing is merely based on the rule-of-thumb, which is obviously not ideal for every situation. Because one bolting plan which is suitable for one mine may not be adequate for the others due to different geological and geometrical conditions. In order to avoid underdesign or overdesign in the roof bolting system, there exists a strong need to establish the design criteria for mechanical roof bolting systems in any underground coal mine which is likely to be encountered in the horizontally bedded strata.

FLEXURAL BEHAVIOR OF THE IMMEDIATE ROOF

In order to analyze the reinforcement mechanism and to design the roof bolting system, it is essential to study the flexural behavior of the immediate roof. Since in some cases, the strength of the roof strata is sufficiently strong, there is no need to install any roof bolt. But for other cases in which the roof strata are thin and weak, roof bolts should be installed to maintain the roof stability.

Based on the strata sequence, the immediate roofs can be divided into three types (Fig. 1). They are: (A) the deflection of each stratum is larger than that of its overlying stratum, hence, each stratum deflects independently, (B) some stratum (or strata) deflects more than that of its overlying stratum, and (C) the deflection of each stratum is larger than or equal to that of its underlying stratum.

The immediate roof of strata sequence A is usually most critical to the roof stability, therefore, an adequately designed roof bolting system is necessary. For strata sequence C, if the strength of the thick stratum is sufficiently strong, there is no need to install any roof bolt. For type B immediate roof, a suitable roof bolting system may be needed, depending on the strength and location of the thick (or competent) stratum. Table 1 shows the flexural behavior of three strata types in terms of total outer-fiber stress. As it can be seen clearly in this table, among three strata types, only strata type A is unstable because the total upper-fiber stress is larger than the tensile strength of the rock.

It should be noted that during the analysis of the flexural behavior of the immediate roof, if any stratum loads on its underlying stratum, then, these two strata are combined into one equivalent stratum. Since in real condition, the strata sequence of the immediate roof is arbitrary, the strata combination process may become very complicated. To overcome this difficulty, a computer program has been developed for the strata combination process and the determination of the maximum bending stress and deflection in a generalized immediate roof (Tang and Peng, 1983).

In underground coal mines, since high in-situ horizontal stress always coexists with vertical stress because of the lateral constraint

or tectonic activities (Aggson and Curran, 1978), the effect of axial loading due to horizontal force should be considered in the design of roof bolting system. The effect of horizontal stress will not only increase the bending stress (due to transverse loading), but also influence the total stress (i.e., the sum of horizontal stress and bending stress) on the outer fiber of the roof stratum. Furthermore, when the horizontal stress is very large, thin strata in the immediate roof will fail due to buckling.

ANALYSIS OF REINFORCEMENT MECHANISMS

Suspension Effect

In horizontally bedded strata, the reinforcement mechanism of suspension effect through roof bolting is mainly due to the transfer of a portion of the weight of the weaker (or thinner) strata to the competent (or thicker) strata (Panek, 1962). In order to apply this type of reinforcement mechanism the strata sequence of the immediate roof should be of such an order that the deflection of the lower stratum is larger than that of the upper strata. However, this strata sequence is not limited to the original strata sequence such as strata type A (see Fig. 1). In some cases, it can also be applied to strata type B as long as the final strata sequence (after strata combination) is of such an order.

The analysis of reinforcement mechanism of suspension effect is made based on the following assumptions:

- (1) All bolted strata are constrained to have equal deflections at bolt locations due to adequate bolt tension.
- (2) There is no friction at the contact points between roof strata.
- (3) Each stratum is subjected to transverse loading and axial loading simultaneously.
- (4) The transferred bolt load acts as a point load.
- (5) Perfect anchorage is assumed.

By means of the load transfer through roof bolting, both the bending stress and deflection of the weaker strata are decreased, thus increasing the roof stability. However, in the mean time, both the bending stress and deflection of the competent (or anchoring) strata are increased. In order to maintain the stability of the bolted strata, the load transfer should be kept within a certain limit such that the total outer-fiber stress (after the load transfer) of the anchoring strata is less than the strength of rock. Otherwise, failure will initiate from the anchoring horizon, which may lead to the failure of the whole bolted strata.

Based on the above assumptions, the major equations for the suspension effect through roof bolting are derived as follows:

$$R_i = \left(\frac{\sum_{k=1}^n q_K}{\sum_{k=1}^n \frac{E_K I_K}{S_K}} \right) / \left(\frac{q_i}{\frac{E_i I_i}{S_i}} \right) - 1 \quad (1)$$

$$V_{Bi} = V_i + \Delta V_i = (1 + R_i) V_i \quad (2)$$

$$T_i = \alpha R_i q_i L \quad (3)$$

$$\sigma_{Bi} = \sigma_i + \Delta \sigma_i = (1 + \beta R_i) \frac{b M_{oi}}{b t_i^2} \quad (4)$$

$$NP_B = \sum_{i=1}^{n-M} T_i (+) = - \sum_{i=n-m+1}^n T_i (-) \quad (5)$$

$$S = 5 \eta(u) - \frac{4 u X(u) \lambda(u)}{\tan u} \quad (6)$$

where: $i = 1, 2, \dots, n$, subscript refers to position of a stratum in the immediate roof

R_i = loading factor depending on q , E , I , and S of all the bolted strata

q = uniformly distributed load per unit length of the roof stratum (due to its self weight)

E = Young's modulus of roof stratum

I = moment of inertia of roof stratum

S = factor due to horizontal stress

V_B = deflection of bolted roof stratum at bolt location

V = deflection of unbolted roof stratum at bolt location

ΔV = change of deflection due to roof bolting

T_i = transferred bolt load for the i -th stratum

L = roof span

σ_B = maximum bending stress of bolted roof stratum

σ = maximum bending stress of unbolted roof stratum

$\Delta \sigma$ = change in bending stress due to roof bolting

b = spacing between sets of bolts along the opening

t = thickness of roof stratum

M_o = maximum bending moment of unbolted roof stratum

n = number of bolted roof strata

N = number of bolts per row (or set)

P_B = bolt tension

$u = L/2(P/EI)^{1/2}$

P = horizontal force

$\eta(u)$, $X(u)$, $\lambda(u)$ = functions of u

α, β = coefficients depending on number of bolts per row and location of bolts

In the derivation of equations related to the suspension effect, the spacings between roof bolts in each row were made to be arbitrary in the beginning; but after extensive study of suspension effect with different number of bolts and various bolt spacings, it is found that the case of equal spacing of roof bolts seems to have better results than other cases of bolt spacing, therefore, roof bolts with equal spacings are adopted in the analysis of suspension effect by using the computer program (Tang, 1984).

Friction Effect

In some underground coal mines, there is no competent stratum within a reasonable distance above the roofline from which roof bolts can be anchored to suspend the lower weaker strata. Under this condition, the successful application of roof bolts to reinforce the immediate roof is by friction effect (or beam-building effect).

The basic concept in the reinforcement mechanism of friction effect is to increase the shear (or frictional) resistance to slip on the interface between the thin strata, thus "building up" a single thick stratum. Since the flexure of the thick stratum is smaller than that of the thin stratum, the stability of the bolted roof strata is increased. To achieve the friction effect, the mechanical roof bolts must be tensioned, because the frictional resistance resulting from clamping action is solely due to the tension in the roof bolts.

Since the major function of friction effect through roof bolting is to increase the shear resistance along the interface, inhibiting slip and reducing the bending stress, what is then the proper amount of required shear resistance which the function of roof bolting should provide? This can be answered by the following example:

Consider a single beam of thickness t and a multiple beam having four members each with thickness $t_1 = t/4$ (Fig. 2). Each member of the multiple beam is assumed to be made of same material and there is no bonding on the interface. For single beam, there is no discontinuity in the shear stress distribution. But for multiple-member beam, the shear stress in each member is maximum at the neutral axis and zero at the outer fibers (or at the interfaces). Therefore, in order to cause the four-member beam to behave like a single beam of the same total thickness, the required shear resistance should be of such magnitude that it can provide the shear stress of magnitude aa' , bb' and cc' at three levels in the cross section X_1X_2 such that the shear stress distribution is continuous (Fairhurst and Singh, 1974).

For a fixed-end beam subject to uniformly transverse loading, the shear force varies from maximum at two ends to zero at the center of the span (Fig. 3). The adequate location or spacing of the roof bolts should be such that more bolts are installed toward the ends of the beam with no bolt at the center of the span. Furthermore, in order to have each bolt subject to equal shear force, which is required for maintaining the equilibrium of shear force on both sides of each bolt,

the "equal shear force" concept should be considered in the determination of proper bolt spacing.

Based on the foregoing discussion, the major equations with regard to the friction effect are derived as follows:

$$Q = \frac{V I_A}{I_z} \quad (7)$$

$$Q_{ave} = \frac{Q_{max} + Q_{min}}{2} \quad (8)$$

$$F_{SA} = \int Q_{ave} dx \quad (9)$$

$$P_B = \frac{F_{SA}}{\mu} \quad (10)$$

$$X_{Ei} = \left(\frac{2i}{N}\right)^{\frac{1}{2}} \frac{L}{2}, \quad i = 1, 2, 3, \dots, N/2 \quad (11)$$

$$X_{Bi} = \left(\frac{2i-1}{N}\right)^{\frac{1}{2}} \frac{L}{2}, \quad i = 1, 2, 3, \dots, N/2 \quad (12)$$

Where

- Q = shear flow at the interface
- V = shear force
- I_A = statical moment of area of individual layer around the neutral axis of the whole cross section, e.g. $\int_{area} y dA$ in Fig. 3.
- I_z = moment of inertia around the neutral axis of the whole cross section
- $Q_{max}, Q_{ave}, Q_{min}$ = maximum, average and minimum shear flow at each segment of equal shear force
- F_{SA} = required shear resistance at each segment of equal shear force
- μ = coefficient of friction at interface
- P_B = required bolt tension
- X_{Ei} = the distance between the center of the span and the rib end of the i-th segment of equal shear force, see Fig. 3
- X_{Bi} = bolt location, distance from the center of the span to the i-th bolt, see Fig. 3

It is important to note that the above equations can be applied to the immediate roof made up of multiple strata with different material properties and variable strata thicknesses. As long as the deflection of each stratum is smaller than or equal to its overlying strata, they are applicable. However, for roof strata made of different material properties, two points must be clarified: First, the equivalent unit weight of all bolted strata is used in the determination of shear

force. Second, the equivalent (or transformed) cross-sectional areas instead of the original ones should be used in the analysis. After a beam of various materials is reduced to an equivalent beam of one material, the above equations still are applicable.

Bolt Length and Row Spacing

For the immediate roof which is reinforced by suspension effect, the bolt length is required to extend at least 30 cm (12 in.) into the competent stratum. For the immediate roof which is reinforced by friction effect, the bolt length is equal to the total thickness of the bolted strata.

With regard to the determination of row spacing, the concept is the same for both suspension and friction effects. Although it is not explicitly expressed in the derived equations, it is mainly dependent on the bolt tension and the anchorage capacity of the anchoring horizon. The larger the required bolt tension, the smaller is the row spacing. However, since the bolt tension cannot exceed the allowable anchorage capacity (i.e., 60% of anchorage capacity), it is, therefore, advisable to choose the allowable anchorage capacity as the adequate bolt tension and, then find out the proper row spacing by the following equation

$$SA = \frac{P_A}{P_B} b \quad (13)$$

where SA = proper row spacing
 P_A = allowable anchorage capacity
 P_B = bolt tension obtained
 b = unit width of beam, or assumed row spacing

In the computer program which is developed for designing the bolting pattern and bolt tension, Equation 13 is used for the determination of proper row spacing.

NOMOGRAPHS FOR THE DETERMINATION OF PROPER BOLTING PATTERN AND BOLT TENSION

Reinforcement by Suspension Effect

For the strata type which should be reinforced by suspension effect, two nomographs are derived for the determination of proper bolting pattern and bolt tension. Figure 4 is used for the determination of maximum bending stress of the weak (or supported) strata. Figure 5 is used for the determination of proper bolt number and bolt tension. The application of these two nomographs are described as follows:

Step 1: In Fig. 4, the maximum bending stress of the supported strata can be determined by following the dotted line abcdef. In

this nomograph K is equal to $1 + \beta R_1$ (see Equation 4). In order to obtain K for different number of bolts per row, Fig. 6 and Fig. 7 are also needed. In the design procedure, starting from one bolt per row, find the maximum bending stress corresponding to different bolt numbers.

- Step 2: Based on the result of Step 1, determine the total upper-fiber stress and total lower-fiber stress. Then compare the former with tensile strength and compare the latter with compressive strength. Choose the smallest number of bolts which can provide the strata stability.
- Step 3: Based on the result of Step 2, find the proper bolt tension in Fig. 5. It should be noted that this nomograph is only for anchoring strata. The bolt tension for various bolt number can be determined by following the dotted line abcdefg in this figure.
- Step 4: Compare the bolt tension from Step 3 with the allowable anchorage capacity. If the former is less than the latter, then the bolt number per row is determined. Otherwise, try the next higher bolt number.

Reinforcement by Friction Effect

For the strata type which should be reinforced by friction effect, two nomographs are derived for the determination of proper number of bolts in each row. In this case, the proper bolt tension is assumed to be equal to the allowable anchorage capacity. Therefore, bolt tension is a known value. Since the stratum thickness under this condition is usually very thin, the maximum bending in this stratum is generally larger than the tensile strength. As a result, without "welding" these thin strata into one thick stratum by roof bolting, they will eventually fail. The procedure of using these nomographs are described as follows:

- Step 1: In Fig. 8, by following the dotted lines ed and abcd, find the intersection of these two lines, which is the required stratum thickness which will not fail. The number of strata which need to be bolted can be obtained by dividing this thickness with the average thin strata thickness.
- Step 2: In Fig. 9, find the C_x values for different number of bolts by following the dotted line abcdefg. The γ values in the horizontal axis in the figure for various number of bolts are listed in Table 2.
- Step 3: Compare C_x value with C_B value in Table 3. The proper bolt number is obtained when C_x is larger than C_B . Then based on Equation 12 obtain the proper bolt spacing.

CONCLUSIONS

The design nomographs and their usages for the determination of proper bolting pattern and bolt tension for mechanical roof bolting systems have been developed and presented. In order to obtain better

results, the assumptions which are made in deriving these nomographs should be understood and satisfied in the prototype. For example, the nomographs for the suspension effect are most valid for strata type A. But the nomographs for the friction effect are most applicable to some cases of strata type C, in which the deflection of all strata must be equal. For other cases, these nomographs can merely give approximate results. If very accurate results are desired, it is suggested that the computer program developed by Tang (1984) must be used.

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Table 1. Total outer-fiber stresses and strengths for three strata types

Strata Type	σ_{TU} psi	C_T psi	σ_{TL} psi	C_P psi	Stable	Unstable
A	237	71.5	-837	1447		✓
B	-56	71.5	-544	1447	✓	
C	-131	250	-470	6270	✓	

Table 2. γ for various number of bolts

Bolt No.	1	2	3	4	5	6	7	8	9	10
γ	0.375	0.265	0.216	0.188	0.168	0.153	0.142	0.133	0.125	0.119

Table 3. C_B for various number of bolts

Bolt No.	1	2	3	4	5	6	7	8	9	10
C_B	1.0	0.707	0.577	0.5	0.447	0.408	0.378	0.354	0.333	0.316

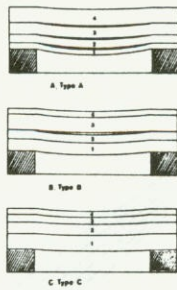


Fig. 1. Three strata types of immediate roof.

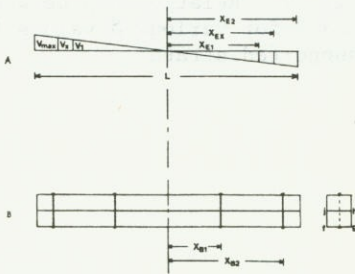


Fig. 3. (A) Equal shear force diagram. (B) Bolt spacing based on "equal shear force" concept.

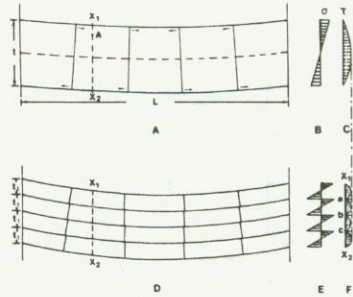


Fig. 2. Flexure of a fixed-end beam subject to transverse uniform loading: (A) (B) (C) - single thick layer; (D) (E) (F) - 4 thin layers.

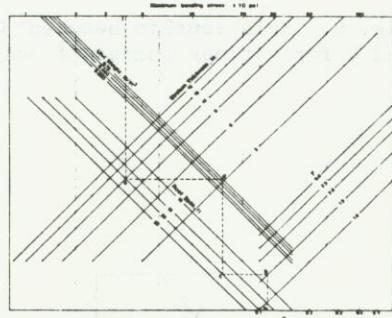


Fig. 4. Nomograph for the determination of maximum bending stress for supported strata.

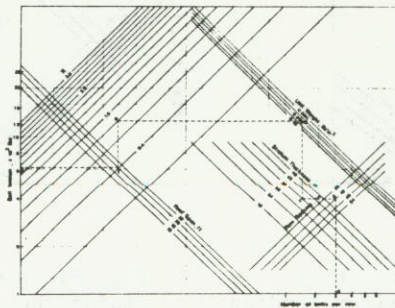


Fig. 5. Nomograph for determining the proper bolt number and bolt tension for suspension effect.

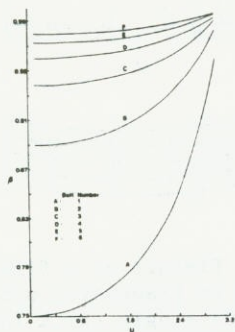


Fig. 6. Relationship between β and U for various number of bolts.

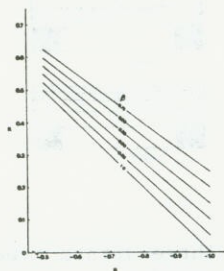


Fig. 7. Relationship between K and R for various β values for supported strata.

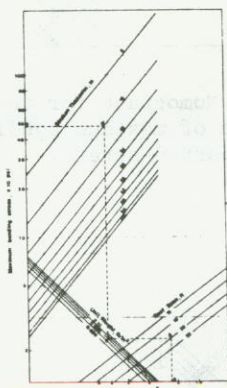


Fig. 8. Nomograph for the determination of maximum bending stress in the unbolted strata.

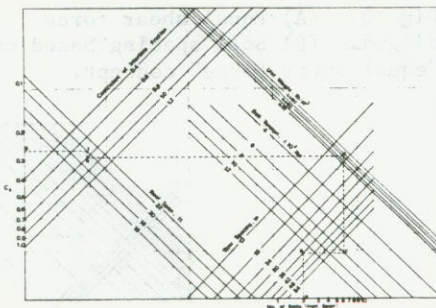


Fig. 9. Nomograph for determining the proper bolt number and bolt spacing for friction effect.

Chapter 64

ROCK BURSTS IN HUNAN COLLIERIES

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ABSTRACT

In this paper the imminent relationship between rock burst and its phenomenon with normal rock pressure and its appearance, as well as the factors of one changing into another have been analyzed. Based on this, the necessary and sufficient appearing factors of rock bursts have been put forward, and the term "burst pressure" has been defined.

According to the statistics data of tremendous rock bursts which appear in collieries in Hunan Province, the features of high frequency, great strength and shallow depth of the beginning burst have been found. There are some special structures in the coal seams, which are closely related with the burst phenomenon. A hypothesis of "star structure" has been suggested for the appearing mechanism of bursts. Inside and around this star structure the distribution rule of burst pressure and rock strength has been researched depending upon the theory of the limited stress field. A new description of the appearing causes of bursts has been raised.

THE ESSENTIAL FEATURES OF ROCK BURSTS IN HUNAN COLLIERIES

In Hunan Collieries, rushing phenomena appear almost by way of bursting. Coal, rock, gas and water all rush out, especially the outburst of coal and gas at Ma-Tian Colliery in April 1955 to the end of 1980, 3,421 bursts had come into being during this period involving 30 mines. A great many of the bursts threatened the mine safety and production seriously as well as the measures taken to prevent them were not very efficient. Miners in China usually called them "mine cancers."

The essential features of bursts in Huanan are as follows:

1. Fast advancing speed and high appearing frequency. From Table 1 and Fig. 1, it is evident that from 1955 to 1967 the increasing speed of bursts was rather slow during these 13 years. From 1968 to 1980 it was very fast during these years and reached a peak in 1979. One hundred eight burst appeared during the first 13 years and 2793 during the second 13 years. Otherwise, in the second 13 years there were 15.5 times as many as in the first. It is very clear that the bursting frequency rises with developed mines and mining depth. The statistics data of 1980 tells us that the average bursting time per 10,000 tons of coal extracted from the mine reached 92.

Table 1. The yearly increment of bursts in Hunan Collieries.

Year	Bursting Ratio (times/year)	Year	Bursting Ratio (times/year)
1955	1	1968	17
1956	0	1969	44
1957	2	1970	93
1958	16	1971	163
1959	4	1972	189
1960	11	1973	173
1961	12	1974	208
1962	24	1975	236
1963	12	1976	182
1964	16	1977	258
1965	14	1978	369
1966	41	1979	440
1967	27	1980	421

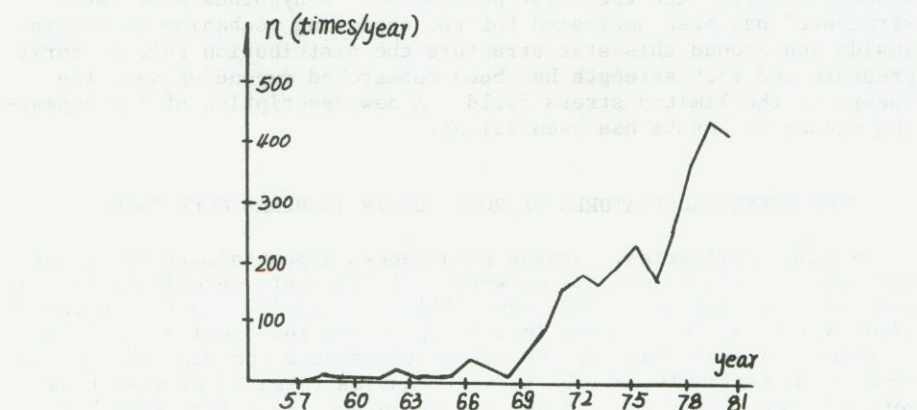


Fig. 1. The yearly increment of bursts.

2. The possibility and strength of bursts increases as the mining depth increases. From Table 2 and Fig. 2, the possibility η , maximum $Q_{\max}$ and average strength $\bar{Q}$ vary as the increasing of mining depth. From 80 to 150 m, the possibility η evidently rises with the increase of mining depth. Beyond 150 m the curve η drops because the drivaging length was not long enough. The maximum strength $Q_{\max}$ and average strength $\bar{Q}$ rise as the mining depth increases.

Table 2. The relationship between bursts and mining depth in Red Guard Mine.

Mining Depth (m)	Bursting Times	Bursting Ratio (times/m)	Max. Strength (T/time)	Average Strength (T/time)
80-100	4	0.2	200	63
101-120	14	0.7	150	30
121-140	56	2.8	2500	86
141-160	130	6.5	2000	88
161-180	92	4.6	3200	163
181-200	25	1.25	1700	285
201-220	9	0.45	4500	569

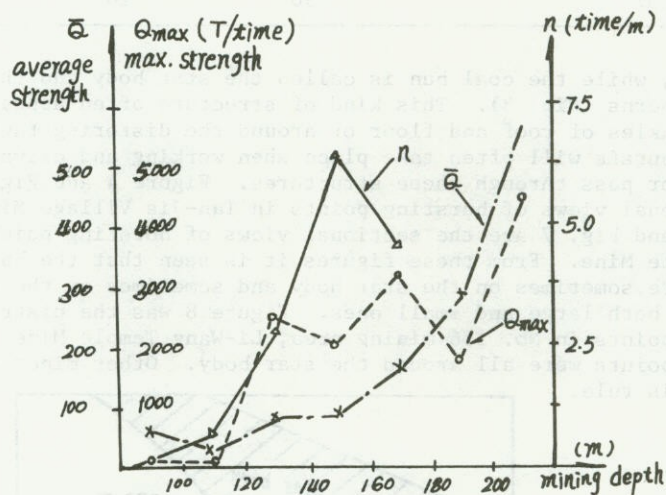


Fig. 2. The relationship between bursting and mining depth.

3. Shallow depth of the beginning burst. From Table 3 the beginning depth of bursts did not go beyond 100 m in 17 mines until 1981 while the static pressure of overburden weight is not beyond 25 kg/cm², and the gas pressure is 2.1 kg/cm² to 34 kg/cm².

4. Bursts often take place inside the star structures or around them. A bun-shaped coal seam consisting of two or more branches which look like a star-structure layer of coal or rock is called a star

Table 3. The conditions of beginning bursts in Hunan Collieries.

Bursting Places	Max. Pressure of Gas (kg/cm ²)	Dep. of Beginning Bursts (m)	Strength of Beginning Bursts (T/time)	Overburden Weight (kg/cm ²)
Uniting M.	10.0	30	150	7.5
Yuan-Jia M.		30	30	7.5
Shi-Cha M.	10.6	45	80	11.25
Sha-Tian M.		50	4	12.5
Li-Wang Temple M.	12.8	60	30	15
Horse Head Pit M.		60		15
Ai-Huo Hill M.		66	20	16.65
Golden Bamboo Hill M.		70	4	17.5
Tong-Zi Hill M.	15	70	4.5	17.5
Nan-ping Shaft		75	4	18.75
New Life M.	2.2	80	4	20
Internal Red M.		83	2	20.75
Xie-Ling M.		80	8	20
Feng-Shu M.		87	50	21.75
Tao-Shui M.		84.6	20	21.15
Red Flag M.	19.2	90	15	22.5
Luo-bo-An M.		90	20	22.5

structure, while the coal bun is called the star body and the branches the star horns (Fig. 3). This kind of structure often appears in small wrinkles of roof and floor or around the distorting fault. Large or small bursts will often take place when working and drivaging faces are near or pass through these structures. Figure 4 and Fig. 5 are the sectional views of bursting points in Tan-Jia Village Mine. Figure 6 and Fig. 7 are the sectional views of bursting points in Li-Wang Temple Mine. From these figures it is seen that the bursting points were sometimes on the star body and sometimes on the star horns, including both large and small ones. Figure 8 was the distribution of bursting points in No. 156 mining area, Li-Wang Temple Mine. The bursting points were all around the star body. Other mines also obeyed this rule.

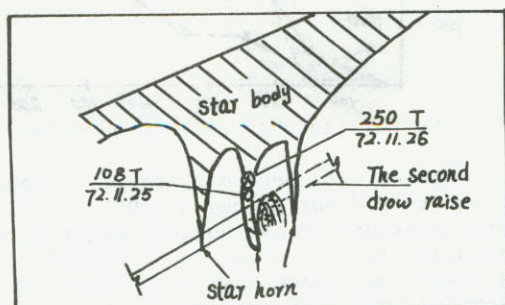


Fig. 3. The star structure and bursting.

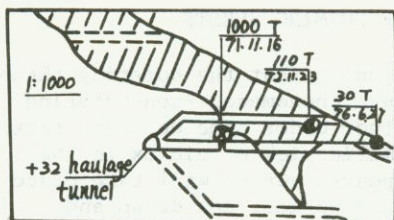


Fig. 4. Sectional view of bursting points in Tan-Jia Village Mine.

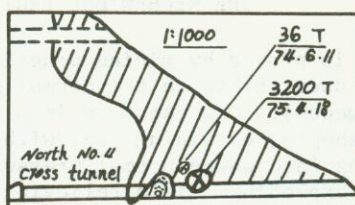


Fig. 5. Sectional view of bursting points in Tan-Jia Village Mine.

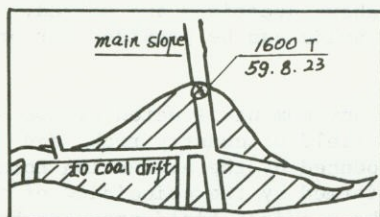


Fig. 6. Sectional view of bursting points in Li-Wang Temple Mine.

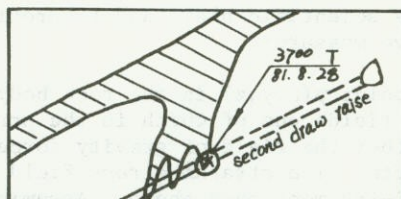


Fig. 7. Sectional view of bursting points in Li-Wang Temple Mine.

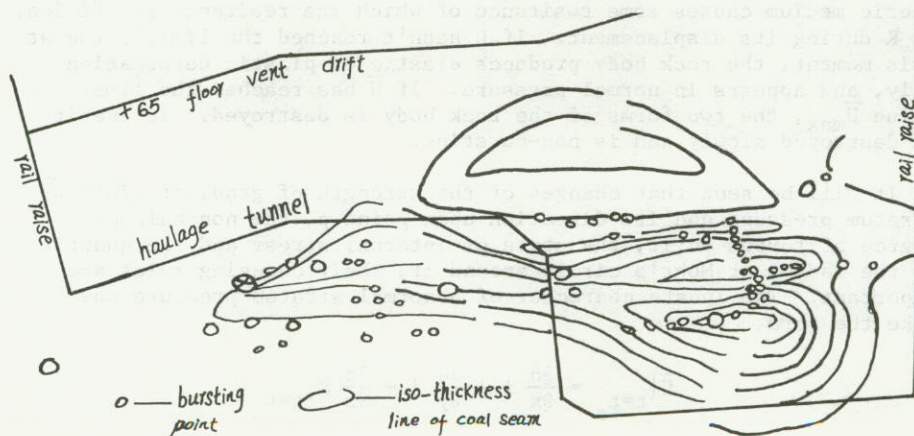


Fig. 8. The distribution of bursting points in No. 156 mining area Li-Wang Temple mine.

In conclusion, a star structure is a "cancer cell" of the "Mine Cancer" in Hunan Collieries.

THE MECHANICAL FACTORS OF FORMING BURSTS

It is proved by mining practice in Hunan that the bursting phenomenon does relate to the normal pressure phenomenon (non-bursting phenomenon). One can turn into the other under some definite factors. For example, obtaining coal with pneumatic pick or pickaxe in the working face where normal pressure appears, bursts will take place after exploding. If a relieving seam is worked, gas drawn and water flooded, the coal seam where bursts often take place, can be turned into non-bursting. What is the imminent relationship between bursting pressure and normal pressure? What is the necessary and sufficient factors for bursting? The factors of turning one into the other can be determined only after the above questions are solved. Then a scientific basis and theoretical guide can be provided for preventive measures.

A point $p(x,y,z)$ in the rock body at any moment is acted by two force fields one of which is the stress field of country rock, and the other the field of gravity force produced by the attraction force of earth. The stratum stress field is formed by the coinciding of the two fields mentioned above. Assuming the gradient field produced by gravity force $\bar{V}$, the principal stress of country rock σ , the gradient field produced by the principal stress $\bar{U}$, and disturbing the rock body by mining and drivaging or something else at the moment t_0 , point p will displace an absolute displacing velocity $\bar{U}$ from point A or the velocity $\bar{V}$ from the disturbing center O' . After an interval dt , a small displacement d_1 is produced and arrives at point B . The peripheric medium causes some resistance of which the resistance coefficient is K during its displacement. If $\bar{U}$ hasn't reached the limit value at this moment, the rock body produces elastic or plastic deformation only, and appears in normal pressure. If $\bar{U}$ has reached the limit value $\bar{U}_{\max}$, the two forms of the rock body is destroyed. In one it is destroyed slowly and is non-bursting.

It will be seen that changes of the strength of gradient field of stratum pressure and its direction upon point p , the non-balance degree of forces on it, the state of internal stress and the quantity of the radius of Mohr's circle around it, their changing rules are important. The innate character of a normal stratum pressure now take the form

$$\bar{U}|_{t=t_0} = \frac{\partial \sigma}{\partial x} i + \frac{\partial \sigma}{\partial y} j + \frac{\partial \sigma}{\partial z} K |_{t=t_0}$$

The innate character of a bursting pressure now takes the form

$$\bar{U}_{\max}|_{t=t_0} = \left(\frac{\partial \sigma}{\partial x} i + \frac{\partial \sigma}{\partial y} j + \frac{\partial \sigma}{\partial z} K \right)_{\max}|_{t=t_0}$$

They lead to the difference of quality as a result of the difference of quantity. Since the static rock mechanics has defined σ as

the stratum pressure (earth pressure), it is evident that it can hardly reflect the rule of the dynamic process of stratum displacement. Therefore, the dynamic rock mechanics has to raise its own new theory for this subject.

Bursting pressure $\bar{U}_{\max}$ is the only necessary factor for producing burst, but is not sufficient. Point p reacted to $\bar{U}_{\max}$ has the lowest speed to leave the body, but is not sufficient for it to rush out to the cavity. From Fig. 9, the displacement velocity $\bar{V}$ is coincided by absolute displacement velocity $\bar{U}$ and disturbing velocity $\bar{b}$, i.e.

$$\bar{V} = \bar{U} + \bar{b} \quad (1)$$

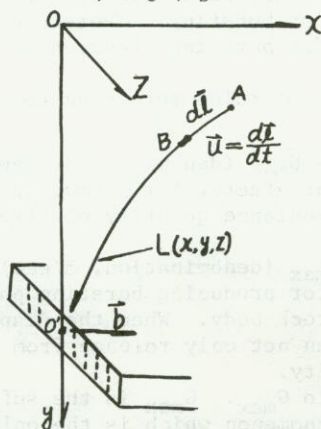


Fig. 9. Pressure and displacement.

The relationship between $\bar{V}$ and $\bar{U}_{\max}$ may be expressed as

$$\bar{U}_{\max}|_{t=t_0} = \bar{V} - K\bar{V} \quad (2)$$

The quantity of $\bar{V}$ depends upon the absolute value $\bar{U}_{\max}$, and its change depends upon the value $d\bar{U}_{\max}/dt = g$. $\bar{V}$ increases while $g > 0$, $d\bar{V}/dt$ also has its own corresponding change until $\bar{V}$ reaches the limit value $\bar{V}_{\max}$ at last. Now the rock body produces burst and forms bursting phenomenon.

At t_0 whether moment point p produces bursting or not, it depends on not only the necessary factor $\bar{U}_{\max}$, but also the energy ratio G . The definition of G is the minus value of the dot product of $\bar{U}$ and $\bar{V}$, i.e.,

$$G = -\bar{U} \cdot \bar{V} \quad (3)$$

When G reaches the limit value, it could be presented by the equation

$$\begin{aligned}
 G_{\max} \text{ at } t=t_0 &= -\bar{U}_{\max} \cdot \bar{V}_{\max} \Big|_{t=t_0} = -\bar{U}_{\max} \cdot (\bar{U} + \bar{b})_{\max} \Big|_{t=t_0} = \left[\frac{\partial \sigma}{\partial t} \right]_{\max} \Big|_{t=t_0} \\
 &= -\bar{U}_{\max} \cdot \bar{V}_{\max} \Big|_{t=t_0} = [K(\bar{U}^2 + \bar{b}^2) + \bar{U} \cdot (2K\bar{b} - \bar{V}) - \bar{V} \cdot \bar{b}]_{\max} \Big|_{t=t_0} \\
 &\quad + \left[\frac{dW}{dt} \right] \Big|_{t=t_0} \quad (4)
 \end{aligned}$$

where W = energy in volume unit of rock body

Owing to the fact that both $\bar{U}$ and $\bar{V}$ reaches the limit value at the same time, rock body can not be destroyed slowly now. It releases numerous energy by way of bursting. Therefore, $G_{\max}$ is the necessary and sufficient factor for bursting phenomenon.

Equation 4 exposes some rules referring to the essential factors of bursting, such as

1. Bursting pressure $\bar{U}_{\max}$ (denomination, Newton/cm^3). Bursting pressure is the necessary factor for producing bursting phenomenon and the limit value of non-balance quantity of stress field of country rock.

2. Bursting speed $\bar{V}_{\max}$ (denomination, m/sec). Bursting speed is the sufficient factor for producing bursting phenomenon and the limit value of stability of rock body. When the displacement velocity reaches $\bar{V}_{\max}$ point p can not only release from the rock body, but can also rush into the cavity.

3. Limit energy ratio $G_{\max}$. $G_{\max}$ is the sufficient and necessary factor for bursting phenomenon which is the only appearance when $G_{\max}$ has been satisfied. When $G_{\max} > 0$, energy accumulates in rock body, and pressure increases sharply. When $G_{\max} < 0$, energy and pressure release sharply. Thus the absolute value $|G_{\max}|$ represents the bursting strength and sharpness. In a different time and place, the value $G_{\max}$ is different due to different conditions.

4. The absolute deformation velocity $\bar{U}_{\max}$. The quantity of $\bar{U}_{\max}$ mainly depends upon the mechanical characters of the rock body itself and the performance of $\bar{U}_{\max}$. As a general rule, before producing bursting $\bar{U}$ increases sharply and rises to $\bar{U}_{\max}$ suddenly at the bursting moment.

5. Disturbing speed $\bar{b}$. The advancing speed $\bar{b}$ from the disturbing center formed by mining and drivaging to rock body is of great influence to bursting. $\bar{b}$ affects $\bar{V}$ directly. As $\bar{b}$ increases suddenly at the exploding moment, $\bar{V}$ and G also varies directly as $\bar{V}$ changes suddenly. Thus the possibility of bursting increases sharply.

6. The stability of rock body K . The stability of rock body, i.e., inertness of rock body, K is its change ratio of density. If it is destroyed $\bar{U}$ increases suddenly, or if the K value decreases suddenly it produces bursting.

7. The strength of rock body $[\sigma]$. The field strength σ of rock body which acted by gradient field of stratum pressure is also its strength $[\sigma]$. The larger the non-uniformity of distribution of $[\sigma]$, the larger the value of $\bar{U}_{\max}$ and the more dangerous the bursting.

8. The detection and prediction. The basic task of the detection and prediction is to detect and predict the time, place and values of $\bar{U}_{\max}$ and $G_{\max}$. If this information can be obtained, we can predict when and where the burst will take place and what the strength is.

9. The conditions of turning bursting phenomenon into non-bursting phenomenon. The conditions of turning one into another depends upon the reaction direction of the equation $G_{\max} \rightarrow G$. If the preventive measures are to make the reaction towards the right, such as extracting releasing seam, or to make $\bar{U}_{\max}$ to $\bar{U}$, a bursting one can be turned into a non-bursting one. If we do exactly the opposite such as extracting a pillar shaped like an isolated island with exploding, we obtain the opposite result.

THE DISTRIBUTION RULE OF THE STRENGTH OF ROCK BODY AND BURSTING PRESSURE

If the distribution rule of $\bar{U}_{\max}$ and $[\sigma]$ in 3D space has to be researched, it must be greatly simplified. The author drew a conclusion in another paper in which he pointed out the distribution rule of $\bar{U}_{\max}$ and $[\sigma]$ along a sliding line (or plane) is

$$\bar{U}_{\max} = \frac{d\sigma}{dx} = \frac{r}{2} [\operatorname{tg} Q + (\frac{1}{\operatorname{tg} Q} + \frac{1}{\operatorname{tg} \phi})^{-1} - (\operatorname{tg} Q + \operatorname{tg} \phi)^{-1}] \quad (5)$$

$$[\sigma] = \frac{r}{4} R \left[\frac{\sqrt{1 + \operatorname{tg}^2 Q}}{\operatorname{tg} Q (\operatorname{tg} Q + \operatorname{tg} \phi)} + \frac{2}{\sqrt{1 + \operatorname{tg}^2 Q}} \right] \quad (6)$$

where

- σ - uniform stress
- C - cohesive force
- ϕ - internal friction angle
- σ_1, σ_3 - the maximum and minimum principle stress under a limiting state
- Q - the angle included between the sliding plane (or weak plane) and the horizontal plane
- r - volume weight
- R - curvity radius of sliding plane (or weak plane)
- X, Y - X -axis represents horizontal direction,
 Y -axis represents vertical direction

It is known that the strength of rock body mainly depends upon the characters of various kinds of weak planes such as stratification plane, cleavage plane, fault plane, etc. It is seen from Equation (6) that the strength of the rock body will become very small in all planes where the curvity radius of the stratification plane becomes very small or slope $\operatorname{tg} Q$ increases suddenly. Why did the bursts often take place around the "star structure?" It is because the R becomes very small at the top of a star horn or at the contact-segment between a star horn and a star body and makes the strength there too low. The bursting points in Hunan Collieries often took place where strength of

rock body became minimum. For instance, Fig. 10 and Fig. 11 are sectional views of bursting points in Tong-Zi Hill Mine. From these figures it is seen that coal seam and rock strata reacted to the great tectonic stress field broke off and folded in some places where the coal body formed various types of star structures by squeezing. When mining and drivaging faces contacted these bursting points would take place.

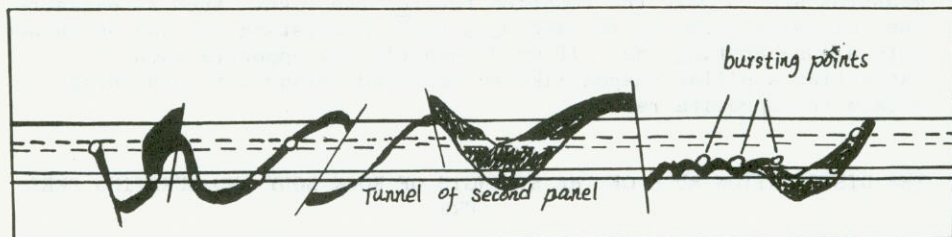


Fig. 10. The sectional view of bursting points at ± 0 tunnel of second panel in Tong-Zi Hill Mine.

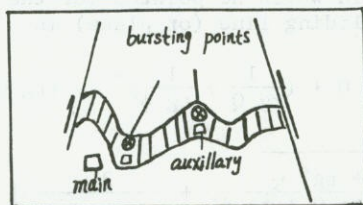


Fig. 11. The sectional view of main and auxiliary tunnels in Tong-Zi Hill Mine.

The rule described in Equation (5) is the distribution rule of $\bar{U}_{\max}$. In this equation, when the absolute value of $\text{tg } Q$ becomes very large, constant l and the influence of $\text{tg } \phi$ to $\bar{U}_{\max}$ became relatively small. They can be neglected if they are zero. Equation (5) and (6) can now be simplified as follows:

$$\begin{aligned} \text{tg } Q &= \frac{3}{4} R \frac{r}{[\sigma]} \\ \bar{U}_{\max} &= \frac{3}{8} r^2 \frac{R}{[\sigma]} \end{aligned} \quad (7)$$

The rule exposed by Equation (7) points out that r and $[\sigma]$ are constant in a short time and $\bar{U}_{\max}$ mainly depends upon the curvity radius R . Figure 12 shows the distribution state of the rock body in a limited state which was calculated from Equation (7). Passing point A on the weak plane a tangent was drawn. In the left area beside the tangent, curvity radius R_{in} is very small while $\bar{U}_{\max}$ is also very small. In the right area, R_{out} is very large and $\bar{U}_{\max}$ becomes very large also. If the weak plane is a horn of a star structure, of course, a stress

concentration beside the star horn will be formed under the limit state.

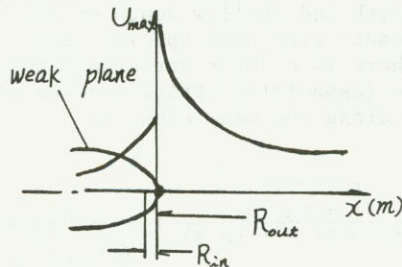


Fig. 12. The distribution of bursting pressure around a weak plane.

CONCLUSIONS

1. Bursting pressure is the mechanical source of bursting damage, sound, vibration and throwing out. Its innate character may be expressed by the formula

$$\bar{U}_{\max}|_{t=t_0} = \left(\frac{\partial \sigma}{\partial x} i + \frac{\partial \sigma}{\partial y} j + \frac{\partial \sigma}{\partial z} k \right)_{\max}|_{t=t_0}$$

Bursting pressure is the necessary factor for producing bursting. The limit value of energy ratio is:

$$G_{\max}|_{t=t_0} = - \bar{U}_{\max} \cdot \bar{V}_{\max}|_{t=t_0}$$

It is the necessary and sufficient factor for producing bursting.

2. The internal relationship between bursting phenomenon and non-bursting one has been described. One can turn into the other if the following condition is satisfied

$$G_{\max}|_{t=t_0} = - \bar{U}_{\max} \cdot \bar{V}_{\max}|_{t=t_0} \frac{\text{non-bursting}}{\text{bursting}} G|_{t=t_0} = - \bar{U} \cdot \bar{V}|_{t=t_0}$$

The measures for preventing rock bursts are to make the equation react toward the right, i.e. to turn the bursting one into a non-bursting one, for instance, to liberate the bursting threat by inducing it to explode or something else.

3. The basic task of detection and prediction is to grasp the time, place and the values of the appearances of $\bar{U}_{\max}$ and $G_{\max}$. Otherwise, the preventive measures must be directed against and aimed at these factors.

4. The main features of bursting in Hunan Collieries are high frequency, great strength and shallow depth of the beginning burst, etc. They related closely with some special structures in bursted coal seams in which there is a "star structure" which made the curvity radius of a weak plane (especially stratification plane) change sharply. The distribution along the weak plane is

$$[\sigma] = \frac{r}{4} R \left[\frac{\sqrt{1 + \operatorname{tg}^2 Q}}{\operatorname{tg} Q (\operatorname{tg} Q + \operatorname{tg} \phi)} + \frac{2}{\sqrt{1 + \operatorname{tg}^2 Q}} \right]$$

$$U_{\max} = \frac{3}{8} r^2 \frac{R}{[\sigma]}$$

Having changes in radius R or tg Q suddenly, $[\sigma]$ and $U_{\max}$ also have corresponding changes. The bursting phenomenon produces immediately when the necessary and sufficient factor $G_{\max}$ appears.

ACKNOWLEDGMENT

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Chapter 65

Constraint - The Missing Variable in the Coal Burst Problem

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ABSTRACT

In this Bureau of Mines report, the authors present the results of laboratory tests on the burst proneness of coal. Many researchers have studied the violent breaking of large coal masses in underground mines, called bursts, but the causes remain uncertain. They have often assigned to geology and gas pressure the causative roles.

The objective of this work is to show that stress can produce bursts in many coals if constraint is necessary for pillar survival and is suddenly lost.

We tested coal samples from 15 mines, in 11 coal seams, in six states in the laboratory, and 13 were made to burst. These bursts were produced in several ways, but segmented steel platens were used in the tests reported. Two parameters analogous to the momentum and kinetic energy were used to rank the burst proneness of the coals tested. Tangential strains over the lower surface of the model coal pillar were measured to reveal a periodic slipping prior to the burst.

High-speed motion pictures (350 frames per second) reveal the burst behavior in slow motion.

We propose a modified Mohr-Coulomb stress-failure criterion to describe the burst behavior. When the horizontal constraint is suddenly lost with little change in the vertical stress, the Mohr stress circle increases in size dynamically, exceeding the static failure condition. When the inertia of the coal mass is overcome, a burst occurs. We conclude that most coals can be made to burst and that such bursts can be expected in mining at increasing depths.

INTRODUCTION

The violent breaking of large coal masses underground by stress or gas, variously called bursts, bumps, outbursts, or gas outbursts, in which miners are injured or killed, is becoming a more serious problem as mining depth is increased. Researchers have made a large effort to understand, anticipate, and control these events, but the causes of the problem are still not well understood.

Our objective is to show that stress can produce bursts in many coals from across the United States if constraint is necessary for pillar survival and it is suddenly lost.

Some awareness of the role of constraint exists in the literature. Holland (1955) concluded that small pillars store energy as an unconstrained body. Large pillars store energy relatively close to the free face (within four to six times the seam thickness) and not in the central constrained core. If this energy zone is split or slabbed, a severe burst may result.

Holland (1958) wrote that a pillar larger than 150 feet could not fail abruptly because of the friction between the roof and floor or within the pillar itself. For smaller pillars, a sudden decrease in friction permits an instantaneous decrease in the constraining force, thereby permitting a pillar burst to occur. Brown stains on the roof, common to all burst areas, indicate relative motion between the coal and top rock.

Belin (1964) reported that U.S.S.R. researchers proposed a four-stage theory of outbursts: 1) elastic energy crushes the coal, 2) free gas ejects the coal, 3) desorbed gas propels the coal, and 4) the outburst ends when the friable part of the coal is expelled.

Farmer and Pooley (1967) reported, "An outburst is a two-phase phenomenon, triggered by the dynamic failure of the outburst coal during the change from confined to unconfined stress conditions in the face of an advancing working excavation, and extended by the subsequent desorption of gas contained in the coal structure." The static Mohr-Coulomb model was used for the stress contribution to the burst.

Brauner, Burgert, and Lippman (1976) produced a complex theoretical model of a coal burst in which 79 equations were used. They considered the role of friction in this work.

Shepard, Rixon, and Griffiths (1981) reported that, "Based on a study of the literature (116 references) and recent geological studies of outburst sites, geological structures have been identified as a prime factor in outburst occurrence." This view is typical of many researchers where geology is assigned the causative role.

RESEARCH PLAN

We obtained coal samples from 15 mines, in 11 coal seams in six states; Alabama, Colorado, Illinois, Pennsylvania, Utah, and West Virginia. Model coal pillars with width-to-height ratios of 8.5:1 (54 mm diameter x 6.4 mm high) were cut from diamond drill cores. The ends were ground parallel to within 0.005 mm. The model pillars were loaded in the compression machine shown in Fig. 1. A fragmented steel platen used to create the burst is shown in Fig. 2. The segmented steel platen constrained the top, while the plexiglas platen attempted to expand laterally at the same rate as the coal. When the vertical stress exceeded the unconfined coal strength, the model coal pillar survival depended upon the additional strength provided by the constraint. When the constraint was lost through slipping measured at the lower pillar-plexiglas interface, the bursting resulted.

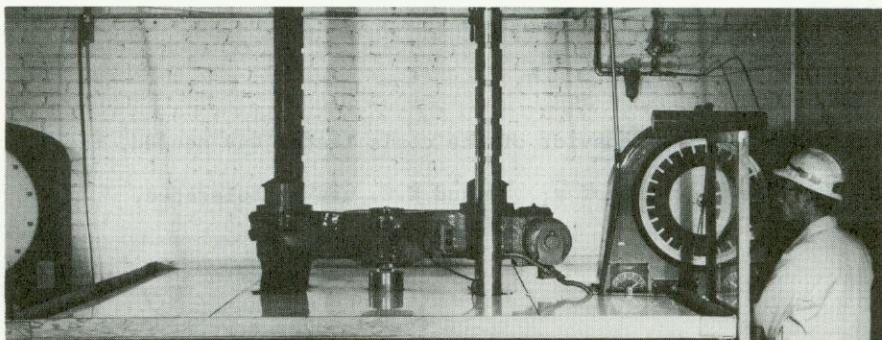


Fig. 1. Compression machine with 2.4-meter-square horizontal surface for model coal pillar burst tests.

A 2.4-meter-square horizontal surface about 9 cm below the burst elevation collected the burst debris. The vector distances the segmented steel platen pieces were thrown were recorded using a polar coordinate grid marked on the surface. The horizontal distances were used to estimate the horizontal velocities of the segments. These were estimates because the segments bounced across the surface. Two parameters analogous to momentum and kinetic energy, called M^* and $K.E.^*$, respectively, were used to characterize the bursts. Four to eight replications were used for each coal for a total of 84 tests. High-speed photography was used to record the burst behavior for slow-motion study.

THEORY

The momentum and kinetic energy can be used to estimate the skewness and magnitude of a coal burst. These are

$$\bar{M} = \sum_{i=1}^n dm_i \bar{v}_i, \quad (1)$$

and

$$K.E. = \sum_{i=1}^n dm_i \bar{v}_i^2 / 2 \quad (2)$$

where dm_i and $\bar{v}_i$ are the mass and velocity, respectively, of the i th mass thrown by the burst. The velocity is estimated from the

distance $\bar{d}_i$ the segment is thrown during the time it takes to fall 9 cm to the horizontal surface. The velocity estimate is therefore

$$\bar{v}_i^* = \bar{d}_i \sqrt{g / 2h_i}. \quad (3)$$

Since the relative behavior of the coals tested was needed, the error in the estimate of $\bar{v}_i$, $\bar{M}$, and K.E. can be tolerated.

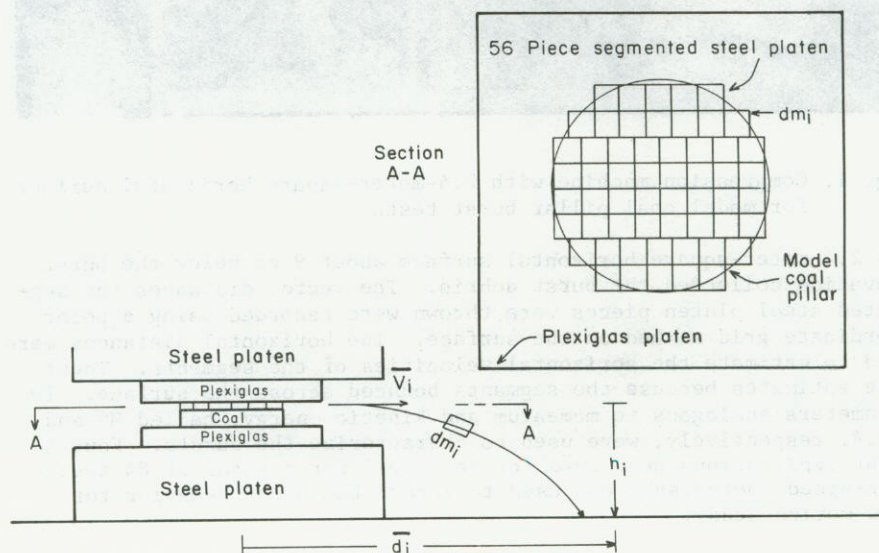


Fig. 2 Segmented steel platen used in tests to produce bursts and to provide a convenient method for estimating momentum and kinetic energy of bursts.

EXPERIMENTAL PROCEDURE

In some preliminary tests a layer of glass placed on the top of the model coal pillar constrained the pillar against horizontal expansion. With the dynamic breaking of the glass layer in tension into small pieces, the constraint was dynamically removed and a burst resulted. This strong layer is similar to the strong roof rock layer present in many mines that have coal bursts. Because the burst occurred after the breaking of the layer, a segmented steel platen was used in the tests reported to make the segment units all the same size so that the estimated values of momentum and kinetic energy were easier to measure. The load rate was 11 kN/s, and the breaking force ranged from 110 to 180 kN.

DATA COLLECTION AND ANALYSIS

Fig. 3 shows a typical frame of a model pillar burst photographed at 350 frames per second. The burst debris was distributed over an area 2,500 times the initial pillar area.

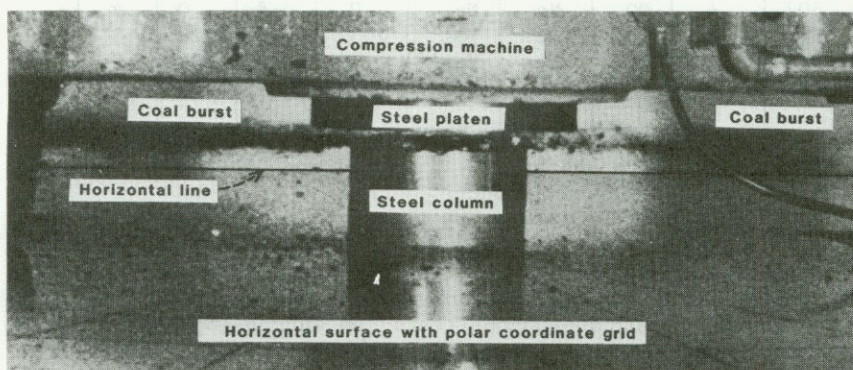


Fig.3. Model coal pillar burst produced in the laboratory and recorded with 350 frame per second photography.

The location of each segment thrown by the burst was recorded in polar coordinates from the horizontal surface of Fig. 1. These distances were used to estimate the velocities $\bar{v}_i^*$ of the thrown segments, to estimate the momentum $\bar{M}^*$ and the kinetic energy K.E.* of the bursts, and to rank the coals tested as to their relative burst proneness. Table 1 summarizes the laboratory results. If a coal has a high rank for momentum, a nonsymmetrical radial bursting pattern is indicated. This directional bursting of coals was more pronounced for western coals than for the other coals tested. In these coals it may be possible to select a mining direction that will reduce the burst hazard.

TABLE 1. Summary of Test Results and Burst Proneness Rank

Depth, meters	Mine Number	State	Mine Bursts Now	Model Bursts	K.E.*, milli- Joules	K.E.* Burst Rank	* $\bar{M}$, gm cm /sec	* $\bar{M}$ Burst Rank
61	14	PA	No	Yes	223	5	958	7
85	10	IL	No	Yes	38	11	355	11
91	1	UT	No	Yes	164	8	1268	4
91	3	CO	No	Yes	69	10	719	10
91	9	CO	No	Yes	259	3	741	9
183	7	CO	No	Yes	281	1	1739	2
253	13	WVA	No	Yes	1	13	905	8
274	12	WVA	No	Yes	273	2	65	13
274	8	CO	No	Yes	124	9	1018	6
344	11	WVA	Yes	No	0	#	0	#
427	2	UT	Yes	Yes	214	6	1177	5
503	4	CO	No	No	0	#	0	#
541	6	CO	Yes	Yes	203	7	1328	3
671	15	AL	Yes	Yes**	1	12	194	12
853	5	C	Yes	Yes	237	4	2300	1

* Estimates of M and K.E.

** Weak bursts sometimes occur.

Did not burst in laboratory tests.

To define the change in end constraint on the lower surface of the model coal pillar, a special strain gage was constructed and used. Fig. 4 shows the results from one test. An increasing load produced a periodic slipping at the lower pillar interface. The tangential strain measured remained between two limit lines called unstable and stable limit, respectively.

Modified Mohr-Coulomb Failure Mode

Workers in the rock mechanics field often use the Mohr-Coulomb stress-failure criterion. In this criterion, if the stress circle touches the failure line, failure occurs. This is a static model in that the circle cannot exceed the failure line. Some researchers believe that the increase in circle size results from an increase in the vertical pillar stress. This produces the failure condition shown in Fig. 5. We believe that in the modified theory shown in Fig. 6 a decrease in the horizontal constraint stress increases the circle size dynamically, causing the stress circle to exceed the static failure condition. When the inertia of the mass is overcome with time, dynamic bursting occurs. In Fig. 6, α and C are the angle of internal friction and the strength in direct shear, respectively.

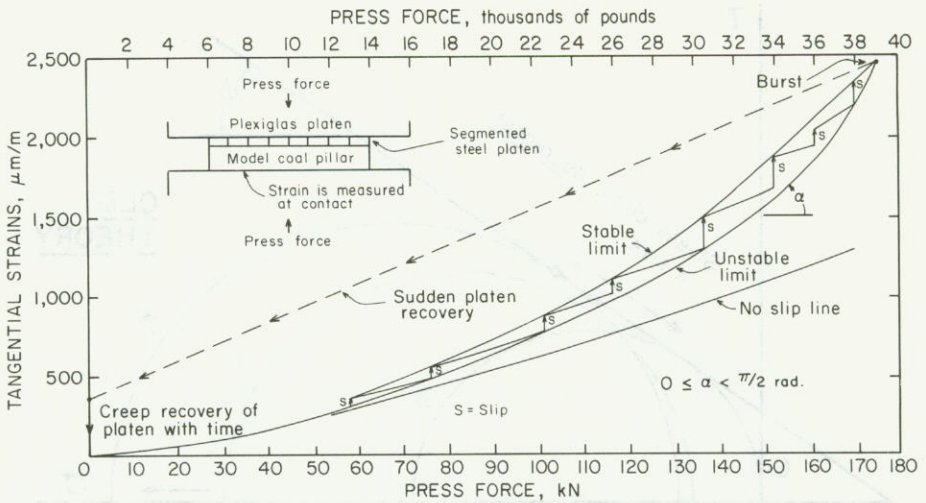


Fig. 4 Tangential strains showing slipping of the lower plexiglas platen.

SUMMARY AND CONCLUSIONS

We believe that many, if not most, coals can be made to burst given the necessary conditions of stress and constraint. In cases where the strength is largely produced by constraint, the sudden loss of this constraint can produce bursting.

While the work presented is based on laboratory results, it seems likely that similar behavior can occur in coal mines to produce bursts. It is important to distinguish between the strength of constrained and unconstrained coal. The constraint can be lost dynamically to produce bursts, while the unconstrained strength cannot be lost in this way. In a mine, a strong roof rock can intermittently constrain the coal seam with frictional sliding or periodic deformation of a soft member between the rock and coal producing the burst.

Strain energy can produce bursts without the help of gas pressure.

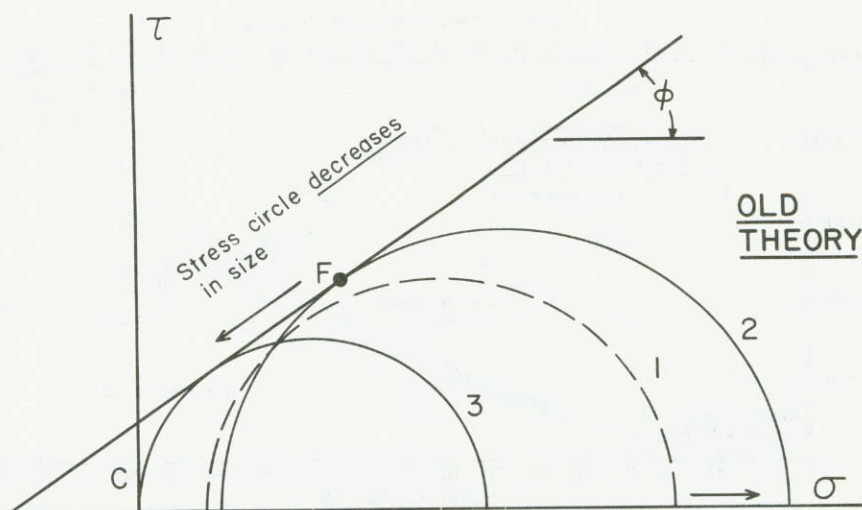


Fig. 5. Prior art does not explain bursts. Vertical stress increases from Mohr circle state 1 to state 2. Static failure occurs at point F. Progressive static failure state 3 follows failure line.

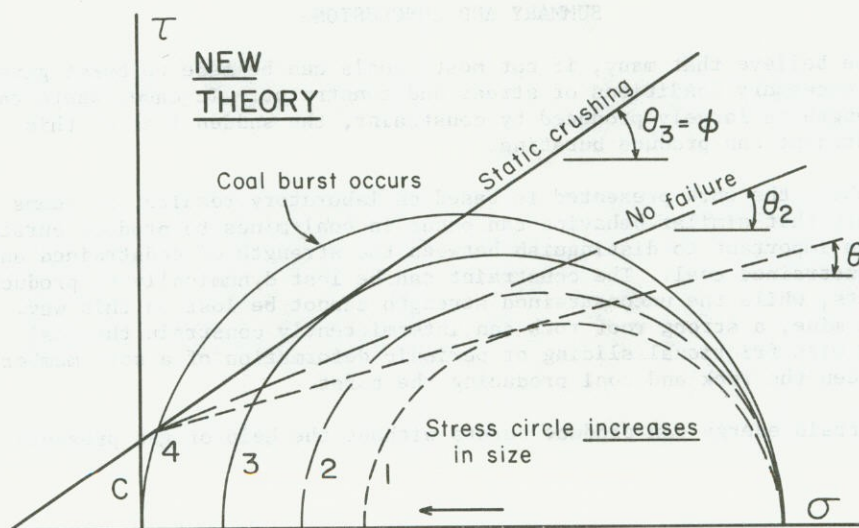


Fig. 6. New theory describes burst behavior. Horizontal stress decreases from 1 to state 2, 3, or 4. The angle of internal friction required for stability increases in size. If the mohr circle dynamically exceeds the failure condition a burst occurs.

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Appendix

1. The first part of the report is a general introduction to the subject of the study. It discusses the importance of the study and the objectives of the research.

2. The second part of the report is a detailed description of the methodology used in the study. It includes information about the sample, the data collection methods, and the statistical analysis.

3. The third part of the report is a discussion of the results of the study. It presents the findings of the research and discusses their implications for the field of study.

4. The fourth part of the report is a conclusion and a list of references. The conclusion summarizes the main findings of the study, and the references list the sources of information used in the research.

5. The fifth part of the report is a list of appendices. These appendices contain additional information that is relevant to the study but is not included in the main body of the report.

6. The sixth part of the report is a list of figures and tables. These figures and tables provide a visual representation of the data and results of the study.



14

LONGWALL MINING GROUND CONTROL

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14

LONGWALL MINING GROUND CONTROL

by

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SYNOPSIS

This paper discusses the various methods of ground control in longwall mining, including the use of hydraulic supports, roof bolting, and caving.

Chapter 66

Prediction of Inter-Strata Movements Above Longwall Faces

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ABSTRACT

This paper presents an analysis of measurements of inter-burden deformations above six longwall faces. An attempt is made to demonstrate some correlation between the movements at the various sites, and to examine their importance with respect to predicting caving height, disruption of overlying seams, and disruption of aquifers. This analysis demonstrates some significant differences between predicted surface subsidence, and inter-burden deformation. In addition it is shown that the caving height above a longwall face is equal to 8 to 12 times the extraction height, with a zone of fractured rock extending to approximately 50 times the extraction height above the seam.

INTRODUCTION

As the easily accessible seams are exhausted, mining companies will be forced into extracting deeper, underlying seams. Due to increased depth, lower seams could well be extracted by the longwall method. To ensure the optimum longwall layout, and to minimize interaction effects, a better understanding of strata deformation above longwall faces is required. This is essential in areas of high extraction, so as to minimize surface subsidence, aquifer disruption, and interaction between superincumbent workings. Over the past twenty years this problem has attracted the attention of many investigators, however, due to the high cost of inter-burden instrumentation programs, the majority of these investigations were confined to surface and in-mine measurements.

This paper presents an analysis of measurements of inter-burden deformations above 6 longwall faces, 5 in the United Kingdom, and 1 in the U.S.A. Details of the various sites are summarized in table 1, along with a summary of the instrumentation utilized. A previous paper by Styler and Dunham described the inter-strata movements, and arrived at the following conclusions:

- No significant vertical movements were measured in boreholes located above, and near to the ribside, indicating that gateroad deformation is mainly caused by localized effects.
- There was a significant difference between strata behavior in the British and American sites. This was due to the behavior of the strata in the American case histories being dominated by strong/competent beds, whilst in the British case histories, there were no strong beds to have a noticeable effect on the movements above the face.
- Empirical methods of subsidence prediction, such as the Subsidence Engineers Handbook (S.E.H.), cannot be used for both countries without a more complete understanding of the influence of geology.
- In order to model deformations above a longwall face, an area of fractured rock must be included, the extent of which is dependent upon the presence of any competent beds in the immediate roof.

	Lynemouth		Whitburn	Markham	Vane Tempest	Somerset Mine No 60
Seam	Brass Thill	Brass Thill	Harvey	Three Quarter	Low Main	Pittsburgh
Face	KS6	KS4	NW6	L58	3rd West	Panel 2
Depth (m)	175	175	463	585	640	130
Extraction Thickness (m)	1.52	1.6	1.06	1.0	1.07	1.7
Face Width (m)	180	180	182	200	140	138
Width to Depth Ratio	1.03	1.03	0.39	0.34	0.22	1.41
Type of Instrument	Magnetic Anchors	Anchor Wire	Anchor Wire	Anchor Wire	Anchor Wire	Anchor Wire
Height of Instruments above Seam (m)	Max	75	75	69	71	145
	Min	7	8	23	7	38
No of Instrumented Levels	5	4	6	16	8	14
No of Boreholes	4	2	2	5	8	2
Reference	Styler	Hodkin	Malone	Batchelor	Dowdell	Barla

Table 1. Site Details.

MAXIMUM SUBSIDENCE/HEIGHT ABOVE THE SEAM

The absolute maximum anchor displacements have been plotted as a percentage of the extraction thickness against height above the seam, figure 1. It was only possible to plot the results from the British case histories, since only relative, and not absolute movements were reported for the American investigation. The plotted results were all obtained from boreholes near to the face centerline, and are therefore, comparable. As would be expected subsidence decreased with height above the seam, and the rate of reduction in subsidence also reduces with increasing height above the seam. For comparison a curve showing the maximum predicted surface subsidence using the S.E.H., has also been plotted on the figure 1. This curve was drawn for a face width of 170 meters, which is the average face width for the sites considered.

The only measured surface subsidence plotted on figure 1, was recorded at Whitburn, 463 meters above the seam. The measured surface settlements were between 37% and 51% of the extraction height, whilst the predicted surface subsidence at this level is 31% of the seam thickness. However as can be seen from figure 1, there is a much larger discrepancy between the predicted subsidence, and the measured inter-burden deformations. At 70 meters above the face at Whitburn the inter strata subsidence is between 45% and 65% of the extraction height. This demonstrates that the majority of the decrease in subsidence occurs in the near seam strata, and above this the strata behave in a competent manner, with very little dilation occurring. This is contrary to what would be expected, with the weight of the overlying strata causing greater compaction of the gob, and therefore causing the inter-burden deformations

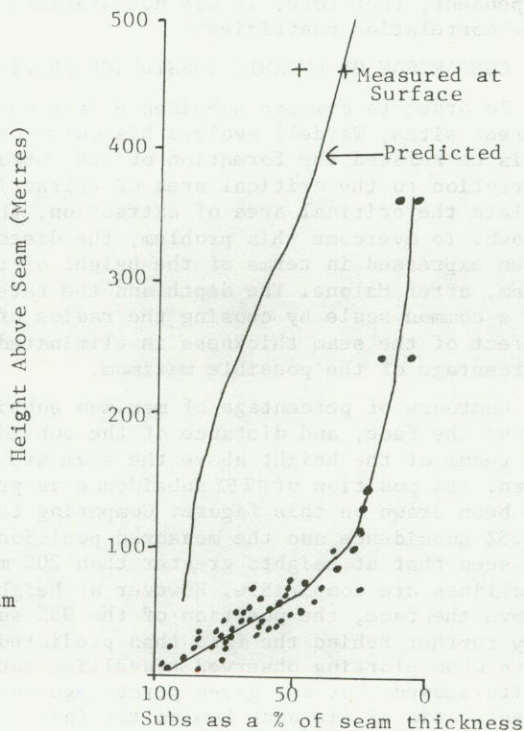


Figure 1:Subsidence/Height

to be larger than the predicted surface subsidence at the same level.

The data shown in figure 1 was analyzed statistically, to calculate the correlation between subsidence (S), face width (W), and the height of the measuring level above the seam (Z). For a total of 51 data points the correlation coefficient between these variables was found to be 0.95, indicating that the subsidence is significantly dependent on the width to height ratio. A polynomial regression analysis using this data was conducted, giving the equation shown below, plus or minus one standard error.

$$\log_e(\%S) = (3.025 + 0.489) + (1 + 0.057) \log_e(W/Z) - ((0.168 + 0.02) (\log_e(W/Z))^2)$$

This equation gives a wide range of predicted values, with its accuracy decreasing with increasing width to height ratio. A similar analysis was carried out to see if the subsidence was dependent on the amount of cover above the measuring level, in addition to the width to depth ratio. This analysis gave a wide scatter of data, which was site dependent, therefore, it was not statistically accurate to calculate the correlation coefficient.

COMPARISON OF DYNAMIC SUBSIDENCE PROFILES AT DIFFERENT HEIGHTS

In order to compare subsidence data on a common basis from different sites, Wardell evolved his curve of subsidence development. In this he related the formation of the subsidence curve in the dynamic direction to the critical area of extraction. However in order to calculate the critical area of extraction, the angle of draw has to be known. To overcome this problem, the distance from the face line has been expressed in terms of the height of the measuring point above the seam, after Malone. The depth and the rate of face advance are reduced to a common scale by choosing the radius of influence as abscissae. The effect of the seam thickness is eliminated by plotting subsidence as a percentage of the possible maximum.

Contours of percentage of maximum subsidence plotted against height above the face, and distance of the subsiding point from the face line in terms of the height above the seam are shown in figure 2, for Whitburn. The position of 97.5% subsidence as predicted from the SEH has also been drawn on this figure. Comparing the predicted position of 97.5% subsidence and the measured position of 95% subsidence, it can be seen that at heights greater than 200 meters above the face, the two lines are comparable. However at heights of less than 200 meters above the face, the position of the 95% subsidence line is considerably further behind the face than predicted. Shadbolt presented similar data when plotting observed travelling subsidence profiles. These results showed that any given percentage subsidence value was at a constant ratio of distance behind the face line in terms of depth, for working at depths of greater than 300 meters. Whilst for depths less than 300 meters he concluded that the movements appear to migrate towards the excavated void. He also mentioned a similar depth related change in horizontal strains for depths of less than 300 meters.

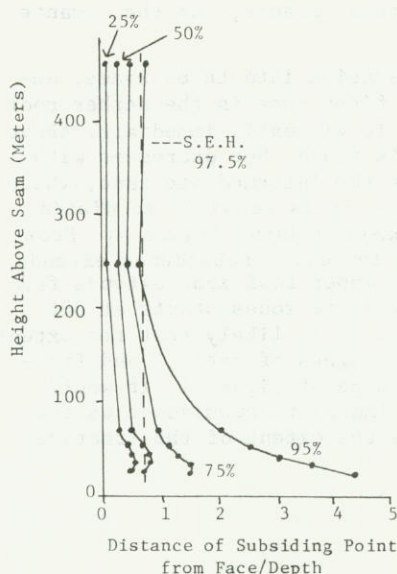


Figure 2. Contours of Inter-burden Subsidence at Whitburn

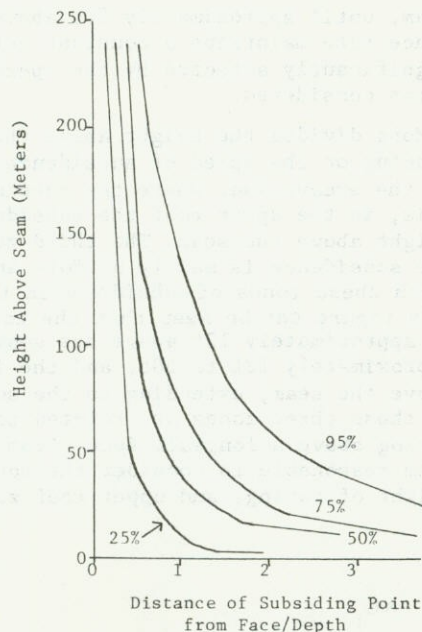


Figure 3. Contours of Average Inter-burden Subsidence.

The data shown in figure 2 has been combined with similar data from Somerset Mine 60, Vane Tempest, and Markham. This data indicated a reasonable agreement between the movements at the various sites. From these results, figure 3 has been drawn, showing the average contours of percentage interburden subsidence. This diagram can be used to obtain an estimate of the dynamic inter-strata subsidence, expressed as a percentage of the maximum subsidence. The ability to predict dynamic subsidence is essential in determining interaction effects. However, the applicability of this diagram needs confirmation over a larger range of mining geometries and geologic environments.

MAXIMUM SUBSIDENCE RATE

The rate of subsidence is dependent on various factors, which complicates comparison of measurements from various sites. These factors include the rate of face advance, the mining thickness, height above the seam, face width, and distance of the measuring point from the ribside. The maximum subsidence rates have been calculated by finding the maximum slopes on the graphs of anchor displacement against face advance. These maximum subsidence rates in mm/m of face advance, are plotted against height above the seam in terms of the extraction thickness (t) in figure 4. It can be seen from this figure that the

general trend is a decrease in subsidence rate with height above the seam, until approximately 50t above the seam, after which the subsidence rate maintains a constant value. The subsidence rates were not significantly affected by the speed of face advance, for the advance rates considered.

Mohr divided the height above the excavation into three zones, depending on the speed of subsidence. The first zone is the nether roof of the excavation, where the subsidence is virtually immediate. Above this, in the upper roof the subsidence is rapid, but decreases with height above the seam. The third zone is the intermediate zone, where the subsidence is mainly uniform and slow. It is possible to distinguish these zones of subsidence in the measured data, figure 4. From this figure can be seen that the zone of immediate subsidence extends to approximately 12t above the seam. The upper roof zone extends from approximately 12t to 50t, and the intermediate zones starts at 50t above the seam, extending to the surface. It is likely that the extent of these three zones are related to the heights of caving, and fracturing above a longwall face. From the shape of figure 4, it would seem reasonable to consider the zone of immediate subsidence as the height of caving, and upper roof zone as the extent of the fracture

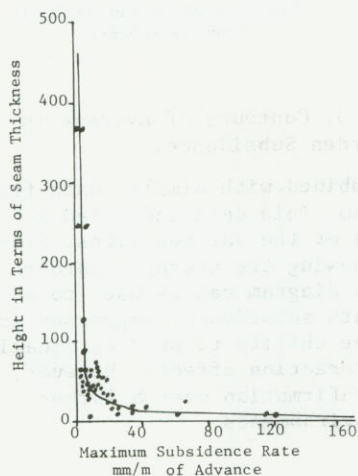


Figure 4. Subsidence Rate versus Height Above the Seam.

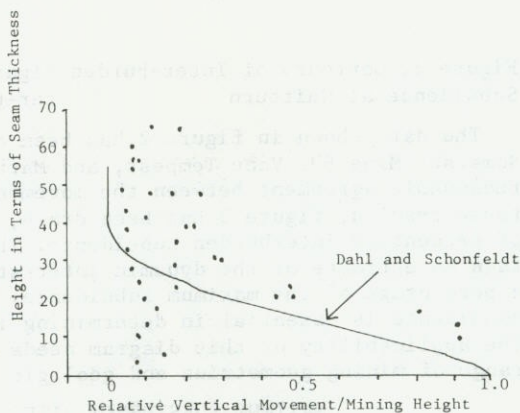


Figure 5. Relative Movements Versus Height Above the Seam.

zone above a longwall face. This area will be the zone of increased permeability above the face, and hence represents the minimum amount of cover when working below aquifers. The affects of bed separations, and fractures on permeability have been observed in the USSR, and New South Wales, as reported by Wardell. In the former case increased permeability was observed at 15 times the extraction height, and in the latter at 35 times the extraction height above the longwall face. It must be emphasized that the height of this fracture zone is not equal to the minimum cover required beneath a surface body of water, due to the zones of fracture caused by surface horizontal strains.

RELATIVE MOVEMENTS ABOVE FACES

Measurements of the elongation of a borehole due to caving of the overburden above a longwall panel, were reported by Dahl and Schondfeldt. The hole was drilled prior to mining, the extraction height was 1.67m, and the average overburden thickness was 135m. The relative deformations were plotted in terms of the mining height against height above the seam, also expressed in terms of the mining height, figure 5. The vertical deformations relative to the surface have been plotted on the same figure for the Whitburn and Somerset Mine 60 case histories. It can be seen that most of the data follows the same trend as observed by Dahl and Schondfeldt. The relative movements decreased from a maximum close to the seam level, to a small uniform value at a height of approximately 50 times the mining height above the seam. The one exception to this are the results recorded in one of the boreholes at Somerset Mine 60. The original anchor movements for this borehole presented by Barla and Boshkov indicated that all the anchors underwent relative small movements, and that these movements were parallel to each other. This sort of behavior suggests a near rigid block movement for the roof strata, probably with very little expansion of the caved material. This could be due to a combination of shallow depth, and competent strata. If the results from this borehole are ignored, it can be seen that the rest of the results follow the same general trend. The plot of maximum subsidence rate against height above the seam, figure 4, showed that the height of the fracture zone extends to approximately 50 times the extraction thickness above the face. The relative movements plotted in figure 5 agree with this conclusion, with the height of fracture being between 35 and 50 times the seam thickness.

CONCLUSIONS

From the results reported in this paper, and by Styler and Dunham, the following pattern of strata deformation above longwall faces is indicated:

As the face advances, failure of the immediate roof is induced, and a zone of caving is formed. The higher strata behave as a series of slumping beams, the abutments of which form an arch above the gob. The curvature of these beams decreases with height above the face, even-

tually reaching a point where a beam remains continuous, and rests on the caved material. This occurs at a height of approximately 12 times the seam thickness above the face. The strata above this zone behave as a series of continuous beams, with some bed separation and fracturing. This dilation decreases with height above the seam, and extends to approximately 50 times the extracted thickness above the face. As the face advances, the weight on the gob increases, causing partial consolidation, which is complete between 100 meters and 150 meters behind the face. There is some residual expansion of the strata, which causes a reduction in the volume of subsidence trough with height above the seam.

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Chapter 67

CONTROLLED LOAD DEVELOPMENT AT LONGWALL COALFACES

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ABSTRACT

The effectiveness of coalface powered support depends upon its structural nature and the manner of application of the hydraulic system. A critical and largely unresolved feature of the system is the optimum setting pressure level at which a support should operate in its geological environment.

This paper provides an argument for a generalised approach to the issue of setting pressure level quoting case study data for a high output British longwall installation.

INTRODUCTION

The uniqueness of each mining location brings into question the validity of a generalised approach to mine design. Despite the wide range of currently available hydraulic supports, it may be argued that strata control at the coalface has successfully been standardised. Design criteria have been produced such that a wide range of geological and mining environments experience controlled strata yield. The present generation of coalface support forms part of an evolutionary process commenced between 1930-1950, and which has since developed rapidly as a result of technological advances in all the major branches of engineering.

The general level of efficiency of mining systems is high with regard to manpower, machine utilisation, and mine layout. A persistent number of low efficiency powered support installations operate in conditions of large roof and floor debris layers, inadequate support and roof bed fracturing, and slow face advance - a situation that can be accurately described as a 'vicious' cycle.

FIELD INVESTIGATIONS

South Wales Coalfield

Research at Cardiff between 1972-1981 at numerous locations in the South Wales Coalfield, confirmed a characteristic loading and convergence pattern for the cycle of extractive operations in which support advance/reset appeared as the major influence upon roof-floor convergence.

The introduction in 1981 of a transducerised data logging system for data acquisition at coalface investigations of support and strata behaviour (Isaac et al, 1982), enabled monitoring of various parameters to be virtually continuous in nature. The pattern of loading and convergence was observed to be similar as before, except that a pressure decay effect was observed in the early stages of support load development. This was explained as the effect of roof and floor debris compaction at support reset. For the relatively low setting levels of British installations at between 10-15 MN/m², this period of pressure decay was accompanied by a high rate of convergence with only a small amount of leg closure. This behaviour suggests uncontrolled load development at the critical period of support advance/reset in the extractive cycle. The implication of this condition is that roof bed strata would suffer induced fracturing, a greater amount of roof-floor convergence would occur, and that powered support performance as expressed by the Pressure/Convergence criterion (Smart and Isaac, 1982) would be less efficient.

Nottinghamshire Coalfield

Investigations at Bilsthorpe Colliery, North Notts. Area, National Coal Board, in June 1982 and April 1983 (Isaac and Bradbury, 1983) provided the opportunity of assessing Gullick Dobson chock-shield supports. The initial study of 4 leg/260 tonne supports was conducted at 23's Face in the Parkgate Seam (1.8m thick, shallow dipping), the second study being performed on 4 leg/280 tonne supports fitted with 'positive set' valves, at 15's Face in the same seam. The positive set valves acting at normal supply line pressure, and triggered individually for each pair of legs at approximately 7.6 MPa, were intended to ensure a uniform setting pressure along the faceline of approximately 15 MPa. Information generated by this investigation included:

- Load distribution along the faceline
- Setting pressures
- Mean setting load densities
- Hydraulic leg pressures
- Base loads
- Mean Load Densities
- Strata convergence/Debris compaction
- Pressure/convergence ratio

The work described in this paper is intended to identify support design and performance that produces a 'kind' cycle of face conditions from which optimum output may be achieved.

SUPPORT LOADING CONSIDERATIONS

Powered support development may be viewed as essentially qualitative in its first phase, 1930-1960, and rather less so in its second, 1960-1980. Design criteria based principally upon field experience related to:

Setting Load/Yield Load

Roof to Floor Convergence

Differential Lateral Movement of Roof and Floor

Support Structural Strength

Waste Caving

Research investigations since 1965 have attempted to rationalise one of the more important of these considerations, viz., support loading. Two previous approaches to estimation of support loading requirements were based upon.

- (i) in-situ measurement of support loads (Ashwin et al, 1970);
- (ii) estimation of the largest volume of rock requiring support for a given face situation (Wilson, 1975).

From in-situ measurement of support loading, it was shown that for Mean Support Load Densities above 0.1 MN/m^2 , there was no substantial reduction in roof to floor convergence, and for values below 0.05 MN/m^2 such convergence increased with an unacceptable loss of strata control (Wilson, 1964). It was concluded that using the approach of in-situ measurement of support resistance for design purposes, Setting Load Densities should equal Mean Load Densities. Using the alternative approach of estimation of rock requiring support, it was concluded that support design should be at least twice minimum Mean Load Density.

Subsequent research (Bates, 1978; Gupta, 1982) has shown that using either approach to support loading, roof-floor convergence can be reduced by higher setting load levels without necessarily creating difficulties in friable roofs and soft floor conditions. This implies that strata yield is a function of setting load level.

Analysis of Results

Results from 23's Face confirmed those obtained in past investigations and showed that for a period lasting up to 3 minutes after support reset, leg pressures reduced by up to 30% of their initial setting value (Fig. 1(a)). It can be seen from Fig. 2(a) that during the period of pressure decay, 8 mm of convergence was required before the stiffness of debris approached that of the hydraulic legs. This uncontrolled lowering of the roof was instrumental in the development of the 'vicious' cycle referred to in Table 1.

TABLE 1. Elements of Comparative Cycles of Extraction

'Vicious' Cycle		'Kind' Cycle	
Element	Value	Element	Value
1. Potential rate of production	10,000 T/week	1. Potential rate of	10,000 T/week
2. Debris production around supports, from conveyor and waste flushing	150 mm*	2. Debris production around supports, from conveyor and waste flushing	150 mm*
3. Variable leg closure/debris compaction from operator-controlled, low setting pressure	-	3. Uniform leg closure/debris compaction from positive set facility	-
4. Reduced strata control, e.g. roof bed separation	8 mm	4. Controlled strata movement, e.g. roof bed separation	0 mm
5. Further debris production around/between supports	+160 mm*	5. Cleaner face conditions	+0 mm*
6. Lower rate of production	6,500 T/week †	6. Actual rate of production	10,000 T/week

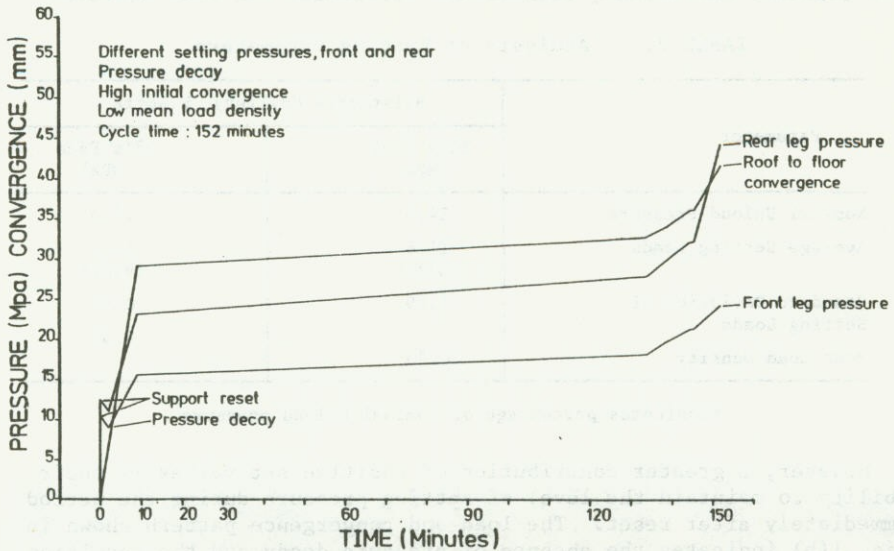
* refers to total roof and floor debris produced per cycle

† an estimate of production that would have been achieved, given that the only difficulties experienced were those of strata control

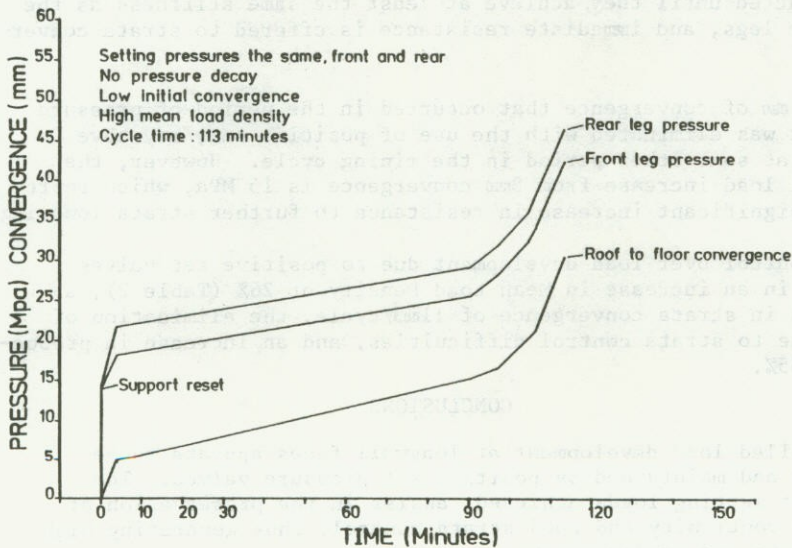
Analysis of the 'vicious' cycle indicates that excessive thickness of debris layers adversely influences the quality of support and increases cycle duration.

The best means of controlling load development and creating the 'kind' cycle referred to in Table 1, is to reduce the effects of debris layers above and below the supports and eliminate further debris production by maintaining roof beam integrity. This may be achieved by use of 'positive set' facilities which improve the situation at the critical time of support advance/reset.

A comparison of results in Table 2 illustrates the effectiveness of 'positive set' in achieving designed setting pressures. Average setting loads have been increased from 11.6 to 14.1 MPa representing



(a) Manually Set Supports, 23's Face



(b) Positive Set Supports, 15's Face

Fig. 1 Support Leg Pressure and Roof-Floor Convergence for Average Cycles of Extraction, Bilsthorpe Colliery, U.K.

an improvement of 11% in support setting efficiency, and as shown by the standard deviations, significant uniformity has been achieved.

TABLE 2. Analysis of Loading Parameters

Parameter	Bilsthorpe Colliery, N.Notts	
	23's Face (MPa)	15's Face (MPa)
Nominal Unload Pressure	14.0	15.0
Average Setting Loads	11.6 (83%)*	14.1 (94%)*
Standard Deviation of Setting Loads	1.69	0.45
Mean Load Density	0.23	0.31

*indicates percentage of nominal unload pressure

However, a greater contribution of positive set valves is their ability to maintain the level of setting pressure during the period immediately after reset. The load and convergence pattern shown in Fig. 1(b) indicates the absence of pressure decay and the resulting lower initial convergence. Fig. 2(b) shows that by maintaining the level of setting pressure, debris layers above and below the support are compacted until they achieve at least the same stiffness as the hydraulic legs, and immediate resistance is offered to strata convergence.

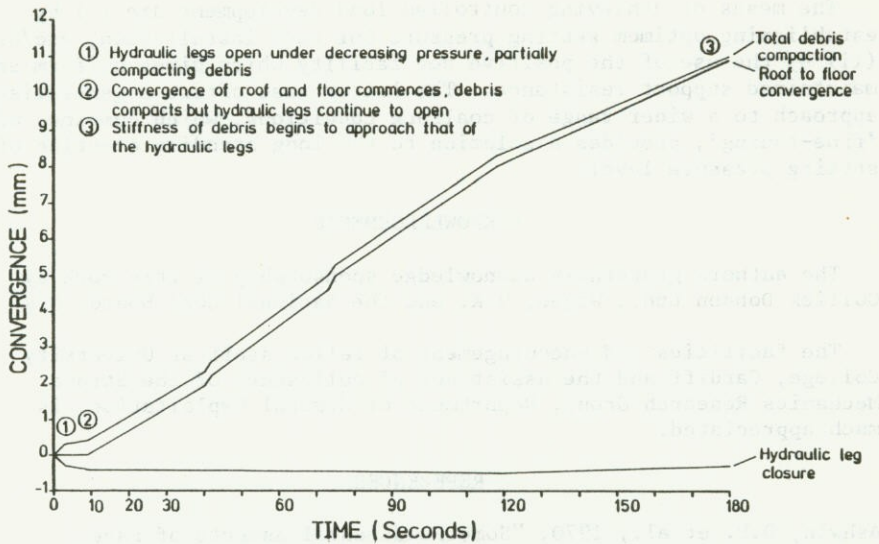
The 8 mm of convergence that occurred in the period of pressure decay but was eliminated with the use of positive set, may have occurred at some other period in the mining cycle. However, the potential load increase from 8mm convergence is 15 MPa, which represents a significant increase in resistance to further strata lowering.

The control over load development due to positive set valves resulted in an increase in Mean Load Density of 26% (Table 2), a reduction in strata convergence of 11mm/cycle, the elimination of delays due to strata control difficulties, and an increase in production of 35%.

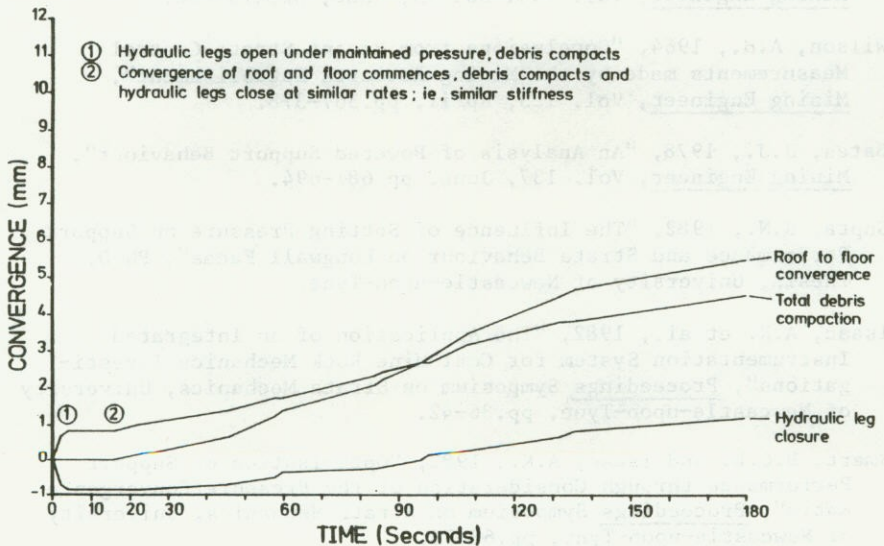
CONCLUSIONS

Controlled load development at longwall faces appears to be initiated and maintained by positive set pressure valves. The consistent setting loads achieved, assist in the preservation of roof beam continuity and good strata control, thus generating high face advance and productivity.

Recent results from other installations confirm the conclusions made with respect to the Bilsthorpe Colliery studies.



(a) Manually Set Supports, 23's Face



(b) Positive Set Supports, 15's Face

Fig. 2 Roof-Floor Convergence, Debris Compaction and Hydraulic Leg Closure after Support Reset, Bilsthorpe Colliery, U.K.

The means of achieving controlled load development are (i) by establishing optimum setting pressure for each installation, and/or (ii) by the use of the positive set facility which gives uniform and maintained support resistance. The latter case offers a generalised approach to a wider range of coalface conditions, which combined with 'fine-tuning', provides a solution to the long standing question of setting pressure level.

ACKNOWLEDGEMENTS

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Chapter 68

TECHNIQUE TO MEASURE RESULTANT LOAD VECTOR ON SHIELD SUPPORTS

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ABSTRACT

The resultant load vector is the representation of the forces applied to a longwall roof support element by caving strata into a single, quantifiable measure of support resistance. The relatively complex kinematics of the shield support prohibit a determination of support resistance simply by summation of leg forces. A method is being investigated by the Bureau of Mines to determine the resultant load vector by instrumenting supports with pressure transducers and strain gages to measure leg, canopy capsule, and lemniscate link forces. This concept has been laboratory tested in the Bureau's Mine Roof Simulator. Functional relationships among variables have been assessed, and confidence intervals have been established for prediction of the resultant load vector parameters.

RESULTANT LOAD VECTOR CONCEPT

Because of inadequate roof control and subsequent failures of several early longwall attempts in the United States, which utilized low-capacity European support equipment, there has been a tendency to increase support capacity with little regard to expected support loadings. Since the cost of a support is related to its capacity, use of excessively large supports represents an unnecessary capital investment and may cause unnecessary fracturing of the roof strata, thereby being detrimental to good roof control. Another major trend, which has occurred in American longwall mining practice during the past decade, has been the dominance of the shield support. The shield design, with its A-frame-type structure, offers the advantage of being able to resist horizontal loading with approximately 25 pct of the shield

structure dedicated solely to this purpose (Peng, 1978). Although horizontal loading is a primary design consideration, the degree of horizontal loading and its association with geological conditions is relatively unknown. Factors such as the uncertainty of horizontal and vertical loading demonstrate the need for additional in-mine support loading information to provide more effective design and utilization of longwall roof support systems.

The Bureau is conducting research to provide a better understanding of support loading and caving phenomena by developing a technique to measure the resultant load vector on shield supports. The resultant load vector is the representation of the forces acting upon a support element by a single, quantifiable measure of support resistance. Being a vector, this measure possesses not only a magnitude, but also spatial parameters of location and direction as depicted in Figure 1. Reference will be made to three resultant load vector parameters:

- Magnitude - Magnitude of loading.
- Location - Position of load vector on the canopy (as measured from canopy hinge pin).
- Angle - Inclination of load vector relative to the shield axis (plane) of symmetry measured in a clockwise direction, with 90° representing pure vertical loading, and angles less than 90° describing face-to-waste horizontal loading.

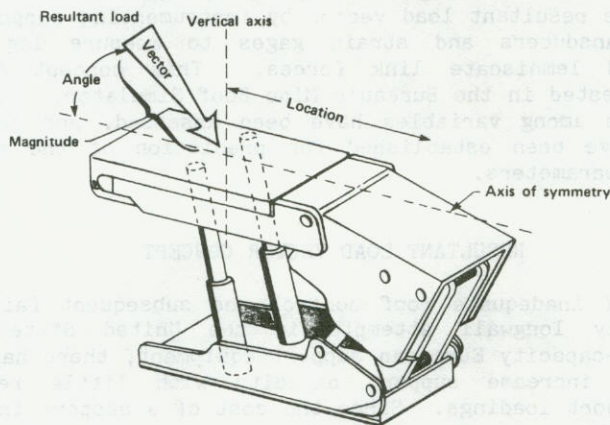


Figure 1. Resultant load vector.

From these parameters, critical design information can be ascertained. For example, by knowing the magnitude and angle of the resultant load vector, the magnitude of horizontal (face-to-waste) shield loading can be assessed. Likewise, insight

into caving behavior of the strata may be gained by examination of vector location as it moves forward or rearward during the mining cycle.

While it is recognized that resultant load vector information must be considered as baseline engineering data, the Bureau's Mine Roof Simulator (MRS) enables such information to be further analyzed under controlled laboratory conditions. The MRS is a massive bidirectional hydraulic press capable of applying 1,500 tons of vertical force and 800 tons of horizontal force, either independently or simultaneously, to full-scale roof support elements (Barczak et al., 1982). A load profile measured underground can be programmed into the MRS computer and simulated by the test rig. This enables comprehensive structural analysis of the support under controlled laboratory conditions and studies to evaluate such things as the effectiveness of the lemniscate design in resisting horizontal loading.

KINEMATICS OF SHIELD SUPPORTS

The relatively complex kinematics of the shield support prohibit a determination of support resistance simply from summation of leg forces as can be done with chock-type supports (Peng and Chyan, 1982). Static rigid body analysis of the shield structure reveals that resultant load vector parameters can be determined from the equations shown in table 1 if the leg, canopy capsule, and compression lemniscate link forces are known. Table 1 presents the relationships so developed for a 2- x 360-ton shield support when used in a 94-inch-high configuration. These are not general relationships, but they represent the shield geometry currently under field study by the Bureau. Similar equations, with different coefficients, will result for other geometries and other shields.

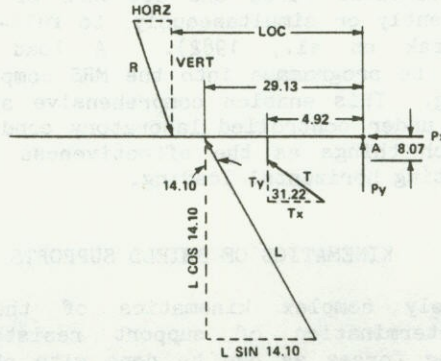
TABLE 1. Resultant Load Vector Parameter Equations

1. $HORZ = .2426 * L + .0912 * T + .3765 * C$
2. $VERT = .9699 * L - .0997 * T + .0038 * C$
3. $MAG = (HORZ^2 + VERT^2)^{1/2}$,
4. $LOC = \frac{28.2524 * L + 9.4515 * T}{VERT}$
5. $ANG = ARCTAN (VERT/HORZ)$,

where HORZ = Horizontal component of resultant magnitude, ton,
 VERT = Vertical component of resultant magnitude, ton,
 MAG = Resultant magnitude, ton,
 LOC = Resultant location, inches from canopy hinge,
 ANG = Resultant angle, degree,

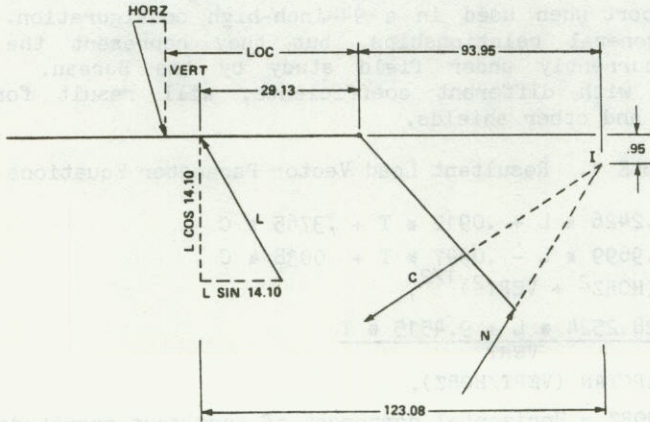
- L = Average leg force, ton,
 T = Net canopy capsule force, ton,
 C = Average compression lemniscate link axial force, ton.

The vector parameter equations are derived from three independent equations produced by summation of moments about the canopy hinge pin, the instantaneous center of the lemniscate links, and the tension link caving shield hinge pin as shown in the diagrams below.



SUMMATION OF MOMENTS ABOUT POINT A: $\Sigma + M(A) = 0$

$$\text{VERT}(\text{LOC}) + L \cos 14.10 (29.13) + T \sin 31.22 (4.92) + T \cos 31.22 (8.07) = 0 \quad (1)$$



SUMMATION OF MOMENTS ABOUT POINT I: $\Sigma + M(I) = 0$

$$\text{VERT} (\text{LOC} + 93.95) + \text{HORZ} (0.95) + L \cos 14.10 (123.08) - L \sin 14.10 (0.95) = 0 \quad (2)$$

vector parameters; but resultant location is strongly correlated to canopy capsule pressure, and resultant angle shows a strong dependence on link strain, as does horizontal force.

The assumptions made in these studies of the resultant load vector concept were primarily two-fold:

- ° A planar static model was utilized as opposed to a three-dimensional model.
- ° No gob (caving shield) loading was assumed.

Because the model was a planar static one, all parameters were measured with respect to a plane defined by the vertical (roof-to-floor) and horizontal (face-to-waste) axes. The lateral dimension (parallel to the face line) was not considered in this two-dimensional model. The lack of a three-dimensional model prevented determination of lateral position (across the canopy width) or lateral inclination (parallel to the face line) of the resultant load vector.

A preliminary theoretical analysis of the effect of gob loading was made for the specific shield geometry under study. Table 3 presents results for a gob load vector applied at the gob (caving) shield center with angles up to 45° from vertical and magnitudes up to 50 tons. The table shows relative deviations in the vector parameters, expressed both as measured differences and percent difference. The base case utilized for the gob study was 29.79 tons of horizontal force and 301.21 tons of vertical force, producing a resultant magnitude of 302.68 tons, an angle of 84.35° , and a location of 28.16 inches. Examination of the data reveals that gob load has the largest impact on the horizontal force prediction. The magnitude of error decreases as the gob load vector inclination moves away from vertical and increases with increasing gob load. In the worst case examined, the magnitude of the horizontal load error was -11.54 tons, which would not prevent the Bureau from achieving its goal of determining a first-order approximation of underground horizontal shield loading. The net result in the prediction of the resultant angle due to the presence of gob load is relatively insignificant since the vertical loading decreased proportionally to the decrease in horizontal load. The effect of gob load on the resultant location is to move the position of the resultant load vector forward and, in the case examined, cause a reduction in support resistance of up to 37 tons as shown by the deviation in the resultant magnitude.

TABLE 3. Impact of Gob Load on Resultant Load Vector Parameters

Load case; angle, deg	Δ horizontal force,		Δ vertical force,		Δ resultant magnitude,		Δ resultant angle,		Δ resultant location,	
	ton	pct	ton	pct	ton	pct	deg	pct	in	pct
GOB LOAD = 10 TONS										
15.....	2.31	7.75	7.11	2.36	7.30	2.41	0.31	0.37	0.68	2.42
30.....	1.52	5.12	6.88	2.29	7.00	2.31	.16	.19	.66	2.34
45.....	.64	2.14	6.19	2.06	6.22	2.06	.00	.01	.59	2.10
GOB LOAD = 30 TONS										
15.....	6.92	23.24	21.32	7.08	21.86	7.22	0.98	1.16	2.15	7.62
30.....	4.57	15.35	20.65	6.86	20.99	6.93	.51	0.61	2.07	7.36
45.....	1.91	6.42	18.57	6.17	18.67	6.17	.01	0.02	1.85	6.57
GOB LOAD = 50 TONS										
15.....	11.54	38.74	35.54	11.80	36.38	12.02	1.72	2.04	3.77	13.38
30.....	7.62	25.59	34.42	11.43	34.97	11.55	.90	1.07	3.63	12.90
45.....	3.19	10.69	30.95	10.28	31.11	10.28	.03	0.03	3.22	11.45

RESULTANT LOAD VECTOR INSTRUMENTATION

Input parameters for determination of resultant load vector parameters have been defined as leg, canopy capsule, and compression lemniscate link forces based upon a static analysis of the shield as presented in the previous section. To measure these variables, an eight-sensor instrumentation array was selected consisting of the following:

- Two pressure transducers (one in each leg cylinder) to measure leg pressure.
- Two pressure transducers, one each to measure canopy capsule extension and retraction pressure.
- Four strain gages (two on each compression link) to measure lemniscate link strain.

Two data acquisition systems were utilized for this study. The first system consists of a strain indicator that requires manual intervention to balance the sensor bridge network so individual sensors can be read. A switching unit is utilized to switch from channel to channel, and data are manually written down or voice-recorded on magnetic tape. The units are battery powered, intrinsically safe, and approved by the Mine Safety and Health Administration (MSHA) for use at the face.

The second system provides 14 channels of analog data with 8 hours of continuous recording onto a magnetic tape recorder. The primary components of the system are the FM recorder and signal conditioning unit, which provide 5 V dc gage excitation and fixed gains of 500 - 1,000 for individual sensors. The system was granted an experimental permit by MSHA for underground use.

The recorder, however, is not permissible and must be kept in fresh air, requiring coaxial data cables to be strung from each sensor to the recorder. All other components are either intrinsically safe or have been placed in explosion-proof boxes.

LABORATORY ASSESSMENT OF RESULTANT LOAD VECTOR STUDIES

Utilizing the Bureau's MRS, a thorough evaluation of the resultant load vector concept and capabilities of the data acquisition system was conducted to establish confidence intervals for prediction of vector parameters. Basically, two types of tests were conducted with the aid of the simulator:

- Load cell tests - A load cell was placed on the support canopy to act as a known load vector for prediction using the resultant load vector sensor array (BSI, 1983).
- Horizontal load tests - A known horizontal and vertical load was applied to the support specimen for prediction using the sensor array (BSI, 1983).

Load Cell Tests

These tests were limited to pure vertical loading, and the primary objective was to assess the ability to predict the location and magnitude of the resultant load vector as the angle was held constant at 90° . A total of 16 load cell positions were selected, representing 6 locations along the shield axis and 4 across its width, as shown in figure 2. Results of the tests are shown in table 4. Maximum vector parameter prediction errors were

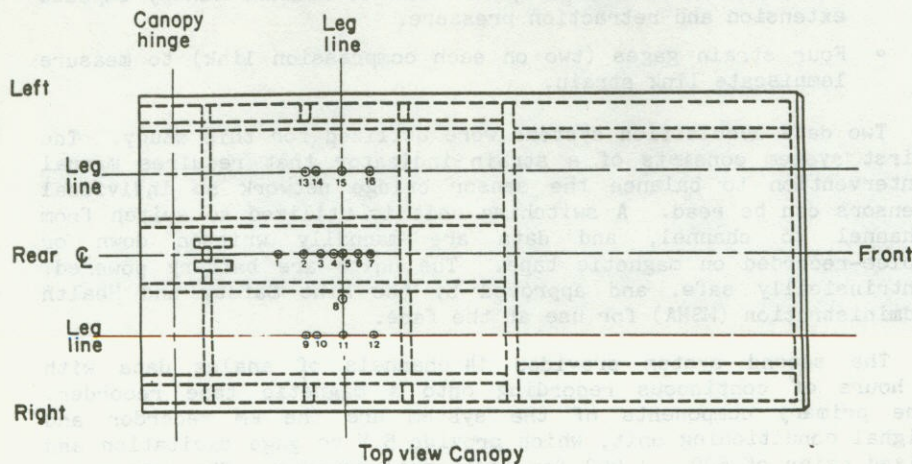


Figure 2. Laboratory test - load cell placement.

TABLE 4. Load Cell Test Results

Cell location	AVF, ¹ kip	F/AVF, ² pct	Δ Loc, ³ in	Δ Angle, ⁴ deg
1	196	-7.67	5.92	1.39
2	210	-5.72	1.69	-0.30
3	325	-2.98	0.91	-1.52
4	599	-3.25	0.75	-0.51
5	602	-3.51	-0.06	0.26
6	602	-2.16	-1.09	0.55
7	457	-2.83	-2.05	0.47
8	586	-3.39	-0.05	0.19
9	201	-5.80	1.95	-2.03
10	236	-6.66	1.69	-1.85
11	498	-2.85	0.05	-0.36
12	481	-2.31	-2.08	0.77
13	197	-6.74	1.82	-2.75
14	266	-5.63	1.93	-2.46
15	512	-3.68	0.09	-0.58
16	462	-2.85	-2.06	0.63
Max		-7.67	5.92	-2.75

¹AVF - MRS-measured average vertical force

²F - Array-predicted vector force magnitude

³ Δ Loc - Difference between measured and predicted resultant locations

⁴ Δ Angle - Difference between measured and predicted resultant angles

7.67 pct for magnitude, 5.92 inches for location, and 2.75° for resultant angle. It should be noted that these maximum errors occurred at a relatively low shield load, only 196 kip, with the vector located in a fairly unstable region of the support several inches forward of the leg center line. If the supports are set properly against the roof, loads of 400 kip or greater can be expected. Deviations more representative of anticipated mine loads are as follows: magnitude - 2-3 pct, location - 2 in; angle-1°.

To put these figures into perspective, preliminary field data indicate leg pressure variations of 5,000 psi during the mining cycle, which alone would result in variations in the resultant magnitude of about 300 tons. Laboratory tests indicate a stability range in terms of resultant location of 10 inches on either side of the leg center line. A vector outside this range would not pass through the support base and would cause instability. As for the resultant angle, a friction coefficient of 0.3 would produce an angle change of 16.7°.

Conclusions drawn from these tests were:

- The larger the vertical force applied, the smaller the error in vertical force magnitude.

- The level of vector magnitude error expected at mine load levels is 2-3 pct (excluding the gob load disturbance).
- The magnitude and location prediction deviations are functions of location.
- All three parameters appear to be independent of location across the width of the canopy, justifying the use of a planar static model for the load conditions analyzed.

Horizontal Load Tests

The objective of these tests was to evaluate the ability of the eight-sensor planar static model and data acquisition system to measure horizontal shield loading. The test consisted of preloading the support with a 600-kip vertical force followed by application of a horizontal load at a rate of 40 kip/min, to a maximum of 120 kip; then using the MRS data as the known, the horizontal and vertical forces were predicted from sensors on the support. The test results are shown in table 5. Horizontal load prediction deviations were systematic, with the maximum error of 22.6 kip (11.8 tons) occurring at no horizontal load. The error became smaller, reaching zero at a horizontal load of about 70 kip, and then it changed polarity as the load increased to the maximum of 120 kip. Average error in horizontal load prediction was 6.8 kip (3.4 tons).

TABLE 5. - Horizontal load prediction test results

Horizontal load, kip		Difference		
Applied	Predicted	pct	kip	ton
0	22.6	NA	22.6	11.8
16	28.8	77.2	12.8	6.4
32	37.0	15.7	5.0	2.5
48	48.4	.9	.4	.2
64	66.0	-1.7	-2.0	-1.0
80	77.8	-2.9	-2.2	-1.1
96	92.0	-4.4	-4.0	-2.0
112	103.6	-7.8	-8.4	-4.2
120	111.8	-6.8	-8.2	-4.1

NA - Not available.

The test results are summarized as follows:

- As a worst case, horizontal load can be measured to within 12 tons. A more optimistic error magnitude would be 5 tons.

- The lemniscate link is subjected to bending from pin eccentricity and friction and therefore requires a minimum of two gages per link to assess axial link loading.
- The dominant variable in determining horizontal load is lemniscate link strain.
- As horizontal load increases, link strain diminishes, making strain measurements more difficult. Strain values ranged from 78 microstrain at zero horizontal load to 27 micro-strain at a 120-kip horizontal load for the rear face gages, and 325 to 99 microstrain for the forward link gages.

FIELD STUDIES

Initial efforts to collect resultant load vector field data were largely unsuccessful because of instrumentation problems. Pressure data of sufficient quality for vector determinations were collected, but problems were encountered with strain data because of data acquisition instrumentation stability problems, low signal levels, and undesired signal attenuation during recording. The absence of lemniscate link strain data prohibited determination of resultant load vector parameters; however, some observations can be made from the leg and canopy capsule pressure data.

A typical leg pressure plot from the analog recorder is shown in figure 3. The shapes of the curves were consistent and matched those provided by mechanical chart recorders. There was some imbalance in leg pressure (right versus left leg), but as a whole, the shield was loaded fairly symmetrically. Leg pressure plots are characterized by an initial contact spike, followed by a valley, with pressure then rising at a declining rate. The passage of the shearer and advancement of the adjacent support are clearly indicated by a sharp increase in the slope of the pressure plot (points A and B, respectively, on figure 3). Setting pressures at this particular mine site were found to be low, ranging from 1,800 to 3,400 psi, with typical values being around 3,200 psi (BSI, 1983).

Another interesting observation can be made from the canopy capsule pressure plots. The large majority of capsule pressure plots were essentially flat line, as shown in figure 4. A flat line indicates very little convergence and that the canopy is not rotating about the leg pivot point. The earlier discussion on shield mechanics showed that the resultant load vector location is strongly correlated to capsule pressure. The flat line capsule response indicates that the resultant location is not significantly changing and is probably located at the ideal location near the leg center line.

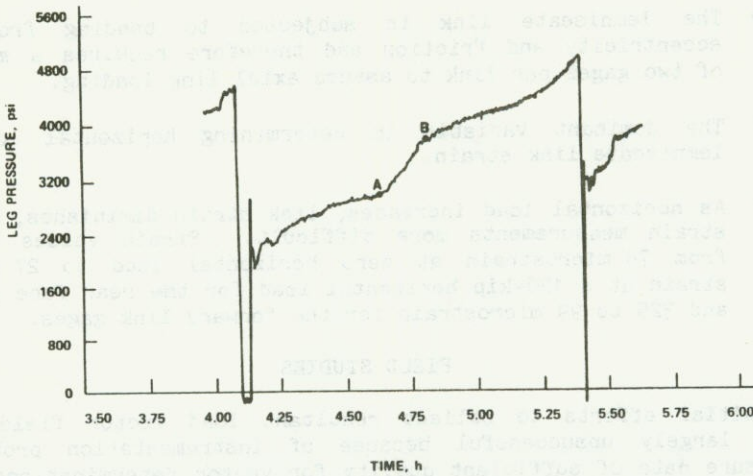


Figure 3. Typical leg pressure plot.

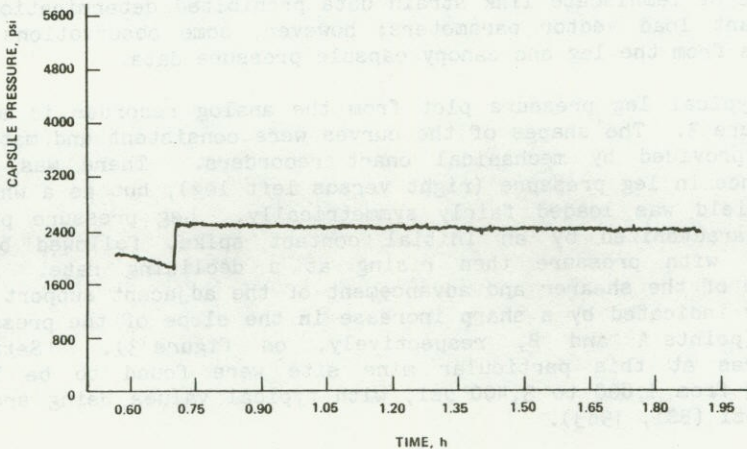


Figure 4. Flat-line behavior of canopy capsule.

The data acquisition problems, which were encountered during initial field efforts, were largely environmental in nature; the mechanical equipment began to behave erratically because of dust contamination. These problems have been corrected, and field efforts are about to resume. The scope of the field effort will involve monitoring five longwall shields for resultant load vector determinations and leg pressure studies of several neighboring

shields. Resultant load vector parameters will be plotted as a function of time (shield cycle) and an assessment of shield behavior will be made.

CONCLUSIONS AND DISCUSSION

Proper ground control is critical to the success of longwall mining. Improvement in support design or utilization would provide benefits of increased production and reduced capital risk for the longwall operator. The resultant load vector concept presented in this paper is a first step in providing a more in-depth understanding of support behavior and requirements for more effective ground control. The technique has been successfully laboratory tested and is currently undergoing field evaluation. The major drawbacks have been (1) the lack of suitable data acquisition equipment, which is a solvable problem, but one that will require considerable resources, and (2) time, because of the permissibility constraints and the need for MSHA approval. It is thought that resultant load vector studies will provide an advancement in state-of-the-art longwall roof support utilization and a foundation for additional studies to improve support design.

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Chapter 69

AN IMPROVED TECHNIQUE OF IN-SITU MEASUREMENT OF FRACTURE ZONES

SURROUNDING COALMINE GATEROADS

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ABSTRACT

An understanding of the nature of yield zones surrounding coal mine excavations is fundamental to the design of mine support systems. Various methods have been designed for investigation of underground rock masses and an extensometric technique, the Magnetic Rod Extensometer, has been developed to overcome inherent design limitations in existing equipment of this type. This paper refers to the prototype form of the extensometer and details changes designed to overcome a variety of drawbacks indicated by routine use of the equipment. The principal change has been one of manual to computerised data acquisition and presentation in the laboratory phase of the system. This has dramatically reduced the time taken to process underground measurements and produce numerical and graphical results. The revised Magnetic Rod Extensometer represents an integrated instrumentation system that combines speed and accuracy in all areas of data handling.

INTRODUCTION

The closure of coal mine excavations results from the development of a failed or yield zone in the surrounding rocks due to redistribution of the primitive stress field. This failed or yield zone presents problems to the mining engineer who is required to design a support system which balances economic and operational factors and allows mining to continue in the presence of this yield zone. An understanding of the nature of the yield zone is therefore a fundamental prerequisite for the design of mine support systems.

Various methods have been developed for investigation of underground rock masses using stress measuring instruments, modulus of deformation instruments, and displacement measuring instruments. A careful appraisal of available in-situ instrumentation indicated that displacement or strain measuring instruments provided the most realistic approach to in-situ measurement in the 'soft rock' coal measures environment (Aziz, 1978). Such instrumentation allows identification of the origin of the pressure zone created by mining activity, and the duration and frequency of strata movements that induce excavation deformation. Hence characteristic deformation processes can be established for support system design in representative geological environments.

Due to inherent design limitations in extensometric techniques, indicated by a market survey in 1976, a new system termed the Magnetic Rod Extensometer was developed for in-situ measurement of fracture zones surrounding coalmine gateroads (Smart et al, 1977).

Principle of Magnetic Rod Extensometer System

The technique consisted of monitoring the positions of magnetic reference points, i.e. small rectangular magnets installed at pre-determined intervals along the axis of a borehole, typically 55mm diameter. The relative positions of the magnets were periodically recorded on a sensing medium which was temporarily inserted into the borehole. The sensing medium was then removed to the laboratory where a scanning arrangement measured the distances between the magnetic signals, these distances corresponding to the distances between reference points in the borehole. The reference points moved with the strata, and thus changing distances between magnetic signals recorded on the sensing medium measured differential movements of the rock adjacent to the borehole.

Since 1977, the Strata Mechanics Research Group, Cardiff, U.K., has used the technique on a routine basis in a study of yield zones around arch and rectangular profile gateroads, and characteristic deformation processes have been identified (Isaac and Smart, 1983). Although the new system overcame design limitations in existing in-situ measuring techniques when used in coal mines, it too suffered from shortcomings that affected its use as a research tool. Because of the shortcomings of the prototype instrumentation, continued development was necessary to remove them from the system, thus improving its general efficiency and accuracy.

The principal demerits of the prototype extensometer were its speed of use both underground and on surface, and to some extent its accuracy. The problems of speed and accuracy are interrelated and the logic behind design modifications is better appreciated by consideration of both with regard to the three elements of the system, viz.,

- the extensometer rods containing the sensing medium;
- the laboratory scanning arrangement;
- data processing.

System Design Modifications

Extensometer Rods The prototype system which has been fully reported, employed 1.2m length, 25mm diameter wooden rods. A groove 8.0mm by 7.5mm machined along each rod length was fitted with a PVC flexible tube, 8.0mm external diameter, filled with iron filings of -75 μ m size, and sealed. The tube of iron filings comprised the sensing medium, and the individual rods could be connected together by means of a specially designed universal coupling to create a string of rods of the required length when inserted into the borehole.

The original rods were found to suffer the following deficiencies:

- (i) because of their rigid nature they were unable to negotiate minor bends in deforming boreholes;
- (ii) the rod connectors were difficult to use by one person and tended to slacken off in use;
- (iii) the rods had no distinct numbering code and required care in uncoupling and subsequent scanning to ensure the correct monitoring sequence was maintained.

To overcome these drawbacks, the wooden rods were replaced by shorter, flexible polypenco nylon rods (1.0m long, 20mm diameter) fitted with improved snap-lock connectors, each having a distinct numbering code.

Laboratory Scanning Arrangement The original scanning arrangement comprised a Hall Effect Transducer (Indium Arsenide) mounted on a moving carriage, which scanned the lengths of rod detecting the presence of magnetic signals which represent the reference points in the borehole. The rods were supported on wooden blocks positioned parallel to the direction of carriage movement, the carriage being driven at constant speed along a 2m long girder by virtue of an endless chain connected to a motor.

The output from the transducer was passed into a signal conditioning unit, an amplifier, and then into a 2 channel chart recorder. One output channel from the recorder produced a graphical representation of each magnetic peak, and the other produced a simultaneous Moire fringe graphical output which allowed the manual calculation of the distances between the reference points detected on the first channel.

This arrangement suffered the following drawbacks:

- (i) a low scanning speed, which prevented the immediate re-use of the rods as demanded by some investigations;
- (ii) processing of chart recorder output was tedious and time-consuming and a possible cause of human error through fatigue;
- (iii) the endless chain used to drive the scanning carriage was subject to occasional 'whiplash' or jerking;
- (iv) the rod supporting block design allowed occasional accidental displacement.

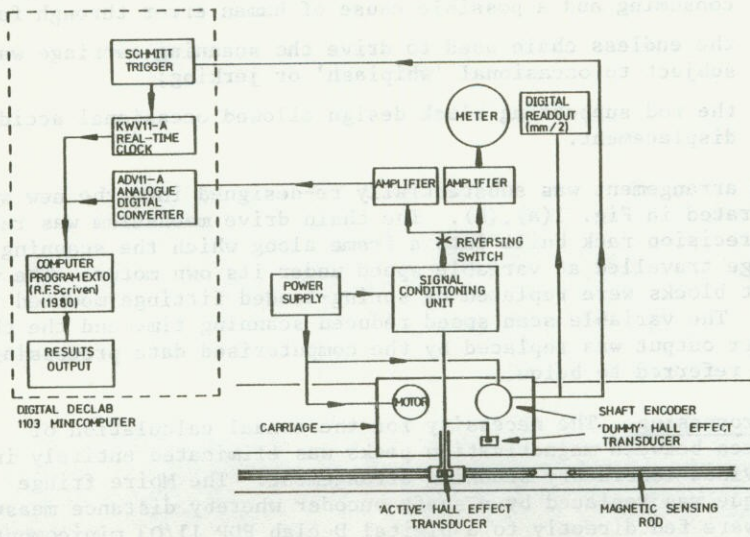
The arrangement was substantially re-designed into the new system illustrated in Fig. 1(a),(b). The chain drive mechanism was replaced by a precision rack built into a frame along which the scanning carriage travelled at variable speed under its own motor. The wooden support blocks were replaced by spring-loaded fittings mounted in the frame. The variable scan speed reduced scanning time and the chart recorder output was replaced by the computerised data processing system referred to below.

Data Processing The necessity for the manual calculation of distances between magnetisation peaks was eliminated entirely in the revised laboratory scanning arrangement. The Moire fringe technique was replaced by a shaft encoder whereby distance measurements were fed directly to a Digital Declab PDP 11/03 minicomputer.

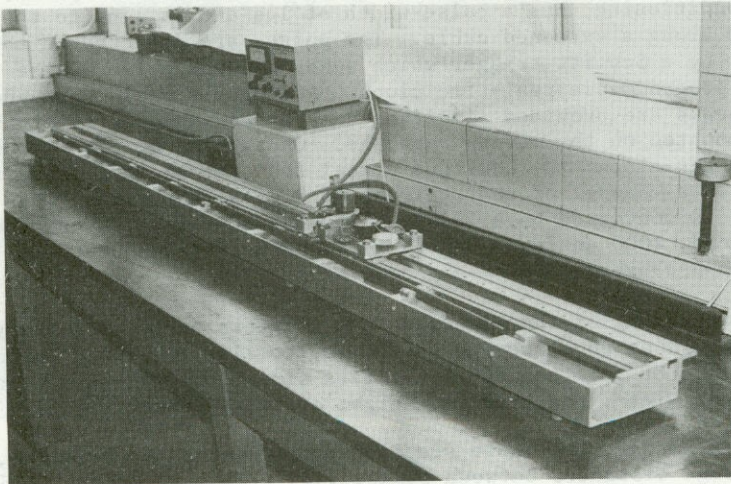
The encoder is mounted on the moving carriage and produces 1000 electronic pulses for each 50mm of travel. Each encoder pulse, after conditioning, has a pulse width of 1µs and is sent to the computer along a screened cable. The pulses are detected in the computer by a Schmitt trigger which sends a 500ns pulse to the KW11-A programmable real-time clock. Analogue signals of magnetisation peaks are provided simultaneously by the Hall Effect transducers mounted on the moving carriage. On receipt of each encoder pulse by the clock, the analogue signal from the Hall transducer present on a channel of the analogue to digital converter, the ADV11-A, is converted to a 12 bit binary value. Thus as the rods are scanned, 1000 analogue values from the Hall transducer are sampled and converted for every 50mm of travel.

The processing of the binary values consists of passing every value through an envelope-processing sub-routine, which determines the presence of peak values in the data stream and provides information on the width, position and height of these peaks.

In this manner, the whole scanning procedure is computerised, with average distances between peaks, i.e. reference points, being calculated automatically by program EXT0. The values obtained from this program provide the input for other programs which calculate rock movement and strain, and produce a graphical representation of results using a standard graph-plotting routine on a GEC 4070 computer.



(a) Schematic Diagram



(b) Laboratory Arrangement

Fig. 1 Computer-Aided Scanning of Magnetic Rod Extensometer

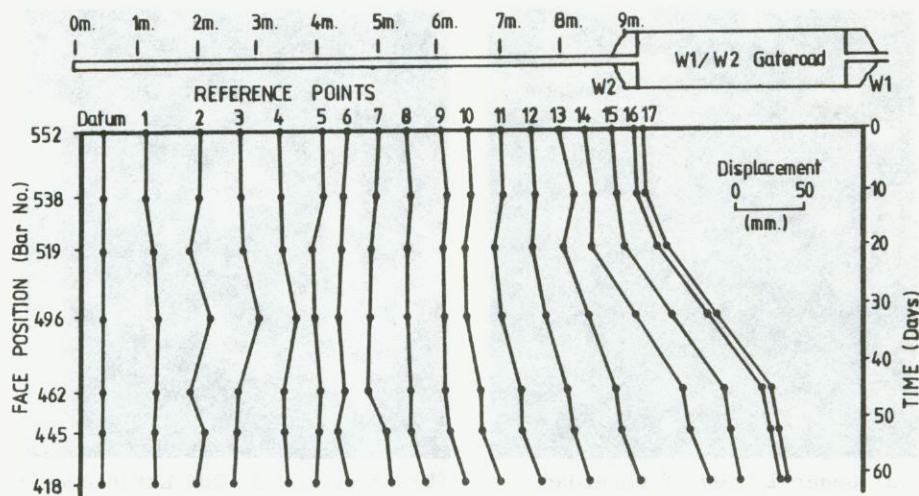


Fig. 2 Computer Plot of Extensometer Results

Trials and Performance The test location for the improved system was the new Betws anthracite mine in West Wales, U.K. A series of rectangular profile dual life gateroads in the Red Vein seam provided extensive results and confirmed the greater ease of use of the system (Fig. 3) (Nicholas, 1982).

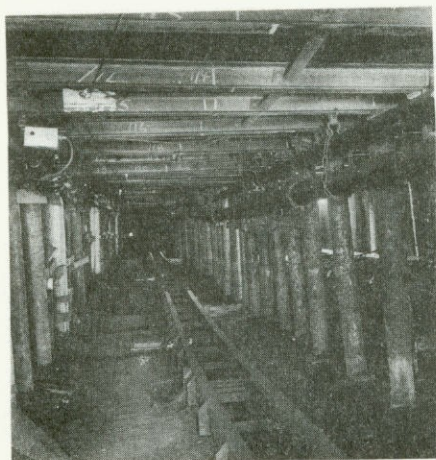
The revised laboratory scanning arrangement was found to be of particular benefit, with a reduction in scanning time of 75%. One 'run' of a 10m long set of rods was reduced from 2 hours to 0.5 hour, while the computerised data processing provided almost instant results for a task that formerly took 1 day. Data reduction for the life of a borehole could also be achieved in less than 1 day - a significant improvement.

Results provided by the new system formed part of a comprehensive gateroad study programme. Speed of data acquisition undoubtedly assisted general interpretation of all results with regard to identification of the general strata deformation process.

A comparative study of the accuracy of the old and new systems revealed that for limits containing 95% of the data points, manual system accuracy was $\pm 2.41\text{mm}$, whereas the computerised technique was assessed as $\pm 1.05\text{mm}$.

CONCLUSIONS

The system has fulfilled its design requirements at a series of investigations in British coal mines, viz.,



(a) General View of Gateroad before 1st face extraction



(b) Insertion of Rod Extensometer and recording of magnets



(c) 1st Stage Deformation of Ribside/Cog - 2nd Face Extraction



(d) 2nd Stage Deformation of Ribside/Cog - 2nd Face Extraction

Fig. 3 W1/W2 Gateroad, Betws Colliery
South Wales Coalfield, U.K.

- (i) it is robust, accurate, low-cost, easy to use, and Intrinsically Safe;
- (ii) numerous compact reference points may be installed in a single borehole giving maximum detail of rock movement;
- (iii) in its new form, the Magnetic Rod Extensometer represents an integrated instrumentation system that combines speed and accuracy of data acquisition, processing, reduction and presentation.

The simplicity of the system and results generated by its application initially at gate roadways and subsequently at coalfaces, is reflected in interest currently expressed in other areas of the mining world in its commercially available form (Kenure Developments, 1984).

In its present form, the system is capable of further improvement and current research is directed towards its incorporation into a data logging system that will eliminate the need for extensometer rods and make possible high frequency monitoring.

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Chapter 70

COMPREHENSIVE OBSERVATION AND RESEARCH ON THE MECHANISM OF WATER-IRRUPTIONS FROM THE FLOOR

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ABSTRACT

In China there are more than 30 coal fields menaced by karst-confined water. In the north and east of China there are many collieries which mine coal seams of the Carboniferous and Permian Systems, which are above the Ordovician limestone (general thickness is 500-600 meters). Ordovician limestone is full of karst fissures, contains a lot of water under high pressure ($15-65 \text{ kg/cm}^2$) near the commercial coal seams (20-80 meters), the water easily infiltrates the thin limestone at the point of the structure crevice, so water-irruptions from the floor are the main source of water-related disasters in these coal fields. According to the statistics there were more than 300 accidents of all kinds of water-irruptions from the floor in these fields since the beginning of their development. It is of great significance in developing coal production in the eastern part of China to solve the problem of safe mining of the seams menaced by the karst-confined water of the Ordovician limestone. The research on this subject consists of three parts: (1) investigating the geological structure and hydrogeological conditions of each mine of the mining area; (2) researching the mechanism of water-irruptions from the floor and proposing reliable methods to predict and monitor; (3) improving the mining system and proposing a mining plan with safety precautions against water-caused accidents. The chief task is to research the mechanisms involved and to discover the causes and conditions of the phenomenon.

The authors have obtained some findings on this subject in the last five years, making use of the comprehensive observation and research

system, which will be discussed under three headings.

CLASSIFICATIONS OF WATER-IRRUPTIONS FROM THE FLOOR

Each water-irruption from the floor has a different cause. In order to determine the causes and conditions of floor water-irruptions we must first classify them as follows.

First, according to the position of the irruption, it can be classified into two types: (1) at the excavation opening; (2) at the working face. According to field statistics, these two kinds of water-irruptions account for 50% each of the total. Most of the irruptions at the excavation opening are under conditions of meeting with permeable faults or the structure crevice zone connecting with the karst-confined water, and the probability of an irruption is greater when the opening is near the fault, or while excavating at the fault. To prevent water-irruptions at the point of excavation, the water-discovery excavating method must be used with caution. The mechanism of water-irruptions at the mining face is very complex and will be discussed separately.

Second, according to the manner of the floor water-irruption, it can be classified into three types: (1) sudden bursting; (2) slow bursting; and (3) delay bursting.

Sudden bursting usually occurs near the excavating and mining faces. As soon as the water-irruption occurs, the water inflow reaches the maximum value (peak) in a short time, the irruption is very strong at high speed, the immediate floor of the seam will lift, and the water will carry rock fragments.

Slow bursting also occurs near the excavating and mining faces, but water inflow increases gradually and reaches the maximum value in several hours or days or even several months.

Delay bursting occurs at a used opening or a mined section; the water inflow may be large or small and lasts for varying lengths of time.

These three kinds of floor water-irruptions indicate that all factors causing them have a continuously acting process.

ANALYSIS OF THE MECHANISMS OF WATER-IRRUPTIONS FROM THE FLOOR

Water-irruption from the floor is a complicated geological and mining phenomenon which occurs tens of meters below the coal seam. So far we cannot effectively monitor the process, which causes some difficulties in research work.

Based on analysis of a great deal of information and on field observations, water-irruptions from the floor at the mining face are

associated with the following factors:

The Geological Structure is the Controlling Factor in Water-Irruptions from the Floor

Field statistical information indicates that more than 80% of the water-irruptions take place near the fault structure, and almost all water channels are formed by the original fissure in the floor aquiclude. The height that the karst-confined water rises along the fissure over the limestone before mining is called the original conducting height. The rising of the karst-confined water near the coal seam increases the probability of water irruptions.

In the eastern part of China, when studying fault structures, the active structures related to the New Structure Movement are more closely observed. According to regional geological information, fault crack structures along the direction of NNE (N 20° E) and NNW are highly developed in the two groups of faults of the New Structure Movement, spread for a long distance along the strike or dip and may conduct a distant water body into the underground mine. There are many serious accidents related to these two groups of the fault crack structure. Field information indicates that when a serious accident occurs, the Ordovician springs more than ten kilometers away may dry up, showing that the area of disturbance is very large.

Underground Pressure is the Initiating Factor in Water-Irruptions from the Floor

Most of the water-irruptions from the floor at the mining face are relative to the underground pressure, except those at the excavating face for exposing the water-bearing faults or if the water-pillar is less strong.

Observations have proved that with the advance of the mining face, the floor in front of the coal-wall is compressed by the action of the abutment pressure; with the further advance of the mining face, the floor into the goaf expands at once and the immediate floor shows lift; and with the continuous advance of the mining face, the caved rock mass is compressed and the floor changes from an expanding state into a compressing state again, while it is acted upon once more by the back abutment pressure. The region affected along the strike of the mining face is the region of one periodic pressure. Along the dip of the mining face, the floor in the inclined length of the face is in the state of expansion and the floor below the coal-pillar at two sides of the face is in the state of compression. Hence, during one step of periodic pressure, the floor below the boundary pillars of goaf and compressed rock mass is a compression zone, the middle region of which is an expansion zone. There is a shearing zone at the meeting points of these two zones; in the sections along the strike and dip of the seam it is a shearing face inclined to the middle part of the goaf, shown in Figure 1; it will be acted upon by each shearing of the underground and hydraulic pressure, especially during periodic pressure

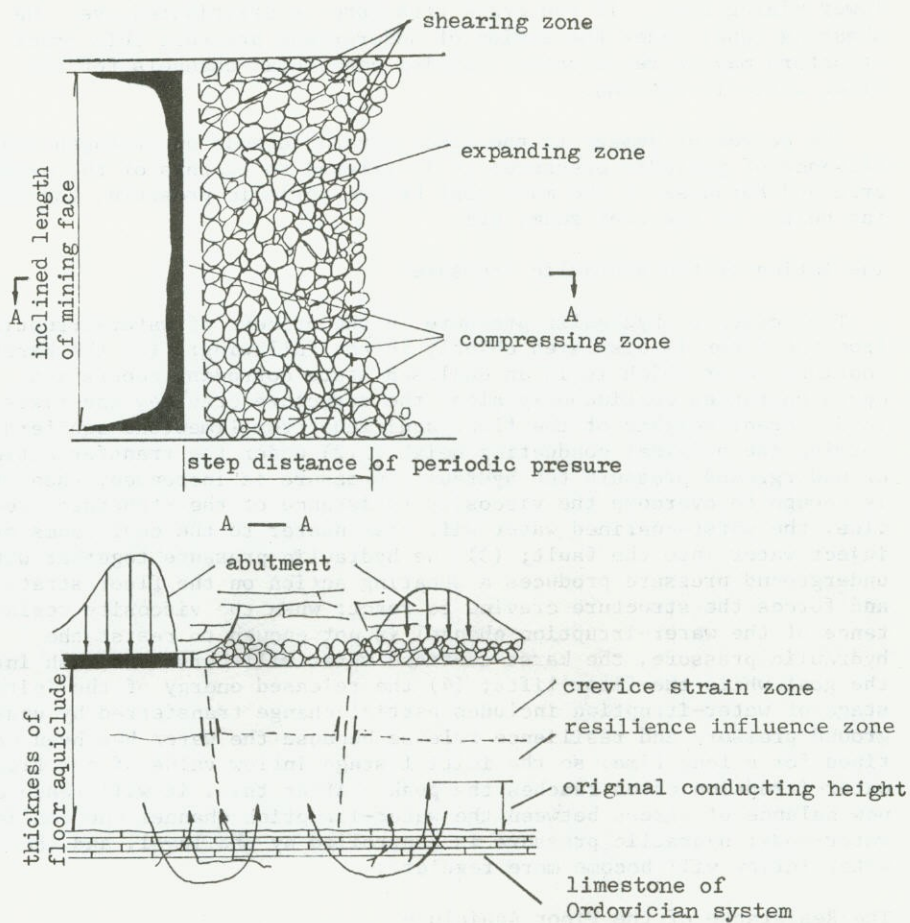


Fig. 1 Sketch map of the mining stress distribution of the floor aquiclude

of the main roof. The strain and damage of the floor strata in the shearing zone is the greatest.

Most field information indicates that the water-irruptions from the floor at the mining face usually happen at the point which is on the shearing zone of the boundary inside of the goaf, especially at the lower mining face. If the crack structure is distributed over the shearing zone, under the action of underground pressure this crack structure may be reactivated, forming beneficial channels for the floor water-irruptions.

The degree of damage to the floor strata depends on strength and brisance of periodic pressure; it is related to factors of the exposed area and hardness of the main roof before periodic pressure, the mining height of the coal seam, etc.

The Action of the Hydraulic Pressure

The action of hydraulic pressure in the process of water-irruptions from the floor is displayed chiefly in the following: (1) the karst-confined water which is in an enclosed state corrodes, scours and opens entrances continuously along the structure crevice, and rises in to different heights of the floor aquiclude from limestone aquifers, forming the original conducting height; (2) under the transfer action of underground pressure the hydraulic pressure is increased; when it is enough to overcome the viscosity resistance of the structure crevice, the karst-confined water will rise nearer to the coal seams or inject water into the fault; (3) the hydraulic pressure together with underground pressure produces a shearing action on the floor strata and forces the structure crevice to react; when the viscosity resistance of the water-irruption channel is not enough to resist the hydraulic pressure, the karst-confined water will burst and rush into the goaf while the floor lifts; (4) the released energy of the initial stage of water-irruption includes partial change transferred by underground pressure and resilience release because the water has been confined for a long time, so the initial stage inflow value of the floor water-irruption often reaches the peak. After this, it will reach a new balance of stress between the water-irruption channel and confined water-body; hydraulic pressure is controlled by the level, and the water inflow will become more regular.

The Resistance of the Floor Aquiclude

The floor aquiclude with fixed thickness and composition, including its original faulted crack zone, has a fixed resistance to the underground and hydraulic pressures. Practical comprehensive observation confirms that under the action of underground pressure the floor strata could clearly divide into two zones: (1) the Crevice Strain Zone, which is the upper floor strata in which the crevice parallel with and crossing the strata appears; it produces the floor-lift in the immediate floor, and it is the place of the strain and damage of floor strata. The depth of the crevice strain zone is also called

damage depth. Observation in some examples indicates that the damage depth reached 14-17 meters when the inclined length of the mining face was 120-130 meters, and only 3-4 meters when 30 meters. Although the resistance of the floor strata in the crevice strain zone is lower, its ability to resist water is not completely lost. (2) The Resilience Influence Zone, which is the other aquiclude under the crevice strain zone and its function is to transfer the underground pressure, in which the strain produced could be renewed and the remaining strain is very small.

When the floor strata has some structure fissures, especially in the shearing zone, its strain and damage depth are generally much more than a normal floor strata. The experts and scholars of China are attaching more and more importance to the research of this field.

The various types of water-irruptions from the floor at the mining face are the results of combined action by these four factors. Research on water-irruptions from the floor must take these four factors into consideration and include field observations at the site. This observation and research on damage status under stress of the floor strata is also a new subject for large scale rock massif mechanics.

THE SYSTEM OF COMPREHENSIVE OBSERVATION AND RESEARCH ON WATER-IRRUPTIONS FROM THE FLOOR

Figures 2 and 3 indicate the engineering distribution of the comprehensive observation and research at the mining face No. 2701 of Fengfeng Mine No. 2.

The experimental mining face was selected at the mining face No. 2701 of the Fengfeng Mine No. 2. The mined seam is the Xiaoqing seam with 1.4 meters of thickness and 10° - 21° of dip; the immediate roof of this seam is limestone and the immediate floor is sandstone; the distance from the seam down to the top of the Ordovician limestone is 48-55 meters and the hydraulic pressure is 12.5 - 13.5 kg/cm^2 . The mining face has some small faults along the strike direction, and its length is 70-120 meters; the mining system used the strike longwall with full caving roof control.

The special observation road includes the lateral opening with 80 meters of length and the uphill opening with 70 meters of length, which was excavated at the Daqing seam about 24 meters under the Xiaoqing seam; its immediate roof is an aquiferous limestone 6 meters thick. During mining it has been dried. For comprehensive observation, each hole room was excavated at the upper, middle and lower parts of the observation-uphill and each group of holes was constructed so that every group had five holes, including one hole for observing the ground movement, one hole for observing the physical exploration and three holes for experiments on controlled water injection. The comprehensive observation used eight methods in all:

1. Observing the stress state of supports using the support

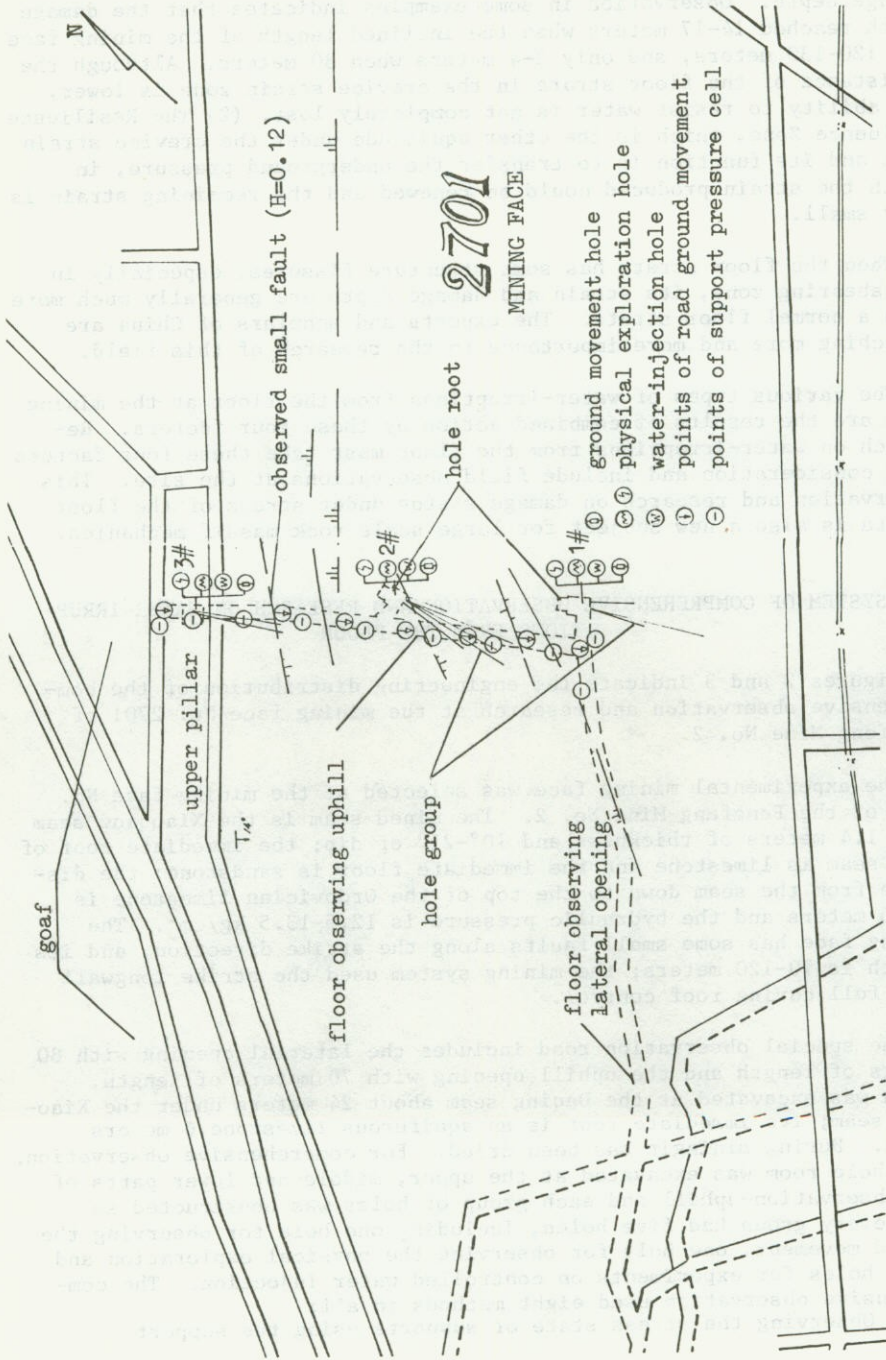


Fig. 2 Plan of engineering distribution of the comprehensive observation and research at the mining face No.2701 of Fengfeng Mine No.2

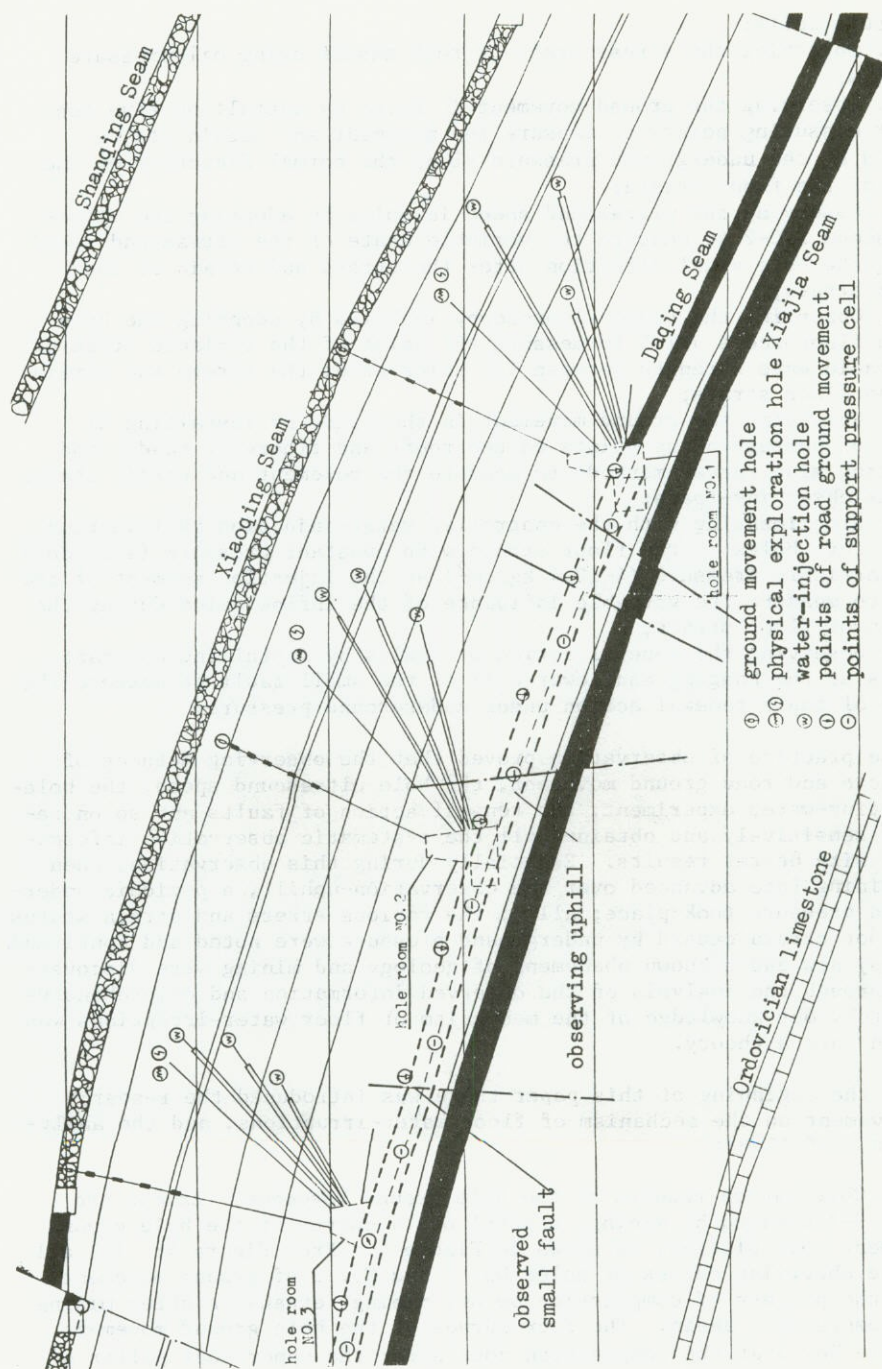


Fig. 3 Section of engineering distribution of the comprehensive observation and research at the mining face No.2701 of Fengfeng Mine No.2

pressure cells;

2. Observing the stress state of rock massif using oil-pressure vessels;

3. Observing the ground movement in holes by installing five constant measuring points to measure the movement and strain status caused by the underground pressure along the normal direction of the seam of the floor strata;

4. Observing the ultrasound speed in holes by adopting the ultrasonoscope SYC-2 to measure the variable state of the ultrasound speed along the hole axial direction after the stress and strain of the floor strata;

5. Observing the radio fluoroscopy in holes by adopting the hole radio fluoroscope JWT-1 to measure the value of the variable state of the radio wave strength between two holes after the stress and strain of the floor strata;

6. Observing the ground movement in the roads by installing 21 pairs of corresponding points on the roofs and floors of roads, and adopting mine survey methods to measure the movement and strain status of the observing-roads;

7. Experimenting with the controlled water-injection by injecting water into holes to the floor strata with constant pressure (4 kg/cm^2) and variable pressure ($4\text{--}12.4 \text{ kg/cm}^2$) at the injection segment of the hole to measure the variable influence of the inflow value during the strain of floor strata;

8. Observing the renewal action of faults by installing constant points at the hanging and lower wall of the small fault to measure the state of fault renewal action under underground pressure.

The practice of observation proved that the observing methods of the hole and road ground movement, the hole ultrasound speed, the hole-injecting-water experiment, the renewal action of faults and so on reacted sensitively and obtained all the systematic observation information, with better results. Especially during this observation, when the mining face advanced over the observation-uphill, a periodic underground pressure took place; all of the various stress and strain status of floor strata caused by underground pressure were noted and confirmed, so many new and unknown phenomena of geology and mining were discovered; through the analysis of the observed information and comprehensive research, our knowledge of the mechanism of floor water-irruptions was formed into a theory.

At the beginning of this paper there was introduced the research achievement on the mechanism of floor water-irruptions, and the analysis is as follows:

1. Observation results of the hole ground movement. Except the point 3-1 damaged by mining, a total of 14 curves of the hole ground movement were plotted, as shown in Figure 4. From Figure 4: (1) all of the observing curves of holes No. 1 and No. 2 of ground movement show the process of compression before mining, expansion after mining and compression again. The four curves of the hole ground movement of hole No. 3 at the compression zone under the upper coal pillar of

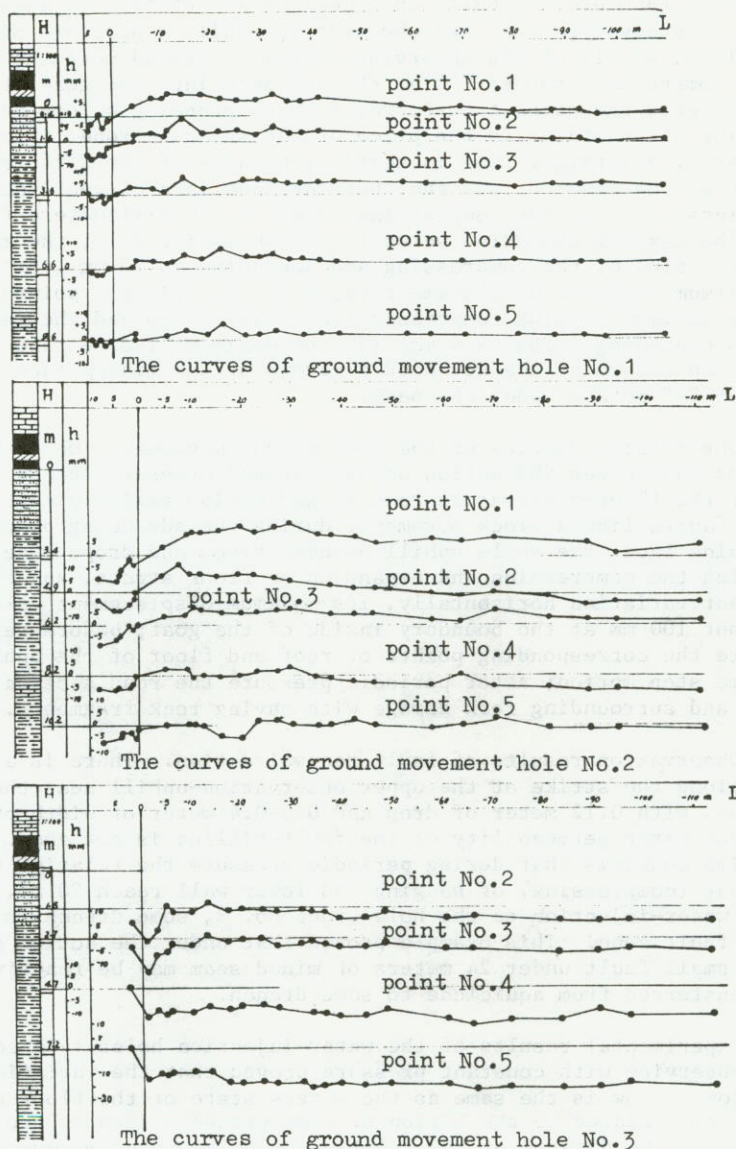


Fig. 4 Diagram of observing curves of the hole ground movement

the mining face are basically in a compression state; (2) when the mining face advanced over the observation-uphill a periodic pressure took place, so all of the observing curves of ground movement showed a sudden compressed status; (3) at the entrance into the goaf of the floor strata, the place transferred from compressing before mining to expanding after mining is the place of the shearing zone, which did not display a straight line in section; because of the different properties of the floor strata the shearing zone is a curved line; (4) at all points, the maximum compressing value is -18 millimeters (point 3-5), the maximum expanding value is +14 mm (point 2-1), the maximum variable value of the compressing and expanding is 25 mm (point 2-1), the maximum relative displacement value of two adjacent points is 9 mm, the maximum surplus compressing value is -16 mm and the maximum surplus expanding value is 4 mm; (5) the maximum fissure rate is 4-5.8% which occurs at the advanced 5 meters of the mining face and a depth of 2-5 meters under the seam.

2. Observation results of the road ground movement. Observation confirms that under the action of underground pressure the surrounding rock of the floor-observation road showed motion entirely controlled by the faults like a block movement; during the advancing process of the mining face, the whole uphill opening rises and drops generally following the compression and expansion of floor strata, and his displacement variation horizontally, its maximum displacement value reaching about 100 mm at the boundary inside of the goaf; before periodic pressure the corresponding points of roof and floor of the uphill show the same step motion; after periodic pressure the road section produces strain and surrounding rock damage with caving rock fragments.

3. Observation results of fault renewal action. There is a small fault along the strike at the upper observation-uphill near the shearing zone, with 0.12 meter of drop and 0.3-0.4 meter of width of fault zone; the water permeability of the fault-filling is not good. Observation confirms that during periodic pressure the relative displacement (compressing) of hanging and lower wall reach 20 mm, and during water-injection to the hole group No. 3, some drench is evident in the fault zone. This example proved that under the action of mining, a small fault under 24 meters of mined seam may be reactivated and transferred from aquiclude to some drench.

4. Experimental results of the water-injection holes. Injecting water observing with constant pressure proved that the variation of injection-inflow is the same as the stress state of the floor strata and closely related to the action of underground pressure, see Figure 5, which is the diagram of the relation of hole ground movement to the variation of hole-injection inflow. Injecting water with variable pressure is a model for the action of hydraulic pressure. The experiments confirm that for the floor strata in the compressing state, the injection inflow does not increase while the hydraulic pressure increases; for the floor strata in the strained and damaged state, the injection inflow only increases 1 L/min while the hydraulic pressure increases from 4 kg/cm² to 12.4 kg/cm². This indicates that for the

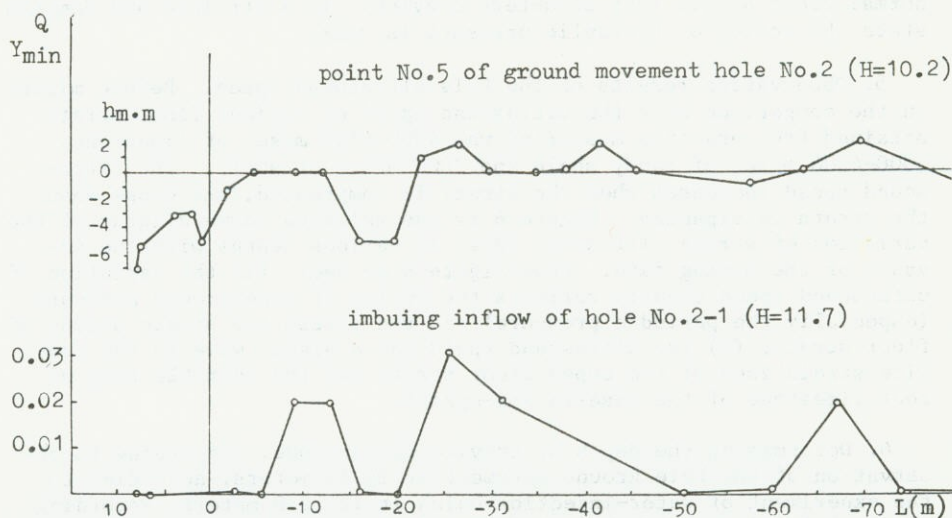


Fig. 5 The diagram of relative compare the hole ground movement with the variety of hole imbuement inflow

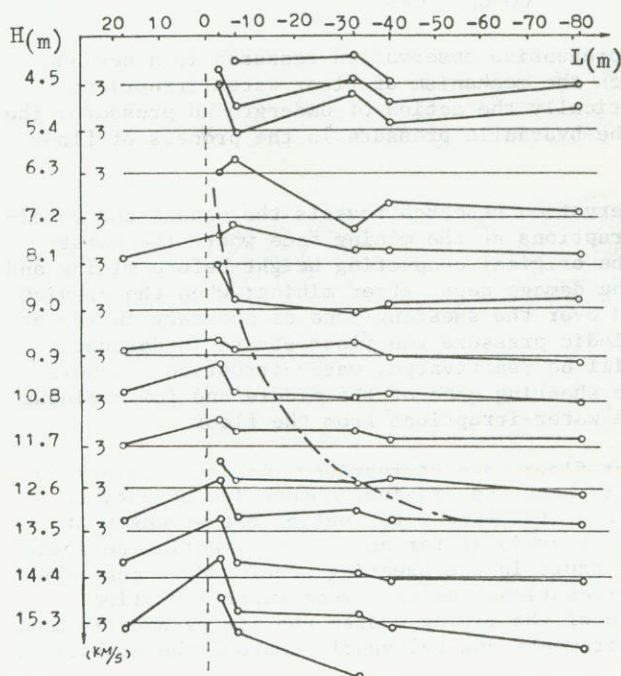


Fig. 6 The relative curve diagram of variety of ultrasound speed in various depth strata with the advancing of mining face in the physical exploration hole No.1

normal floor strata (not structure crevice), in a strained and damaged state the action of hydraulic pressure is weak.

5. Observation results of the hole ultrasound speed. Before mining in the comparison hole the ultrasound speed of various floor strata obtained from practice measuring was 4000-4500 m/sec of limestone, 3500-4000 m/sec of sandy shale and 3000 m/sec of shale. The ultrasound speed increases when the strata is compressed, and drops when the strata is expanded. Figure 6 is the relative curve diagram of the variation of strata ultrasound speed at various depths with the advance of the mining face. From Figure 6 we see: (1) the variation of ultrasound speed clearly reflects the action of underground pressure (especially the periodic pressure) and the stress and strain status of floor strata; (2) the ultrasound speed has a bigger wave in the crevice strain zone of the upper floor strata and the unstable ring of roof limestone of the observation-uphill.

6. Determining the depth of crevice strain zone. According to observation of the hole ground movement it is 13 meters, according to the experiment of water-injection-holes it is 12.8 meters, according to the hole ultrasound observation it is 13.5 meters and according to the observation of radio fluoroscopy in holes it is 14 meters; through comprehensive analysis we obtain 14 meters.

CONCLUSIONS

1. The system of comprehensive observation research is a new and effective way to research the mechanism of floor water-irruptions, and can examine systematically the action of underground pressure, the crevice structure and the hydraulic pressure in the process of floor water-irruptions.

2. Comprehensive observation research reveals the causes and conditions of floor water-irruptions at the mining face where the karst-confined water formed the original conducting height before mining and the floor strata form the damage depth after mining; when the crevice structure is distributed over the shearing zone of boundary inside of the goaf and during periodic pressure the floor strata is damaged, the crevice structure will be reactivated, water-irruption channels are easily formed on the shearing zone of the middle and lower mining face, so the results are water-irruptions from the floor.

3. In order to prevent floor water-irruptions, it is necessary to explore and observe before hand the original conducting height, to forecast the damage depth of the floor, to control the strength of underground pressure and to avoid as far as possible meeting periodic pressure when there is a fault in the shearing zone, and to make preparations about taking precautions against floor water-irruption accidents. The selection of the mining system and its parameters must adopt the aforesaid requirements and relevantly improve the methods of roof control.

In China there are five practical examples of experimental mining and research at the mining face of the seams menaced by karst-confined water of Ordovician limestone, adopting the system of comprehensive observation research, and the results have been used in actual coal production. The further development of this research will obtain further heightening of theoretical and practical research on water-irruptions from the floor.

Chapter 71

DISPLACEMENT DISCONTINUITY METHOD FOR THREE-DIMENSIONAL STRESS ANALYSIS OF TABULAR EXCAVATIONS IN NON-HOMOGENEOUS ROCK

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SUMMARY

This paper describes a numerical method for calculating the three-dimensional stresses and displacements induced by tabular excavations in a rock mass consisting of parallel layers of distinct material properties. The method is based on the superposition of numerical solutions for elementary displacement discontinuities in a parallel layered rock mass.

The method can be used to analyse complicated excavation layouts such as those encountered in room-and-pillar or longwall mining. The method can automatically take into account the effect of the earth's surface.

The method offers an economic and efficient solution to stress analysis problems which cannot be adequately modelled using two-dimensional methods and for which the expense of three-dimensional finite element or general boundary element analysis cannot be justified.

INTRODUCTION

Underground mining often takes place in tabular or lenticular orebodies. In many cases, for example room-and-pillar and longwall mining, the excavation geometry cannot be adequately modelled by two-dimensional methods of analysis. In these cases resort must be made to three-dimensional methods of analysis.

Although in principle three-dimensional finite element models can be used for these geometries, cost limitations have restricted their application to a small number of cases. For example, room-and-pillar systems of regular pattern have been analysed by numerically modelling

the smallest representative part of the geometry subject to boundary conditions that reflect the interaction with the remainder of the system. These analyses required of the order of 400-800 solid finite elements. Models of longwall systems have used 5000-8000 elements (Van Dillen, 1980).

In recent years integral equation or 'boundary element' methods (Banerjee and Butterfield 1982, Crouch and Starfield 1983) have become popular, the main attraction being the brevity of the input data requirements. For a homogeneous rock mass only the excavation surfaces need to be discretized. However, although the boundary element method has been generalized to analyse problems with non-homogeneous rock properties (Wardle and Crotty 1978, Diering 1980) additional elements are required on the interfaces between the homogeneous regions.

In routine excavation design and analysis the cost and effort associated with full three-dimensional modelling cannot be justified. Under such circumstances it becomes necessary to introduce simplifying assumptions that will allow use of a cheaper method of analysis that retains the overall behaviour of the system.

For the analysis of 'tabular' excavations, i.e. excavations that are characterized by their negligible thickness in comparison to their plan dimensions, a special boundary element method known as the displacement discontinuity method has been used successfully (Salamon 1963, Plewman *et al.* 1969, Starfield and Crouch 1973, Brady 1978). This method is based on the assumption that the excavations can be replaced by thin slits of the same plan area as the excavations. The relative movement between roof and floor is treated as a displacement discontinuity. Studies have shown that displacement discontinuity solutions for tabular excavations are of acceptable accuracy for aspect ratios (i.e. ratio of excavation width to height) as low as 5 (Crouch 1976, Brady 1978).

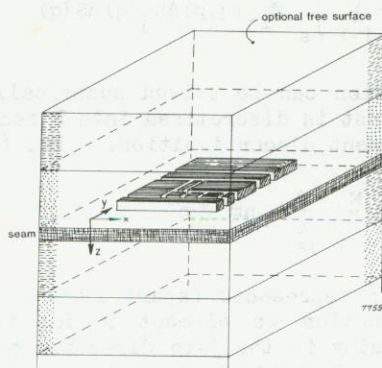


Fig. 1. Excavation layout in non-homogeneous rock mass.

Most displacement discontinuity codes have been restricted to an homogeneous rock mass. This paper describes extensions to the displacement discontinuity method that allows the efficient modelling of tabular excavations in a rock mass composed of parallel layers with distinct elastic properties (Fig. 1). Optionally, the unloaded upper surface of the system can be used to take account of ground surface effects. The method is based on the use of numerical solutions for elementary displacement discontinuities within a parallel layered elastic system. Because the solutions automatically satisfy the interface continuity conditions, the interfaces do not need to be discretized, contrasting with alternative boundary element methods for non-homogeneous media.

NUMERICAL METHOD

Let x -, y - and z - coordinate axes be chosen so that the x - and y -axes lie in the plane of the seam and the z -axis is normal to the seam. If an unloaded top boundary is used the z -axis is directed downwards. The corresponding displacements are u_j ($j=1,2,3$ corresponding to x,y,z) and the stress changes induced by mining are σ_{ij} ($i,j=1,2,3$). The displacement discontinuity components $\Delta u_j(q)$ at q represent the jump in displacement at point q given by

$$\Delta u_j(q) = u_j(q^+) - u_j(q^-) \quad (1)$$

$u_j(q^+)$ represents the heave of the floor and $u_j(q^-)$ the sag of the roof. The normal component Δu_3 is called the closure while the transverse components Δu_1 and Δu_2 are called the ride components.

The tractions (i.e. normal and tangential stress components) introduced by the displacement discontinuity distribution are given by the integral equation (Wardle, 1982).

$$t_i(p) = - \sum_{j=1}^3 \int_S K_{ij}(q,p) \Delta u_j(q) dS(q) \quad (i=1,3) \quad (2)$$

This integral equation can be solved numerically by assuming that the plan area of interest is discretized into N rectangular 'elements' with constant displacement discontinuities. Eq. (2) is replaced by

$$t_i^m = - \sum_{n=1}^N \sum_{j=1}^3 k_{ij}^{nm} \Delta u_j^n \quad (i=1,3) \quad (3)$$

The coefficient k_{ij}^{nm} represents (apart from a change of sign) the i -th component of traction on element m due to a unit constant displacement discontinuity in the j -th direction on element n . For a rock mass consisting of parallel homogeneous isotropic or cross-anisotropic layers, values of k_{ij}^{nm} can be obtained numerically by integral transform methods (Wardle, 1980).

Each element is assigned boundary conditions chosen from the following:

- i) for mined out areas, the total and shear stresses are zero, i.e.

$$\sigma_{i3} + t_i = 0 \quad (i=1,3) \quad (4)$$

- ii) for elements transmitting load between roof and floor, one-dimensional stress-strain relation are used for compression and shear, (Starfield and Crouch, 1973) i.e.

$$\sigma_{i3} + t_i = k_i \Delta u_i \quad (i=1,3) \quad (5)$$

where k_i are the stiffnesses of the seam material:

$$k_1 = k_2 = G/s \quad k_3 = E_s/s \quad (6)$$

where E_s , G and s are the Young's modulus, shear modulus and thickness of the seam.

Equation Solution

Because the coefficient matrices are in general fully populated, direct solution methods are not feasible for large numbers (N) of elements and an iterative method must be used. The method of successive over-relaxation has been found to have adequate convergence properties for three-dimensional displacement discontinuity formulations (Plewman *et al.*, 1969, Starfield and Crouch, 1973) and is used in the present work. A uniform rectangular mesh is used and the effects of blocks of 'remote' elements are 'lumped' together and replaced by equivalent sources, resulting in considerable savings in computational effort and memory requirements. For problems involving excavations at sufficient depth (say, deeper than maximum width) the ride components can usually be ignored, thus reducing computational effort by a factor of 9.

EXAMPLES AND APPLICATIONS

In order to demonstrate the efficiency and accuracy of the new displacement discontinuity method a number of case studies are analysed and comparisons made with previously published finite element solutions. Another case study using the method is described by Wardle and Enever (1983).

Example Problem 1

This problem is drawn from studies of the mechanism of caving of the stiff dolerite sill that commonly overlies South African coal deposits (Salamon *et al.* 1972). The geometry and material properties are given in Figure 2. The displacement discontinuity model involved 20 elements over the width of the excavation. Poisson's ratio of each

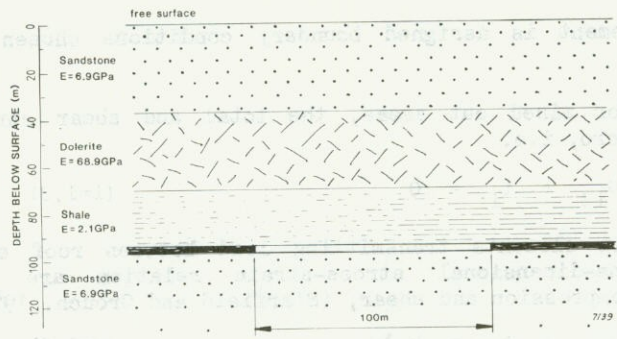


Fig. 2. Example problem 1 : Geometry and material properties

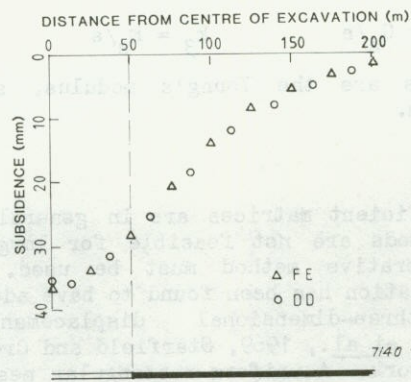


Fig. 3. Example problem 1 : Surface subsidence profiles

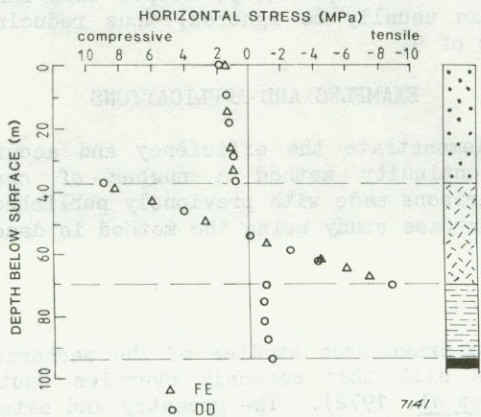


Fig. 4. Example problem 1 : Horizontal stresses over panel centreline

layer was taken to be 0.25; values used in the finite element analysis were not given. Figure 3 shows good agreement between the surface subsidence profiles obtained from the displacement discontinuity and finite element solutions. Figure 4 gives results for the horizontal stress distribution along the vertical centreline of the panel. The high horizontal stresses attracted by the dolerite sill demonstrate the importance of incorporating representative stratigraphic units in numerical models.

Example Problem 2

Results of three-dimensional finite element models of longwall layouts are given by Van Dillen (1980). The particular problem chosen for analysis by the displacement discontinuity method relates to 'Test Area 5' of the Sunnyside No. 1 mine (Fig. 5). The depth of cover is 455 m. The finite element model used 5120 solid elements. The top and bottom boundaries of the mesh were at distances of 54 m and 40 m from the excavation plane, respectively. Rollers were used on all four sides and the base.

Because the displacement discontinuity program does not exploit symmetry all four quadrants were modelled. The mesh consisted of 4800 square elements of side 3.25 m.

Figure 6 compares the vertical stress distributions along a number of sections parallel to the face. Figure 7 shows results obtained from the displacement discontinuity model for the vertical stresses on the plane corresponding to the top boundary of the finite element model which was loaded uniformly. It can be seen that the uniform loading assumption is erroneous. The fact that the finite element vertical stress results on the excavation plane (Fig. 6) are consistently higher than the displacement discontinuity results is probably due to the artificially higher total load used in the finite element model.

CONCLUSIONS

The displacement discontinuity method has been shown to give results comparable with alternative methods. For three-dimensional problems it offers economical and reliable solutions. The geometrical input data for the program is minimal consisting solely of the excavation layout (the property of each element is defined by a single character code) and layer thicknesses. Unlike finite element models, artificial remote boundaries are not required.

The method is ideally suited to the routine design of mine layouts in tabular orebodies because of the ease with which extraction patterns can be changed.

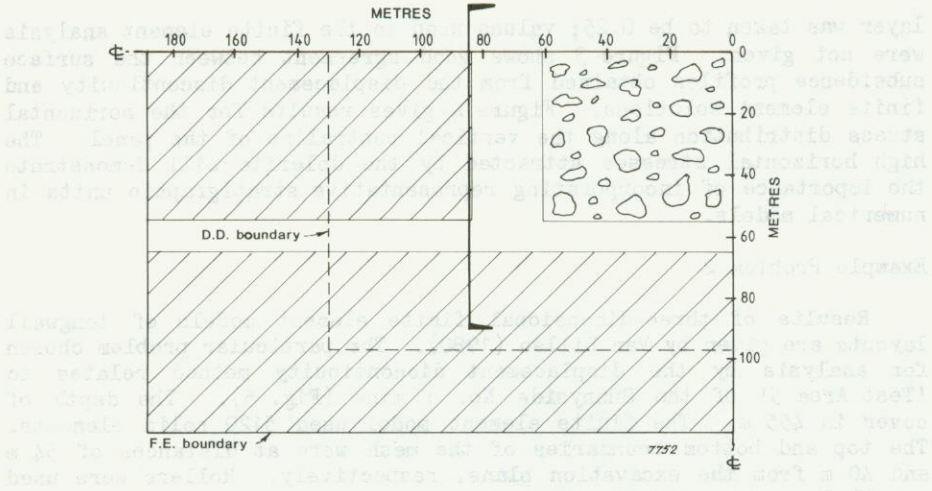


Fig. 5. Example problem 2 : Geometry

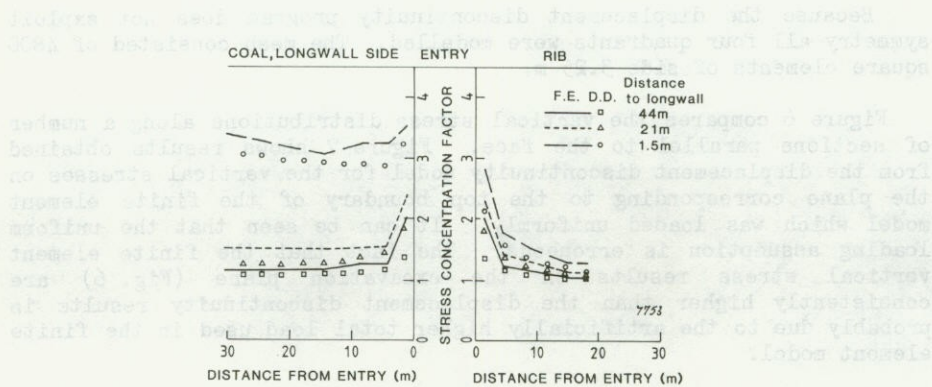


Fig. 6. Example problem 2 : Vertical stresses on excavation plane

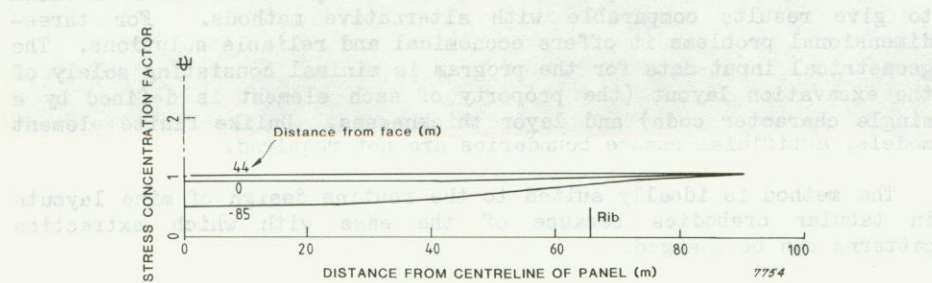


Fig. 7. Example problem 2 : Vertical stresses 54 m above excavation plane (displacement discontinuity model)

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MINING INDUCED SUBSIDENCE

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15

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Chapter 72

RECENT DEVELOPMENTS IN SUBSIDENCE PREDICTION AND CONTROL FOR THE EASTERN U.S. COALFIELDS

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ABSTRACT

Underground mining will disturb the natural equilibrium of the rock mass, causing significant stress redistributions in the vicinity of the excavation with corresponding horizontal and vertical displacements. Subsidence of the ground occurs when these displacements propagate from the mine opening, through the overlying strata, to the surface. Such ground movements may take either of two general forms: a sudden and sometimes violent collapse, or a more gradual and progressive ground movement. The former, characterized by localized failure of roof or pillar in old, shallow room and pillar workings, is rather unpredictable and difficult to analyze. As a result, most control measures are directed toward the latter type of movement, which is associated with the fracture and flow of the strata overlying longwall and high-extraction room and pillar systems, resulting in horizontal and vertical strains in the affected area.

Such ground movements can often cause considerable surface disturbances ranging from simple land settlement to severe structural damage. Due to the increasing incidence of this phenomenon in more populated areas and the rising awareness of the problem, it has become necessary to establish a balance between the development of mineral resources and the protection of the environment and society. This paper presents a summary of the major findings derived from a substantial research effort on mining subsidence carried out, during the past few years, in the southern Appalachian coalfield. The research program encompassed the following major tasks: identification of the most significant factors influencing ground movements above mined openings, collection of subsidence case studies for the southern Appalachian coalfield, development of subsidence and strain prediction techniques for that region and initiation of a subsidence monitoring program in southwestern Virginia.

SUBSIDENCE PREDICTION

Due to the lack of subsidence information for the eastern U.S. coalfields, the preliminary establishment of a data bank on surface settlement was a prerequisite before regional subsidence characteristics could be determined. Since the number of parameters involved in describing ground movements above longwall mining seemed to be less than those necessary for the characterization of room and pillar subsidence, the collection process was first directed toward the compilation of longwall case studies. All relevant published information was collected and numerous coal companies were contacted to contribute any unpublished information that might be of interest to this study. This data collection phase generated 34 longwall case studies from the Appalachian region. The more important parameters involved in subsidence modeling are given in Table 1 for these case histories.

Through analytical and statistical investigation of this data, the significant subsidence relationships were determined, including the dependence of the angle of draw, the maximum subsidence factor and the location of the inflection point on the width-to-depth ratio. In addition, a correlation between the maximum subsidence and the geology of the superincumbent strata was developed. From these relationships, the dependence of subsidence on the panel geometry and amount of hardrock (sandstone and limestone) in the overburden was formulated (Figure 1).

Once these subsidence relationships were established, the development of a longwall subsidence prediction model was attempted. However, before an empirical prediction model could be developed for the Appalachian region, it was necessary to determine the characteristic subsidence profiles. Thus, all case studies were divided into either subcritical or critical (including supercritical) extractions. From these profiles a mean value curve was calculated. A hyperbolic tangent function was then chosen to mathematically describe these curves, where the subsidence at any point, $s(x)$, is given as:

$$s(x) = 0.5 S[1 - \tanh(cx/B)] \quad (1)$$

where,

x = distance from the inflection point to the point in question,

S = maximum subsidence for the profile,

B = distance from the inflection point to S ,

c = constant, 1.4 for subcritical panels,
1.8 for critical and supercritical panels.

TABLE 1. Subsidence Parameters for Collected Case Studies

NUMBER	WIDTH, W	DEPTH, h	W/h	SEAM THICKNESS, m	S/m	ZHR	LOCATION	REFERENCES
1a	140	185	.75	1.7	.48	37	WV	Adamek & Jeran (1981, 1982)
1b	140	185	.75	1.7	.35	37		Adamek & Jeran (1981, 1982)
1c	185	201	1.7	1.7	.59	23		Unpublished
1d	185	168	1.10	1.7	.73	23		Unpublished
1e	185	185	1.00	1.7	.74	37		Unpublished
2	185	165	1.12	1.4	.47	41	PA	Unpublished
3	150	70	2.15	1.4	.52	3	PA	Unpublished
4a	140	457	.31	1.6	.05	30	AL	Unpublished
4b	335	457	.73	1.6	.26	30		Unpublished
5a	150	88	1.71	1.5	.26	58	WV	Unpublished
5b	150	78	1.92	1.5	.31	58		Unpublished
6a	150	90	1.67	1.5	.60	32	WV	Unpublished
6b	150	75	2.00	1.5	.52	32		Unpublished
7a	135	150	.90	1.2	.32	33	WV	Unpublished
7b	140	138	1.01	1.2	.31	39		Unpublished
7c	135	158	.85	1.3	.31	28		Unpublished
7d	135	85	1.59	1.2	.56	26		Unpublished
8	135	144	.94	2.1	.59	23	WV	Unpublished
9	180	150	1.20	1.37	.64	26	AL	Unpublished
10	675	520	1.30	--	.20	47	VA	Von Schonfeldt et al. (1980)
11a	135	196	.69	--	.38	55	WV	Von Schonfeldt et al. (1980)
11b	105	188	.56	--	.35	51		Von Schonfeldt et al. (1980)
12	135	225	.60	1.8	.44	--	WV	Choi & McCain (1979)
13	135	185	.73	1.8	.36	41	WV	Dahl & Choi (1973)
14	130	118	1.10	1.6	.65	25	PA	Grayson & Mishra (1982)
15	145	121	1.20	1.1	.65	21	PA	Hardy et al. (1977)
16a	150	231	.65	2.0	.61	23	OH	Roscoe (1981)
16b	150	136	1.10	2.0	.64	23		Roscoe (1981)
16c	145	107	1.35	2.0	.59	23		Roscoe (1981)
16d	145	210	.69	1.3	.58	23		Roscoe (1981)
16e	145	210	.69	2.2	.65	22		Conroy & Gyarmaty (1983)
16f	145	210	.69	2.2	.65	22		Conroy & Gyarmaty (1983)
17a	185	231	.80	1.7	.62	25	OH	Adamek & Jeran (1981) and Tandandand & Powell (1982)
17b	185	210	.88	1.7	.58	29		

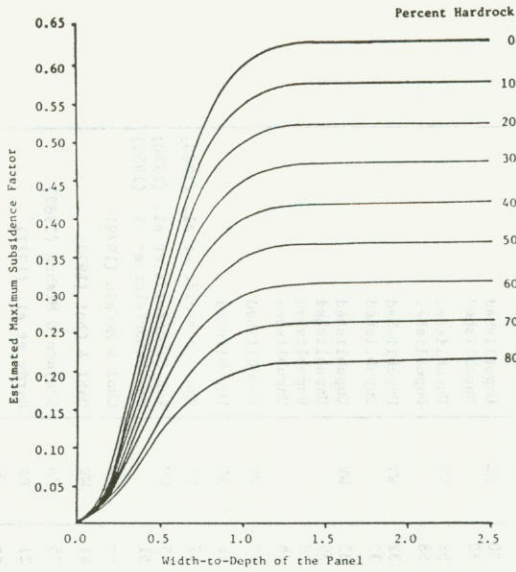


FIGURE 1. Dependence of the subsidence factor on the width-to-depth ratio and percent hardrock.

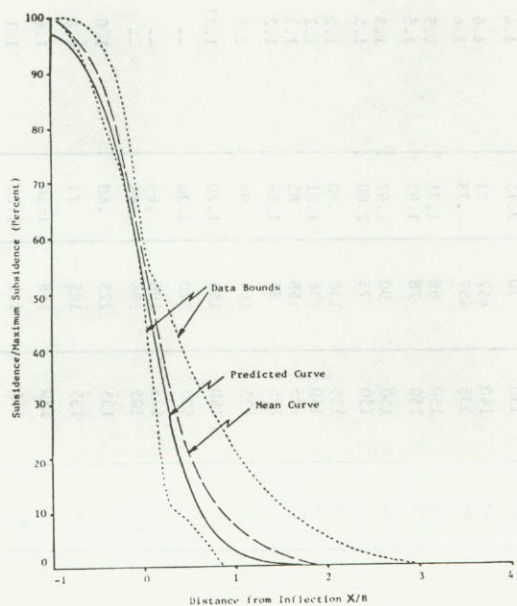


FIGURE 2. Hyperbolic tangent function for critical and supercritical panels.

Figure 2 shows the hyperbolic tangent function along with the mean value curve for critical and supercritical panels. A similar correlation was found for subcritical extractions. Once these functions were known, it was possible to predict the complete subsidence profile above a longwall panel in Appalachia (Karmis et al., 1983).

In order to form a complete subsidence prediction program, this research also concentrated on ground movements above room and pillar extractions. Once the longwall model was complete and general subsidence correlations were determined, the relationships were expanded and applied to room and pillar situations. Furthermore, over 60 individual room and pillar case studies were acquired through a literature review and correspondence with industry and government sources. To formulate a room and pillar subsidence prediction model, a technique based on influence functions was employed. This technique, known as the zone area method, was first developed for longwall extractions. The model was then modified and applied in conjunction with statistically and analytically determined subsidence relationships to yield the room and pillar prediction method (Karmis et al., 1984a and 1984b).

STRAIN PREDICTION

The ultimate goal of any ground movement analysis should be the prediction of damage to surface structures. Since horizontal strains are the primary cause of structural damage, this research also stressed the formulation of a strain prediction model. However, due to the lack of strain data, an empirical strain prediction method could not be developed. Therefore, it was necessary to develop the strain prediction technique using theoretically as well as empirically derived principles.

Since the strain is a function of curvature, which in turn is determined by the shape of the profile, the horizontal strain at any point can be found if an appropriate subsidence function is assumed and a relationship between curvature and strain is determined. In order to develop the latter relationship, several known formulations from various coalfields were studied and compared to strain data available from the United States. Figure 3 shows the plot of strain versus curvature for the Silesian coalfield in Poland, the Chinese and British coalfields along with the U.S. data. Through a statistical analysis of the U.S. data, it was found that:

$$\text{strain} = 0.92 (\text{curvature})^{1/2} \dots \dots \dots (2)$$

Furthermore, as previously explained, a subsidence function has been determined for the Appalachian region as given by Equation 1. Since

the subsidence of any point along the profile is given as a function of x , the curvature of the profile can be found by taking the second derivative of $s(x)$ with respect to x in the hyperbolic tangent equation. Thus, the strain at any point along a subsidence profile above a longwall panel in Appalachia is given by (Karmis et al., 1983):

$$e(x) = 0.92 \frac{c}{B} [S \operatorname{sech}^2(cx/B) \tanh(cx/B)]^{1/2} \dots \dots \dots (3)$$

SUBSIDENCE MONITORING

Due to the sparsity of subsidence and strain data, every available case study had to be used in the development of the subsidence and strain prediction methods; therefore, the models could not be properly tested. To help validate the prediction techniques and to allow the compilation of a comprehensive subsidence and strain data bank for both longwall and room and pillar mining, a systematic vertical and horizontal measurement program has been initiated for underground mines in southwestern Virginia. Table 2 gives subsidence and strain values for one longwall and three room and pillar mines where vertical and horizontal movements are currently being monitored. It should be noted that the surface movements have not reached equilibrium for several of these mines; therefore, the values corresponding to complete subsidence may be different.

These panels are monitored using longitudinal and transverse monument lines with 2.54 cm diameter steel rods ranging in lengths from 0.6 to 1.5 m. The monuments are positioned using the most widely accepted criteria, and the movements are measured using a high-accuracy electronic computing tacheometer with built-in EDM which converts measured data to coordinates and elevation. All data, measured and computed, are recorded and transferred to an HP-86 computer system enabling permanent storage and data analysis, including plotting capabilities. This total monitoring program should allow the accurate and efficient accumulation of data on vertical and horizontal surface movements in Appalachia (Karmis et al., 1984a).

CONCLUSIONS

The expansion of underground coal mining into more populous areas, and the resultant increase in the potential for surface and structural damage, have rendered the formulation of accurate surface deformation models a prerequisite. In this research, a comprehensive effort was made to develop subsidence and strain models for the Appalachian region. Although preliminary testing does show these models to be accurate for the Appalachian region, it is not suggested that these models have assumed their final form. The prediction methods can easily be adapted to accommodate the addition of new data. It should also be noted that the procedures used in the development of the

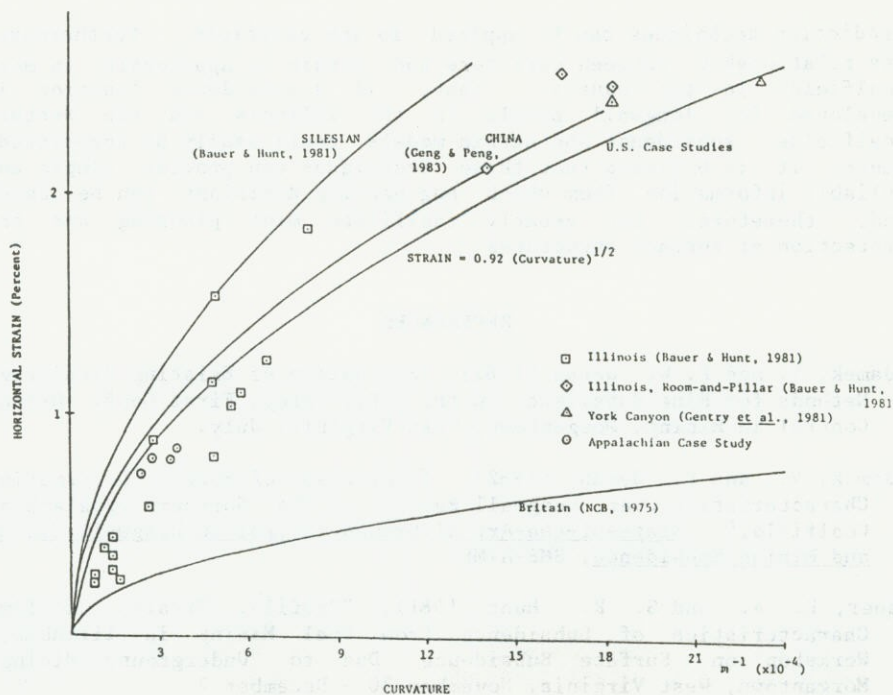


FIGURE 3. Relationship between strain and curvature.

TABLE 2. Subsidence Parameters for Three Room and Pillar and One Longwall* Panel in Virginia

NUMBER	WIDTH, W	DEPTH, h	W/h	SEAM THICKNESS, m	S/m	ZHR
1	135	165	.82	1.5	.40	32
2	126	105	1.20	1.27	.43	80
3	90	66	1.36	1.27	.56	80
4*	180	270	.67	2.16	.44	55

prediction techniques can be applied in any coalfield. Furthermore, the relationship between curvature and strain is applicable to most coalfields in the country. Thus, if a subsidence function is developed for longwall panels in the Illinois and the Western coalfields, subsidence and strain models could easily be formulated. Hence, it is believed that these techniques can provide simple and reliable information from which engineering decisions can be based and, therefore, can greatly facilitate mine planning and the protection of surface structures.

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Chapter 73

A METHOD FOR ASSESSING THE POTENTIAL OF MINE SUBSIDENCE AT ABANDONED MINE SITES THROUGH THE ASSISTANCE OF FINITE ELEMENT MODELING

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ABSTRACT

Computer assisted finite element model analysis, originally developed to study stress distributions in aircraft, has been modified into a versatile mining program, that can be used to assist the engineer with predictions of mine subsidence. International Exploration, Inc. has developed an in-house finite element model program which incorporates a multitude of factors influencing mine subsidence such as mine design, physical rock properties, structure, faults, joints and topography. Finite element analysis of a mine model can mathematically determine the magnitude and direction of the principal stresses within each of the elements forming the model. By comparing the calculated in-situ stresses to the actual rock strengths, high risk regions within a mine can be identified. These potentially unstable regions of the mine can be allowed to fail within the model and the resultant stress redistribution and element movement calculated. Thus, the effects of the mine collapse at the surface can be predicted for the purpose of determining the potential damage, if any, that could occur to surface structures from deep mine failure. Stabilization materials can also be mathematically inserted into the model to design and analyze the effectiveness of subsidence prevention programs. Two examples are presented of the finite element program being used to predict mine subsidence at substantially different geologic and mining provinces in Pennsylvania.

INTRODUCTION

Abandoned subsurface coal mines in Pennsylvania have posed a substantial problem to hundreds of property owners situated above them. The mines may collapse, with time, from various causes such as weathering, pillar robbery or mine flooding. The resulting subsurface collapse may cause significant surface movement of a magnitude and direction capable of damaging surface structures. As a result, mine stabilization projects are faced with determining which areas within hundreds of square miles warrant special consideration. Through the use of computer assisted finite element modeling, specific high risk areas can be delineated and recommendations suggested for developing strategies to prevent future damage to surface structures.

Using finite element modeling, the mining and geologic conditions of a specific study site are mathematically recreated to calculate the stresses present within mine pillars and rooms. Over stressed areas within the model can then be caused to fail and the resulting magnitude and direction of surface movement and stress then determined. The mathematical model can be modified to consider the effects of pillar decay and robbery as well as introduce faults, secondary mining and stabilization materials. The ability of the program to calculate the stress conditions within a mine under the complex situations commonly found in Pennsylvania greatly exceeds the capabilities of empirical approaches.

FINITE ELEMENT MODELING

The finite element method is a general mathematical technique of structural analysis in which the body to be studied is subdivided into individual structural elements, in this case, triangles (Zienkiewicz, 1977) (Figure 1). The triangles are interconnected at their corners or nodes. When forces are applied to the body, the nodes displace, and the triangles experience strain. The amount of displacement of each triangle depends on the level of the forces applied to the triangles and the material properties of each triangle.

Although the method is not restricted by the type of material to be studied, elastic or elastic-plastic rheologies provide the best subsidence model results (Scott, 1978). Each nodal point has a force vector acting upon the node, each element may have its own individual stiffness, and each nodal point may experience displacement. Forces can be applied to the boundaries of a problem or

act as body (gravitational) forces within each element. Since the applied forces as well as the material and geometric properties of each element are generally known, the only unknown is the displacement of each nodal point.

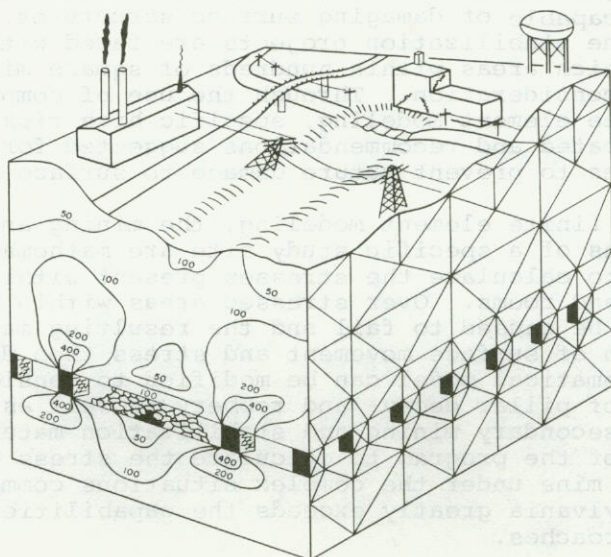


Figure 1

Depiction of finite element model, showing element and node configurations (right) and resultant shear stress plot (left).

In most problems, the number of triangles subdividing the elastic continuum number ranges in the hundreds. Therefore, large amounts of computer time and central memory are required. The computer inverts the stiffness matrix for each element and then calculates the displacement at each node and the stress and strain in each element. The magnitudes of the principal stresses and their directions are then calculated so that the stress and strain within each elastic element is known (Figure 1).

Any number of unequally shaped or spaced openings can be computer-modelled by subdividing the problem into small triangles. Since any type of rheological behavior can be incorporated into the analysis, the finite element method is ideally suited to the study of subsidence problems involving multiple underground openings. Furthermore, geologic layering of various rock types and dipping beds can easily be incorporated into the analysis. The INTEX program has been modified to include discontinuities in the form of faults or joints. Therefore, the INTEX finite element program is a powerful tool that can be used to study the complex geometric and geologic relationships that exist around abandoned mine workings.

APPLICATION OF THE FINITE ELEMENT PROGRAM

Two detailed mine subsidence studies of significantly different geology and mine settings were undertaken, incorporating the use of finite element modeling. These studies were made for the Pennsylvania Department of Environmental Resources Division of Abandoned Mine Reclamation (Contract No. SL 455-101.2). Site number one is an abandoned rib pillar coal mine located in the Bituminous coal region of western Pennsylvania. Site two is an abandoned room and pillar coal mine which contains multiple coal seams and is within the Anthracite coal region of north-eastern Pennsylvania.

Site number one is underlain by a 1.07 meter (3.5 foot) thick coal seam which was mined 61 meters (200 feet) beneath the site using the rib pillar method of extraction. The majority of the rib pillars were found to be marginally capable of providing support. A channel sandstone lens within the roof of the mine is believed to have influenced collapse of specific pillars at the site.

Computer modeling of the study area revealed the importance of a shallow sandstone unit in controlling surface subsidence. The relatively stronger sandstone unit acts as a supportive beam within the weaker shales, significantly limiting surface subsidence. Computer results accurately predicted an angle of draw of 48 degrees with a maximum subsidence of .22 meters (.73 feet). Actual subsidence at the site has an angle of draw of 46 degrees with maximum subsidence of .24 meters (.80 feet). Surface subsidence movement is subtle yet was significant enough to cause damage to larger structures oriented parallel to the direction of tensile stress. Should the sandstone have been unable to support the underlying collapsed void, finite element modeling indicates that maximum subsidence

would have been greater.

At the second site, two separate coal seams located 7.6 and 15.2 meters (25 and 50 feet) beneath the surface were mined. Supportive pillars were later removed from the mines except from beneath surface structures where pillars were left as protection against subsidence damage. Finite element modeling of the mines indicates that the remaining coal pillars and roof rock are capable of supporting the stresses within the mine. Modeling shows that the major parameter influencing subsidence at the site is primary jointing and that significant horizontal separation of the joints would most likely occur at the periphery of the pillar region. As the roof within the robbed area of the mine collapses, a cantilevering of the overlying rock occurs. This results in a surface separation of joints above the pillars at the periphery of the protected region causing large fissures to open at the surface. Thus, pillar collapse is not necessary for subsidence damage to occur. Surface observations and core-hole investigations confirm the finite element model results.

CONCLUSIONS

The analysis of subsidence prone sites reveals the importance of understanding and applying all of the parameters influencing mine subsidence. Finite element modeling is capable of incorporating the parameters specific to each site which is an important shortcoming of empirical methods. The finite element model's usefulness in interpreting the in-situ stresses within mines and the surface subsidence anticipated over mined regions is exemplified in the increased efficiency and effectiveness of subsidence prevention programs.

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Chapter 74

GROUND MOVEMENTS ASSOCIATED WITH PILLAR EXTRACTION COAL MINING IN NORTHERN WEST VIRGINIA

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ABSTRACT

An investigation was made of ground response to pillar retreat mining in a 1.7 meter thick seam at 108 meter depth at a site in northern West Virginia. This paper describes mining-related stress changes and movements at mine level, displacements within the overburden, ground surface subsidence and groundwater level variations. Subsidence approached 40 per cent of the mine height and continued more than a year after mining. Water levels declined in certain deep-lying strata.

INTRODUCTION

Pillar extraction has a long history of use in U.S. coal fields and, even though longwall mining is becoming increasingly popular, still accounts for a sizable proportion of the coal mined. Ground movements associated with pillar extraction are sometimes large enough to disrupt the ground water regime as well as damage buildings and other surface structures. The federal government has recognized the need for a better understanding of subsidence processes so that a greater compatibility can be achieved between coal mining and current and future land use. To further that understanding, GAI Consultants, Inc., under contract to the U.S. Bureau of Mines and U.S. Department of Energy, is conducting a study of subsidence at Kitt Energy Corporation's Kitt No. 1 Mine in northern West Virginia. This paper presents the initial findings. Further reports will follow.

SITE DESCRIPTION

The monitoring site occupies approximately 0.26 square kilometers of rolling grasslands having a maximum relief of 55 meters. The mine is located in the nearly flat-lying 1.7 meter Lower Kittanning Coal approximately 180 meters below ground surface. Overburden strata (Figure 1) are of the Pennsylvanian age Allegheny and Conemaugh Groups and consist of alternating units of claystone, siltstone, shale and sandstone, along with a few thin coal units. One-third to one-and-one half meters of colluvial and residual soils mantle much of the site. Alluvial sediment three to five meters thick occupies the topographically low, south-central portion of the site.

MINING

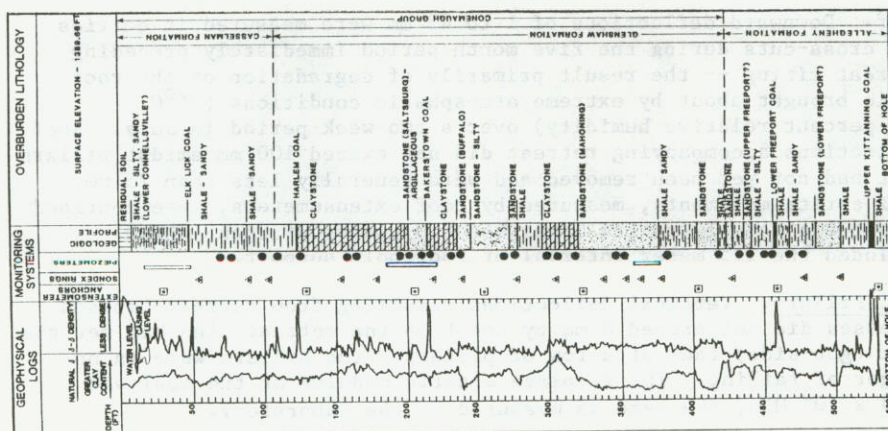
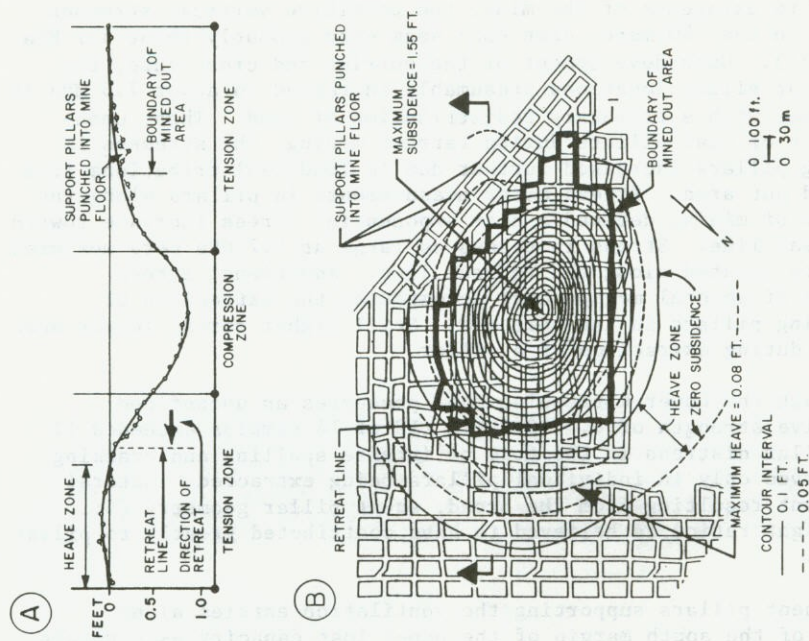
The room and pillar system employed at the mine permitted almost total (>90 percent) removal of coal from the panel. Developmental (advance) mining consisted of driving a series of 5.2-meter wide entries on 18-meter centers from the mains to the far end of the panel, joining them on 27-meter centers with crosscuts of the same width. Ventilation entries were driven along the south and east margins of the panel. Retreat mining began with Pillar 1 (Figure 2) and progressed row by row toward the main entries. Pillars were extracted in pairs. All coal was removed with a single continuous miner. Developmental mining took nine months; retreat mining six months. The retreat rate was relatively steady. Coal in adjoining panels was unmined. No prior mining had been conducted in seams above or below the panel.

INSTRUMENTATION

A variety of devices were installed at the monitoring site to measure displacements, stresses and water levels: at mine level -- convergence points, roof extensometers, instrumented roof bolts and stress meters; in the overburden -- inclinometers, time domain reflectometry cables, multiple position borehole extensometers, and piezometers; at ground surface -- survey monuments and benchmarks. Details are discussed in Speck and Bruhn, 1983.

FINDINGS

The recovery of coal pillars led to redistribution of the overburden load, resulting in intensified stresses in remaining pillars, compression of the pillars, convergence of entries, and shearing, fissuring and deflection of the overburden strata. As a result, water levels in certain water-bearing strata declined and a broad, shallow depression formed at ground surface, its areal extent and depth increasing with the dimensions of the mined-out area.



Stress Changes in the Coal Seam

Prior to existence of the mine, the primitive vertical stresses existing in the 180-meter deep coal seam were probably about 4.1 MPa (Figure 3A). Upon development of the entries and cross-cuts, the stresses at pillar locations presumably increased to about 7.6 MPa in conformance with a tributary redistribution of load. Then, upon extraction of coal pillars during retreat mining, the stresses in remaining pillars increased further due to load redistributions from the mined out area. Stress meter measurements in pillars along the direction of mining defined a near exponential stress increase toward the retreat line. Stress increases as large as 5.7 MPa were measured in pillars located along the retreat line. Additional stress increases of several megapascals accompanied the extraction of neighboring pillars in the same row. Still higher stress levels were achieved during extraction of a pillar.

Although the Lower Kittanning Coal possesses an unconfined compressive strength of 6.9-19.3 MPa (12 of 14 samples exceeded 11 MPa), pillar distress in the form of intense spalling and cracking was observed only in individual pillars being extracted. Lateral confinement resulting from the broad, squat pillar geometry (5:1 width/height ratio) is believed to have contributed greatly to pillar capacity.

Permanent pillars supporting the ventilation entries along portions of the south margin of the panel lost capacity as a result of bearing failure of the underclay floor and roof instability (Figure 3B). This ultimately influenced the geometry of the subsidence trough (Figure 2).

Mine Level Ground Movements

Roof. Downward deflections of 1 to 43 mm were measured in entries and cross-cuts during the five month period immediately preceding retreat mining -- the result primarily of degradation of the roof shale brought about by extreme atmospheric conditions (35°C, 100 percent relative humidity) over a two-week period in July. Roof deflections accompanying retreat did not exceed 100 mm amidst pillars that had not yet been removed and were generally less than 25 mm. Roof strata movements, measured by roof extensometers, were confined almost exclusively to the lowermost 2.5 meters of rock, which included the 1.5 meter interval of roof bolt support.

Coal Pillars. Vertical deflections resulting from retreat-imposed stresses did not exceed 8 mm by the time the retreat line had met the back (gob-side) face of a row of pillars. The pillars were in no danger of failing. The in-place elastic modulus of the coal was 1.93×10^3 MPa, the same as measured in the laboratory.

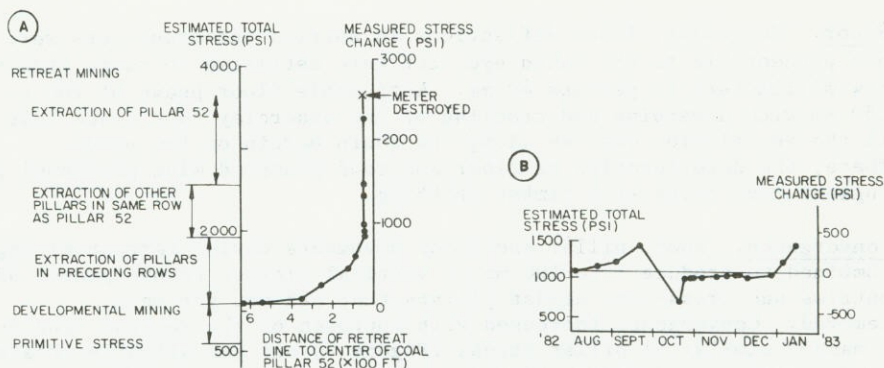


Fig. 3. Vertical stress in coal pillars.

A. Pillar 52 within panel.

B. Support pillar along south ventilation entries that experienced bearing failure.
(1 psi = 6.895 kPa)

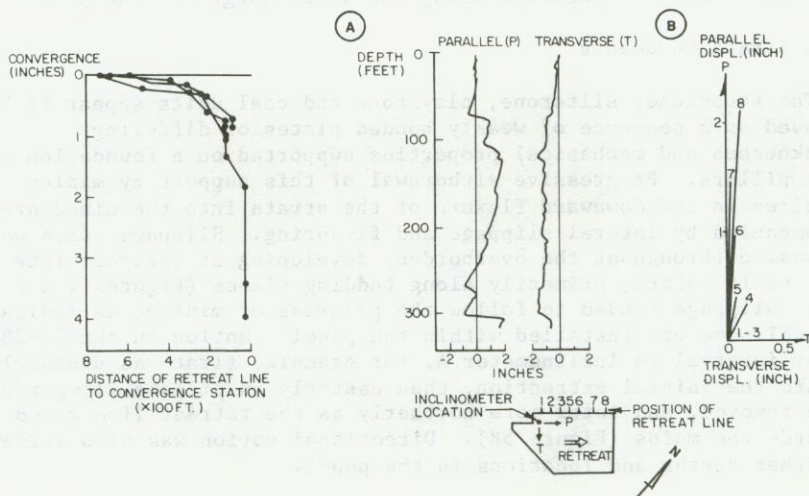


Fig. 4. Convergence of mine openings
(1 in. = 25.4mm)

Fig. 5. Lateral movements within overburden, Inclinator A.
A. Displacements with depth.
B. Slip in the 106-126 ft. (32-38 m.) interval. (1 ft. = 0.3m)

Floor. Generally, floor deflections in entries and cross-cuts were not perceptible to the naked eye, but were estimated to range from a few millimeters to perhaps 40 mm. Noticeable floor heave of 100 to 150 mm with upwarping and cracking of the underclay took place only in the ventilation entries along the south margin of the panel. There, the deterioration of floor and roof prompted mine personnel to augment roof bolts with timber cribbing.

Convergence. Roof, pillar and floor movements during retreat mining combined to produce 1 to 100 mm of vertical closure (convergence) of entries and cross-cuts amidst pillars that had not yet been removed. Convergence increased with approach of the retreat line in a manner similar to pillar stress (Figure 4) and was slightly greater in four-way intersections than in entries or cross-cuts midway between intersections. The relative contributions of roof, pillar and floor to convergence were somewhat variable, probably due to local variations in geology, mechanical properties and stress field. These local variations in convergence of mine openings, however, are considered less important to the subsidence process than variations in the load-carrying capacity of the roof/pillar/floor system, particularly those deriving from local bearing failure of the mine floor such as occurred along the south margin of the panel.

Rock Strata Movements

The sandstone, siltstone, claystone and coal units appear to have behaved as a sequence of weakly bonded plates of differing thicknesses and mechanical properties supported on a foundation of coal pillars. Progressive withdrawal of this support by mining resulted in the downward flexure of the strata into the mined area, accompanied by lateral slippage and fissuring. Slippage zones were pervasive throughout the overburden, developing at vertical intervals of 1 to 15 meters, primarily along bedding planes (Figures 1 and 5A). Slippage tended to follow the progress of mining, as indicated by inclinometers installed within the panel. Motion in the 32-38 meter interval in Inclinometer A, for example, first was southerly toward the initial extraction, then easterly as pillars alongside were removed, and later more northerly as the retreat line moved towards the mains (Figure 5B). Directional motion was also evidenced at other depths and locations in the panel.

Local strata dislocations of 25 mm or more limited the inclinometer sonde to successively lesser depths as mining progressed. Owing to the large spans and the low tensile and shear strengths along slip surfaces, bedding separations may have occurred at or below the slip surfaces. The strata dislocations precluded detection of any separations using the wire extensometers or Sondex (magnetic induction) device.

Groundwater Response

Water levels in the six piezometers located above the mine panel -- two in each of three water-bearing strata (Figure 1) -- remained unchanged during the nine month period of developmental mining. Later, during retreat mining (Figure 6), water levels in the Lower Connellsville Sandstone at 15 meter depth again remained unchanged. Water levels in the Saltsburg and Mahoning Sandstones began declining almost as soon as retreat mining began. Those in the Saltsburg Sandstone at 61 meter depth declined 21 meters and then recovered to elevations 11 to 15 meters below premining levels. Water levels in the Mahoning Sandstone at 98 meter depth dropped 82 meters and remained depressed over the one year post-mining monitoring period. These observations are consistent with a characterization of the overburden as a downward deflecting, thick plate whose upper fibers are in compression and whose lower fibers are in extension.

Field permeability tests conducted on several occasions during mining indicated no significant hydraulic conductivity changes in the three sandstone strata. However, flexure and slippage probably caused at least local (perhaps temporary) increases in the hydraulic conductivities of intervening beds of claystone, shale and siltstone.

Water inflows to the mine panel were first recognized a month-and-one half after retreat mining began and continued up to and beyond the completion of mining four-and-one half months later. The rate of flow into the panel was virtually constant over the period of observation. Six and one-half months after mining ended, mine personnel reported water emanating from the panel at the main entries.

Ground Surface Movements

Developmental mining produced no perceptible movement of the ground surface. Retreat mining produced a dish-shaped depression, or trough, that increased in depth and areal extent as the mined out area enlarged and eventually encompassed the entire mine panel (Figure 2B).

At no time was subsidence perceptible with the naked eye. No ground cracks appeared, even in zones of known ground extension. No modifications to ground slope or lowering of the ground surface could be discerned within the mildly hilly topography of this typical Northern Appalachian site. Subsidence was detected by survey only after the mined-out area had reached dimensions of approximately 115 meters square. Maximum subsidence at that time was 25 mm. Maximum subsidence at the completion of retreat mining was 0.53 meter, corresponding to 32 percent of the mined thickness. Movements in the horizontal plane were as great as 90 mm.

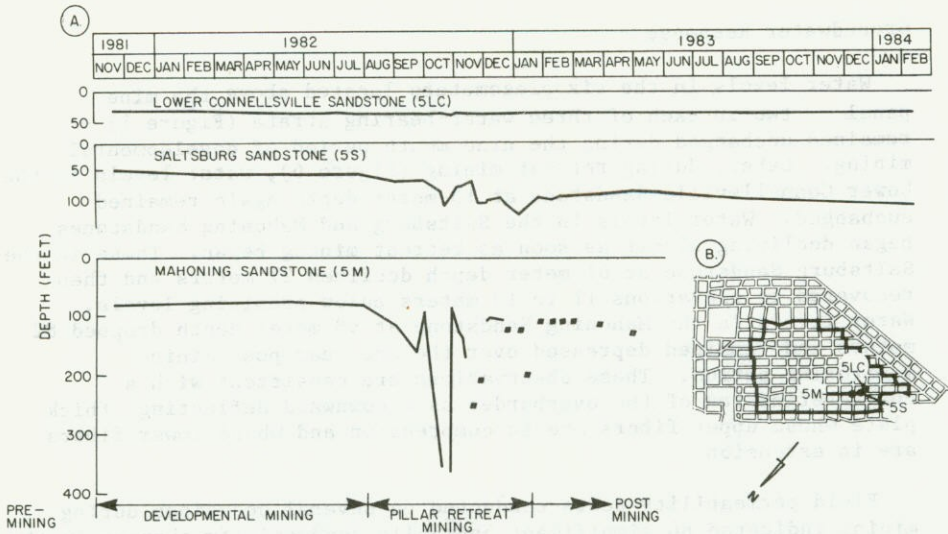


Fig. 6. Water level variations during mining
 A. Water Levels B. Piezometer Location Map
 Stratum locations are shown in Fig. 1.
 Piezometer casing dislocation depths are denoted by ▴.

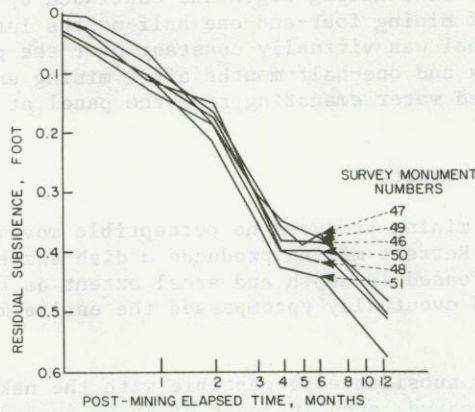


Fig. 7. Residual subsidence at center of trough.

Bounding the mined-out area to a distance of approximately 100 meters (0.58H) was a heave zone whose amplitude was at most 25 mm (Figure 2B). The heave zone is consistent with a characterization of the overburden as a broad, thick plate spanning across elastic abutments.

The mine panel was not of critical dimensions. Consequently, additional subsidence would be expected if the dimensions of the panel were increased. However, subsidence data began replicating in the direction of retreat mining after the mined-out area had reached a length of 110 meters (0.6H), suggesting that near limiting conditions had been achieved at the site relative to the 180-meter width of the panel.

Subsidence parameters, such as angle of draw, ground strain, etc. depended both on location above the panel and the stage of mining. Tensile and compressive ground strains reached 0.005 locally. At the end of mining, the angle of draw ranged from 8 to 15 degrees where the margin of the panel remained firm to as much as 28 degrees where the margin experienced bearing failure.

Time-dependent ground movements took place while mining was underway, as evidenced by surveys made during lulls in mining. At least some of time-dependent movements were the result of pillars along the margin of the panel punching into the mine floor. The rate of "residual" subsidence first appeared to diminish about four months after the completion of mining (Figure 7). By that time, maximum subsidence had increased 0.12 meter (about 24 percent) in excess of the 0.53 meter measured at the completion of mining. Further observations have shown subsidence to be continuing. When residual subsidence will become insignificant is not certain.

CLOSURE

This paper has presented the initial findings of an investigation of ground response to pillar retreat mining in a 1.7 meter thick seam at 180 meter depth in northern West Virginia. These data and observations, in combination with analyses to be reported at a later date, promise to improve our understanding of the subsidence process and our ability to deal with its potential impacts on surface structures and the environment.

ACKNOWLEDGEMENTS

This report is based on work conducted by GAI Consultants, Inc., for the U. S. Department of Energy [DOE] (Contract DE-AC22-80PC30120) and the U. S. Bureau of Mines [USBM] (Contract J0233920). William Eichfeld served as Technical Project Officer for the DOE. Timothy

Hackett and Vladimir Adamek served in succession as TPO for the USBM. The views and conclusions are those of the authors and not necessarily those of the U. S. Government.

The writers gratefully acknowledge the cooperation of Kitt Energy Corporation without which this study could not have been conducted.

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Chapter 75

APPLICATION OF TIME DOMAIN REFLECTOMETRY TO MINING

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ABSTRACT

Examples are presented in which Time Domain Reflectometry (TDR) was employed to locate deformation in rock masses induced by mining. The first example involved monitoring the propagation of overburden fractures above a longwall coal panel and the second example involved the use of TDR to monitor rock mass deformation in the vicinity of a strip mine highwall. Rock mass deformation, strata separation and large block movement generated strains and failure along coaxial cables which were installed in drill holes and the TDR reflections generated by these strains and failures were monitored. Analysis of field data and a preliminary laboratory test indicate that it may be possible not only to locate rock mass movements but also to quantify these movements with TDR data.

INTRODUCTION

Time Domain Reflectometry (TDR) is an electrical pulse testing technique originally developed to locate breaks in power transmission cables. Recently, this technique has been adapted for use in monitoring the movement of rock masses during mining. Coaxial cables have been installed in drill holes or along mine entries such that rock mass movement will damage the cables. When electrical pulses are transmitted along the cables it is possible to locate rock mass movements by virtue of the pulse reflections created by cable breaks.

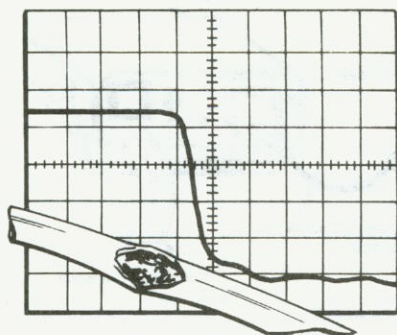
TDR OPERATING PRINCIPLE AND INSTALLATION

A TDR installation consists of a TDR cable tester and the cable to be tested. Ultra-fast rise time voltage pulses are sent down the cable from the tester. Cable defects such as crimps, short circuits or breaks reflect these pulses back to the cable tester. The returned voltage, sometimes called the reflected voltage, is superimposed on the advancing initial step and will appear as a step-up or step-down transition on the cable tester display as shown in Figure 1. Inductive faults or faults of higher resistance than that of the cable (such as a frayed or cut cable) cause reflections in phase with the initial step resulting in a step-up transition. Capacitive faults or faults of lower resistance than that of the cable such as a short circuit or crimping will cause out-of-phase reflections resulting in step-down transitions. Assuming a constant pulse propagation velocity, the distance to the cable fault is proportional to the elapsed time between initiation of the voltage step and arrival of the returned voltage. The cable tester measures this time in units of distance. Consequently, it is possible to determine the location of a cable fault and the nature of the fault by inspection of the TDR signal that is recorded.

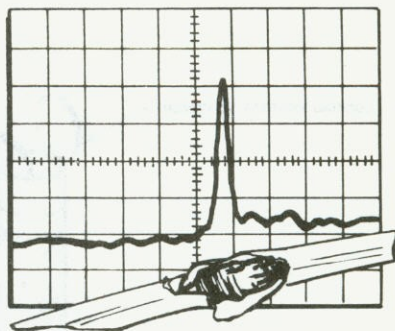
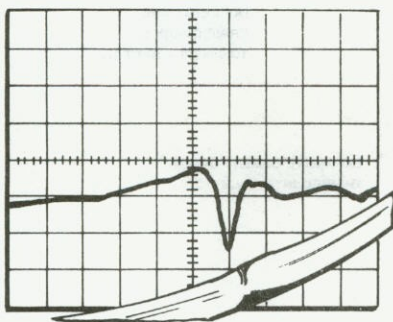
The capability of distinguishing cable breaks from cable crimps provides a convenient means for improving the accuracy with which cable breaks can be located. This accuracy is on the order of 2% of the distance from the tester to a cable break so that a break at a distance of 30 m (100 ft) can only be located to within 0.6 m (2 ft) of its actual position. If a cable is crimped at selected intervals however, a set of reference marks is recorded each time the cable is tested. The location of cable breaks or any other cable defects which result from rock mass movement can then be determined with respect to these reference marks.

Cable crimps not only provide reference marks on TDR records, but also produce weaknesses in the cable that deform as the cable is strained. Deformation of the crimps causes changes in the cable properties with a resultant change in the shape of the TDR signatures associated with the reference marks. As will be shown below, these changes in shape can be correlated with rock mass movement.

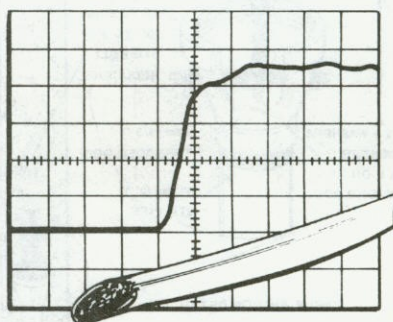
Features of a TDR coaxial cable installation are shown schematically in Figure 2. Prior to installation the cable is crimped at 3 m (10 ft) intervals and a UHF connector is attached to the top-of-hole end. The cable is connected to a TDR cable tester via a special 50 ohm connector cable and a reading is made to ascertain that all crimps will be recorded. The crimps must be deep enough to ensure a strong reflection but not so deep as to cause a short circuit in the cable. Signals attenuate over long cables especially when there are a large number of crimps (Panek and Tesch (1981) found it necessary to put as many as four closely-spaced



a) FRAYED CABLE - NO SHORT

b) FRAYED CABLE - SHORT BETWEEN BRAID
AND CENTER CONDUCTOR

c) CABLE CRIMP



d) SEVERED CABLE

Figure 1. TDR REFLECTIONS GENERATED WITH
DIFFERENT COAXIAL CABLE FAULTS
(AFTER TEKTRONIX, 1979)

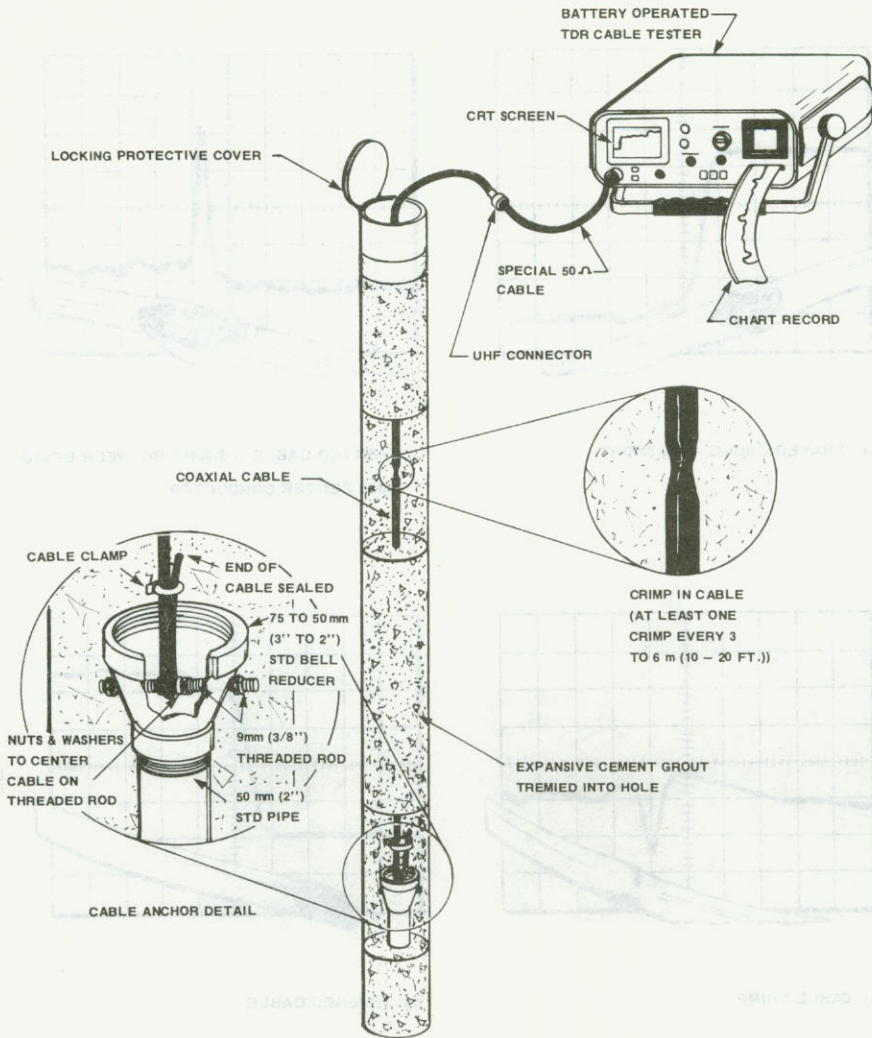


Figure 2. SCHEMATIC TDR INSTALLATION

crimps at the most distant locations on a long cable). After being adequately crimped, the cable is attached to an anchor, lowered downhole, and bonded to the surrounding rock with an expansive cement grout that is tremied into the hole.

A locking protective cover is placed over the hole to minimize the possibility of vandalism. However, even if the top of the cable should become severed, it is still possible to attach a new UHF connector to the cable and resume monitoring since the cable crimps are being used as the measurement reference. In one case, TDR cables were located and successfully interrogated after 20 m (70 ft) of overburden (in which the cables were embedded), was removed.

EXAMPLE APPLICATIONS

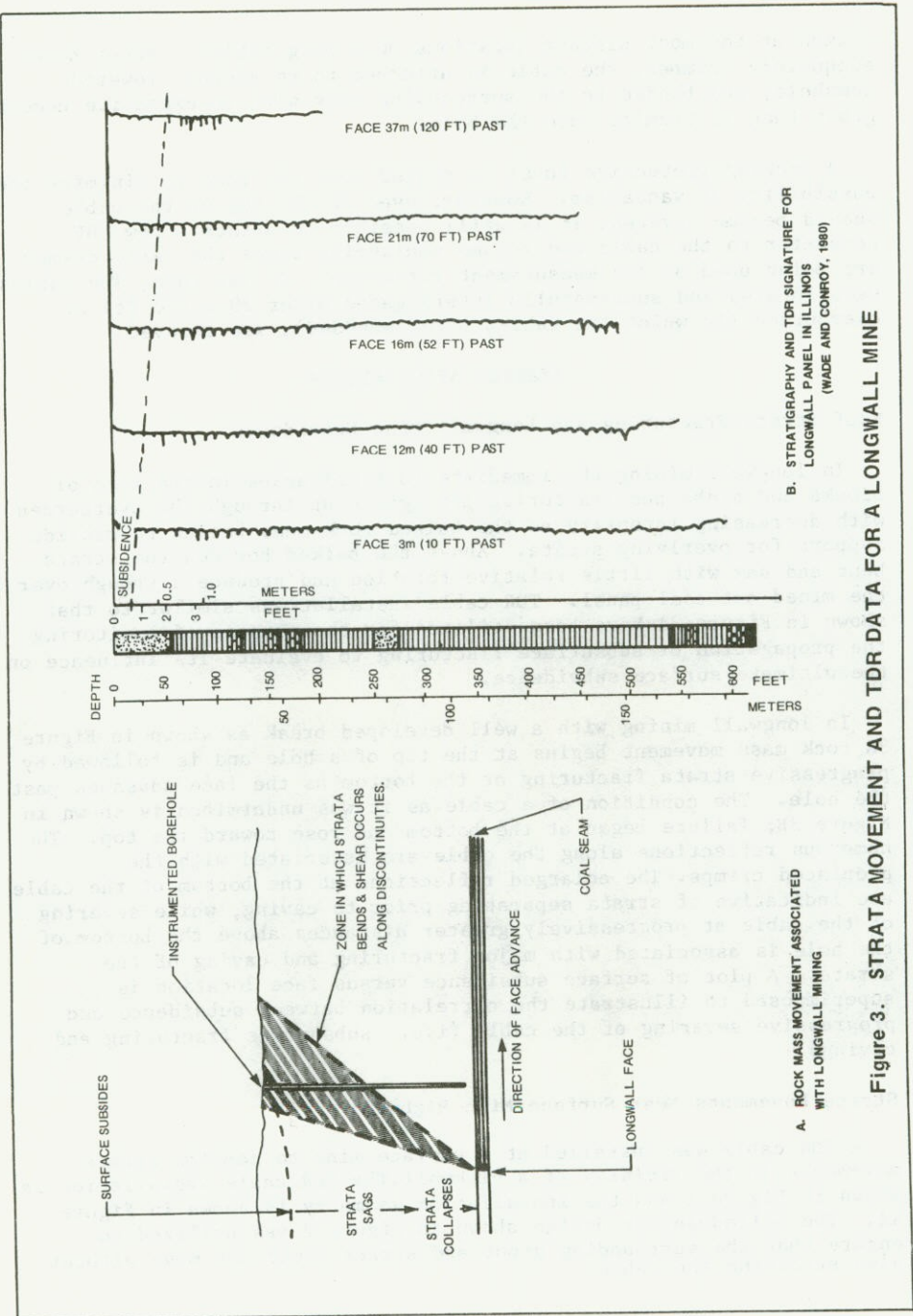
Roof Strata Fracturing and Longwall Mine Subsidence

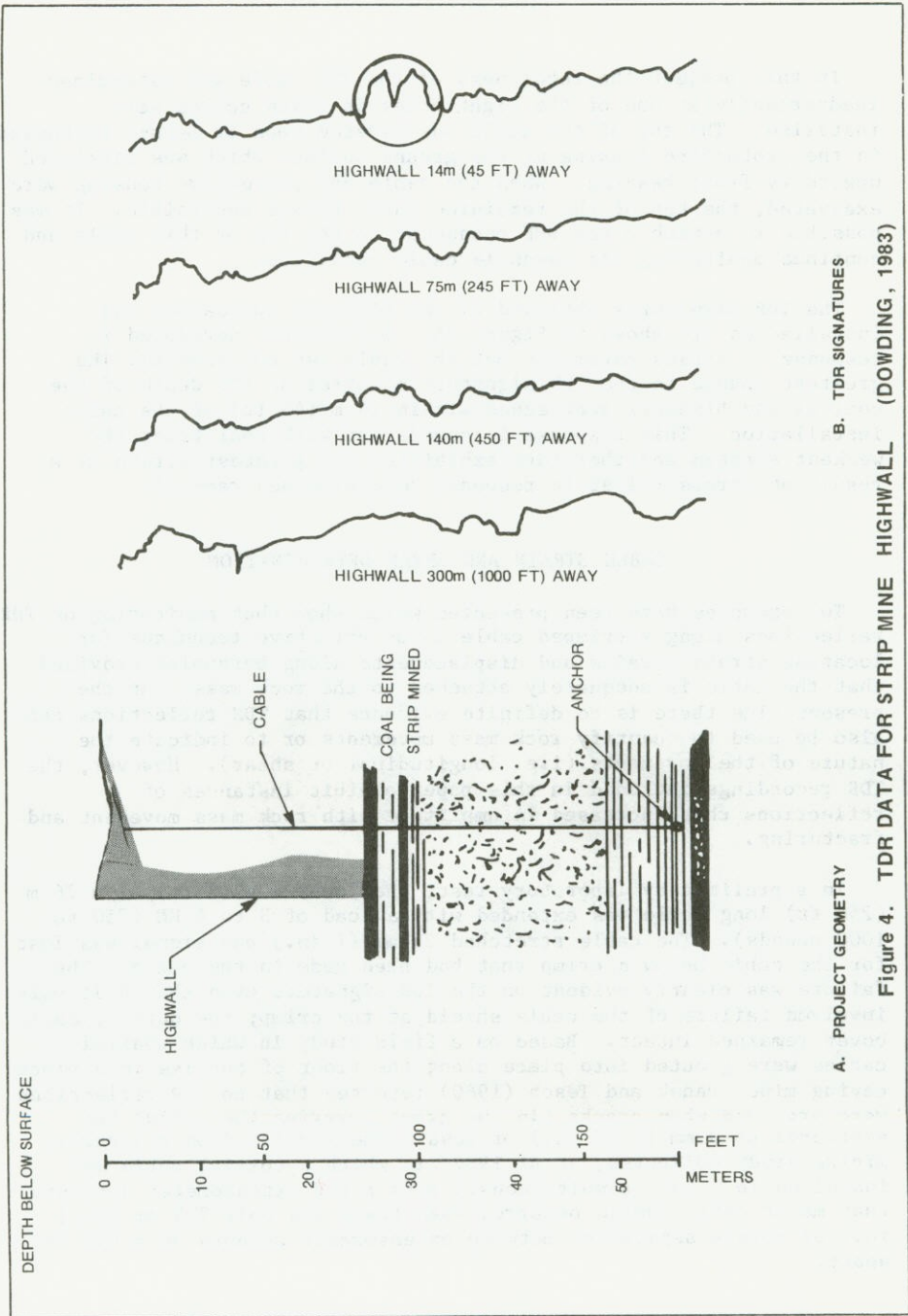
In longwall mining the immediate roof collapses in the form of blocks and slabs and fracturing propagates up through the overburden with decreasing intensity as the bulked rock mass begins to provide support for overlying strata. Above the bulked horizon the strata bend and sag with little relative rotation and produce a trough over the mined out coal panel. TDR cable installations similar to that shown in Figure 3A have been utilized for the purpose of monitoring the propagation of subsurface fracturing to evaluate its influence on the ultimate surface subsidence.

In longwall mining with a well developed break as shown in Figure 3A rock mass movement begins at the top of a hole and is followed by progressive strata fracturing at the bottom as the face advances past the hole. The condition of a cable as it was undermined is shown in Figure 3B; failure began at the bottom and rose toward the top. The numerous reflections along the cable are associated with the preplaced crimps. The enlarged reflections at the bottom of the cable are indicative of strata separating prior to caving, while severing of the cable at progressively greater distances above the bottom of the hole is associated with major fracturing and caving of the strata. A plot of surface subsidence versus face location is superimposed to illustrate the correlation between subsidence and progressive severing of the cable (i.e., subsurface fracturing and caving).

Strata Movements Near Surface Mine Highwall

A TDR cable was installed at a surface mine to monitor strata movements in the vicinity of a highwall. The TDR cable installation is shown in Figure 2 and the installation geometry is shown in Figure 4A. The belled-anchor design shown in Figure 2 was utilized to ensure that the surrounding grout and strata could not move without also straining the cable.





In this project the robustness of the TDR cable was determined inadvertently at one of the eight holes in which cables were installed. The top of the cable was severed when it became entangled in the protective housing at the ground surface which was displaced upward by frost heaving. When the cable and protective housing were excavated, the top of the remaining cable became accessible. It was possible to attach a new UHF connector to the top of this cable and continue monitoring the downhole cable condition.

The TDR signatures obtained as the highwall approached one installation are shown in Figure 4B. Reflections developed in response to strata movements but the cable was not severed. The greatest change in the TDR signature occurred at the depth of the coal as the highwall approached within 14 m (45 ft) of the cable installation. This response is consistent with coal being the weakest stratum and therefore exhibiting the greatest strain as a result of stress relief in response to overburden removal.

CABLE STRAIN AND BREAK DETERMINATION

Two examples have been presented which show that monitoring of TDR reflections along a crimped cable is an effective technique for locating strata strains and displacements along boreholes provided that the cable is adequately attached to the rock mass. At the present time there is no definite evidence that TDR reflections can also be used to quantify rock mass movements or to indicate the nature of the movements (i.e. longitudinal or shear). However, the TDR recordings included in this paper exhibit instances of reflections that increased in amplitude with rock mass movement and fracturing.

In a preliminary laboratory test, the lower 1 m (3 ft) of a 76 m (250 ft) long cable was extended with a load of 3 to 4 kN (750 to 1000 pounds). The cable stretched 25 mm (1 in.) and signal was lost for the cable below a crimp that had been made in the cable. The failure was clearly evident on the TDR signature even though it only involved failure of the cable shield at the crimp; the outer plastic cover remained intact. Based on a field study in which coaxial cables were grouted into place along the floor of entries in a block caving mine, Panek and Tesch (1980) reported that no TDR reflections were produced when cracks (in the grout covering the cable) had apertures of 3 mm (0.12 in.) or less. The results from a longwall mining study (O'Rourke, et al 1982) in which a coaxial cable was installed in a hole simultaneously with a rod extensometer indicate that major cable damage occurred when there was only 2.8 mm (0.11 in.) of strata separation between extensometer anchors 30 m (99 ft) apart.

Definite conclusions about the magnitude of displacement required to produce TDR reflections at cable breaks cannot be made based on available data. However, it may be possible to develop correlations on the basis of laboratory-scale tests.

SUMMARY AND CONCLUSIONS

Rock mass movements associated with longwall and surface mining have been monitored by utilizing the principle of Time Domain Reflectometry (TDR). Monitoring was performed by placing crimped coaxial cables in drill holes and anchoring them to the rock mass with expansive cement grout. Rock mass deformation, strata separation and large block movement generated strains and fractures in the cement grout which, in turn, generated strains and breaks along the coaxial cables. The TDR reflections generated by these cable defects were referenced to reflections generated by crimps deliberately made in the cable prior to installation. These crimps were also points of weakness which changed cable properties (and the resultant TDR signatures) as the cable was strained prior to strata separation.

Provided that the coaxial cable is adequately anchored to the surrounding rock, TDR monitoring can produce a reliable record of rock mass fracture locations. There is not sufficient data available at the present time to determine if TDR monitoring can be used to quantify rock mass movements and/or to differentiate between shear movement and strata separation. The recordings presented in this paper and other references, however, do exhibit many instances of TDR signatures that increased in amplitude in response to rock mass movement.

For holes deeper than 30 m (100 ft), TDR cables are less expensive and easier to install than rod or wire type multiple-point extensometers. Data acquisition is simple and a permanent record can be obtained each time a reading is made. Even if the cable is cut, as long as some portion of the remaining cable is accessible a new connector can be installed and monitoring continued. Since it is common for exploratory drill holes to be backfilled with cement grout, it would be a simple matter to install crimped coaxial cables in these holes for purposes of monitoring rock mass movements generated by mining.

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Chapter 76

PROPOSED CRITERIA FOR SUBSIDENCE DAMAGE TO BUILDINGS

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ABSTRACT

United States federal and state regulatory authorities require underground mine operators to adopt adequate measures to minimize material damage to the surface caused by mine subsidence. This paper presents an approach to define and determine the extent of material damage due to subsidence from underground coal mining. The basic steps involved in the development of adequate criteria have been enumerated together with the approaches and methodologies used.

INTRODUCTION

The Office of Surface Mining of the U.S. Department of the Interior was directed by the Congress to develop regulations to minimize surface damage due to mining subsidence in accordance with the provisions of the Surface Mining Control and Reclamation Act of 1977 (SMCRA). This requires that underground coal mine operators must "adopt measures consistent with known technology in order to prevent subsidence causing material damage to the extent technologically and economically feasible," Implied in the constraints of Section 516(b)(1) of SMCRA is the determination of what constitutes material damage to surface structures and renewable resource lands.

This paper discusses the results of an investigation for developing criteria to determine material damage caused to different categories of buildings, by underground coal mining operations in the United States.

BACKGROUND

Subsidence entails five major components, which affect surface structures and renewable resources, and cause damage to them:

- vertical displacement
- horizontal displacement
- slope of the subsidence trough, that is, the derivative of the vertical displacement with respect to the horizontal
- strain in the horizontal direction, that is, the derivative of the horizontal displacement with respect to the horizontal
- vertical curvature, which may be approximated by the derivative of the slope, or the second derivative of the vertical displacement with respect to the horizontal.

Surface damage is usually brought about by a combination of these basic phenomena. These parameters, therefore, form the basis for assessment of damage to surface structures and renewable resource lands.

Limiting ground deformations refer to such magnitudes of subsidence that would not give rise to permanent damage to surface features with regard to their future use. Implied in this definition, therefore, is the fact that some movement is permissible, and the characteristics of the surface feature, as well as its intended future use, will determine the limiting value. In other words, restrictions would be more stringent for critical surface features such as strategic structures, public buildings, monuments, and industrial establishments, while on farmlands and relatively unused land some leniency may be exercised. The restrictions can be further relaxed where specially constructed subsidence-resistant structures are present.

Structures subjected to subsidence movements may experience bending, angular distortion, torsion, contraction or extension, tilt, and translation, in different conditions, depending on site conditions. Generally these manifest themselves as distinct types of damage (discussed in Bhattacharya and Singh, 1984). Usually the nature and degree of structural deformation will vary along the length of the structure in response to the varying ground surface displacements.

Observations have shown that building deformation from ground movements usually begin at foundation level and propagate upward through the basement to the superstructure. The transmission of the deformations from the foundation to the superstructure depends on the nature of the structure, in particular its continuity and attachment. In many cases, where the superstructure is of more flexible construction than the foundation, the basement is damaged more severely than

the walls or the roof.

METHODOLOGY

Literature from the United States, United Kingdom, Germany, France, Poland, Russia, Sweden, South Africa, India, Japan, and Australia, was analyzed to isolate data that were most pertinent to the requirements of this study. Due to high variability of the data, different data locations, and varying geological and mining conditions, the data did not lend themselves to rigorous statistical treatment. Instead, they were evaluated on the basis of selection criteria derived from engineering judgement.

The development of adequate surface damage criteria involved four interrelated steps:

1. Formulation of Conditions Representing Damage. The definition of conditions representing damage is a requisite starting point for any set of damage criteria, because it defines what constitutes damage for a given surface feature. Parenthetically, this step has its counterpart in engineering design where it is first necessary to establish characteristics of performance. The conditions representing damage may be defined as those leading to impaired use or threat to safety associated with a structure.

2. Selection of Appropriate Engineering Parameters. Most surface structures exhibit varying degrees of susceptibility to different components of ground movement. For example, a tall structure may be more adversely affected by tilting, whereas a long structure will probably be more susceptible to horizontal strain. It is, therefore, prudent to select a set of engineering parameters that could be best used to assess the impact of subsidence on a specified surface feature.

3. Quantification of Allowable Ground Movements. The next step in the development of damage criteria involved stipulating quantitative limits of the engineering parameters corresponding to the major building categories and degree of damage. A definitive limit for each item is usually difficult to assign; stipulating a range of values was considered more appropriate.

4. Correlation of Surface Effects to Underground Workings. Since an important application of the surface damage criteria lies in the design of underground mine layouts in areas occupied by surface structures and renewable resource lands, the final step in the development of the criteria should involve the correlation of subsidence movements to the mine geometry, mining methods, and geology of a given mining area. The primary objective of this step is to improve upon the current subsidence prediction methodologies of the United States. The efficacy of the surface damage criteria will depend to a large

extent on how well surface movements can be related to underground geometry.

PROPOSED CRITERIA FOR DAMAGE TO BUILDINGS

Building Categories

Buildings were subdivided into four distinct groups of building construction commonly found in an urban or rural environment of the United States, as shown in Table 1. Since the primary application of the criteria developed in this study is to aid regulatory authorities and mine operators in estimating potential damage from subsidence over extensive undermined areas, it was considered expedient to make the building classes such that it would be possible to classify buildings over an entire area in an inexpensive and timely manner. Furthermore, details of building foundations and structural intricacies may be difficult to obtain since construction records on every building are rarely available and direct inspection of every building may not be possible. Consequently, these were not included in the proposed classification.

TABLE 1. Proposed Categories of Buildings

<u>DESCRIPTION</u>	<u>CATEGORY</u>
BRICK AND MASONRY STRUCTURES/BRICK BEARING WALLS/LOW RISE STRUCTURES	1
STEEL AND REINFORCED-CONCRETE FRAME STRUCTURES	2
TIMBER FRAME STRUCTURES	3
MASSIVE STRUCTURES OF CONSIDERABLE RIGIDITY/CENTRAL CORE DESIGN	4

Damage Categories

The damage categories along with class definitions are shown in Table 2. The terminology used is frequently referenced in civil engineering applications.

It is important to recognize that although Table 2 associates collapse potential with the structural damage level only, the levels of distortion associated with functional and even architectural damage may represent a hazard and potential life threat in special cases. For example, minor cracking in some ornamental masonry can cause it to detach from doorway and ceiling locations. If the pieces of masonry fall a significant distance, they can do great harm. The external cladding of some buildings may be especially sensitive to

differential movements so that only small distortions may cause a dangerous fall.

It should also be emphasized that the structural damage category, as defined in this report, is not strictly analogous to that commonly accepted in engineering practice. In this study, it refers to a somewhat advanced level of structural damage, whereby primary structural members may be affected leading, perhaps, to instability and collapse. The less severe levels of structural damage are included in the functional damage category.

TABLE 2. Proposed Levels of Damage Severity

<u>TYPE OF DAMAGE</u>	<u>SYMBOL</u>
ONSET OF ARCHITECTURAL DAMAGE CHARACTERIZED BY SMALL SCALE CRACKING OF PLASTER AND STICKING OF DOOR AND WINDOWS	ARCH
ONSET OF FUNCTIONAL DAMAGE CHARACTERIZED BY INSTABILITY OF SOME STRUCTURAL ELEMENTS, JAMMED DOORS AND WINDOWS, BROKEN WINDOW PANES, BUILDING SERVICES RESTRICTED	FUNC
ONSET OF STRUCTURAL DAMAGE CHARACTERIZED BY IMPAIRMENT OF PRIMARY STRUCTURAL MEMBERS, POSSIBILITY OF COLLAPSE OF MEMBERS, COMPLETE OR LARGE-SCALE REBUILDING NECESSARY, MAY BE UNSAFE FOR HABITATION	STRUC

Ground Movement Parameters

The ground movement parameters used in the damage criteria are shown in Table 3. As far as possible, the units have been kept consistent. It was considered advantageous to use several parameters since, depending upon the application, different parameters may take precedence over others. Angular distortion is distinguished from differential settlement by subtracting out the rigid body rotations. The deflection ratio is indicative of the bending deformation of the structure, while angular distortion estimates the relative vertical strains.

TABLE 3. Ground Movement Parameters

<u>PARAMETER</u>	<u>SYMBOL</u>	<u>UNIT</u>
ANGULAR DISTORTION	AD	$\times 10^{-3}$
HORIZONTAL STRAIN	HS	$\times 10^{-3}$
DEFLECTION RATIO	DR	$\times 10^{-3}$
RADIUS OF CURVATURE	RC	mile (km)

Quantitative Limits

A summary of all pertinent data obtained during the study is presented in Table 4. In some cases, the category of structure that particular data referred to could not be determined; these were omitted. Units of measurement were modified to the standard units specified in Table 3.

The final damage criteria for buildings is illustrated in Table 5. For a given building category and damage level, a range of ground movement values and a recommended value is prescribed. The range of values refer to the data spread observed in the literature, and the recommended values are ones that were judged to be the most representative. Due to the lack of data, a number of blanks were left in the table (marked NA). It is hoped that future data collection efforts will enable the entire chart to be completed.

The column showing the number of data sources was included to indicate the degree of confidence associated with a recommended value. In some instances, where the data spread is large, a range of recommended values has been stipulated. The user is advised to select the appropriate value on the basis of judgement and the level of conservatism desired.

TABLE 4. Survey of Tolerable Ground Movements in Buildings

DATA SOURCE	DATA LOCATION	BUILDING CATEGORY	DAMAGE CATEGORY	TYPE OF MOVEMENT	ALLOWABLE MAGNITUDE	REMARKS
NIEMCZYK (1949)	GERMANY				1.0-2.0	
MEYERHOFF (1953)					0.5-1.0	
SKEMPTON & McDONALD (1956)					1.0-2.0	RECOMMENDED FOR DESIGN
POLSHIN & TOKAR (1957)	USSR				1.0	
BOWERS (1962)	US	I	ARCH	AD	1.0-2.0	
O'ROURKE (1976)	US				1.0	
ATTEWELL (1977)	UK				1.0	
BOSCARDIN (1980)	US				1.2	
NIEMCZYK (1949)	GERMANY				0.6	
BEEVERS & WARDELL (1954)	UK				0.4	
POLSHIN & TOKAR (1957)	USSR				0.5	
PRIEST & ORCHARD (1958)	UK				0.8	
GOTO (1968)	JAPAN	I	ARCH	HS	0.5	
SINGH & GUPTA (1968)	INDIA				0.4-0.5	
LITTLEJOHN (1975)	UK				0.25	VERY MINOR CRACKING
NCB (1975)	UK				0.5-1.0	30-60m LONG STRUCTURE
O'ROURKE (1976)	US				<0.75	
ATTEWELL (1977)	UK				0.5-1.0	
CORDING ET AL (1978)	US				1.0-1.5	
YOKEL (1978)	US				0.5	
BOSCARDIN (1980)	US				0.5	
POLSHIN & TOKAR (1958)	USSR				0.3-0.7	
GRANT (1974)	US	I	ARCH	DR	1.0	
BURLAND & WROTH (1975)	UK				0.4	
MEYERHOFF (TERZAQHI) (1953)	US				3.5	
SKEMPTON & McDONALD (1956)	US				3.3	
VNIMI (1958)	USSR				4.0-6.0	
BJERRUM (1963)	US				2.0	
GRANT (1974)	US				3.3	
STARZEWSKI (1974)	POLAND	I	FUNC	AD	3.3-5.0	
ULRICH (1974)					3.0	
BROMS ET AL (1976)	SWEDEN				2.0-3.3	
THORBURN & REID (1977)	UK				2.7	
ADAMEK ET AL (1982)	POLAND				2.5	
NISHIDA ET AL (1982)	JAPAN				3.0-6.0	

TABLE 4. Survey of Tolerable Ground
Movements in Buildings (continued)

DATA SOURCE	DATA LOCATION	BUILDING CATEGORY	DAMAGE CATEGORY	TYPE OF MOVEMENT	ALLOWABLE MAGNITUDE	REMARKS
VNIMI (1958)	USSR	1	FUNC	HS	2.0-4.0	
ULRICH (1974)	US				1.0	
CORDING ET AL (1978)	US				2.5-3.5	
ADAMEK ET AL (1982)	POLAND				1.5	
RIGBY & DEKOMA (1952)	US	1	FUNC	DR	0.14-0.22	
WOOD (1952)					0.25	
HORNE & LAMBE (1964)					0.6	
VNIMI (1958)	USSR	1	FUNC	RC	1.9-12.4 mile (3-20km)	
ULRICH (1974)	POLAND				12.4 mile (20km)	
ADAMEK (1982)					12.4 mile (20km)	
NISHIDA ET AL (1982)	JAPAN				8.0 mile (13km)	
OROURKE ET AL (1977)	US	1	STRUC	AD	7.0-8.0	30m LONG STRUCTURE
NCB (1975)	UK	1	STRUC	HS	3.5	
BOSCARDIN (1980)	US				2.75	
SKEMPTON & McDONALD (1956)	US	2	ARCH	AD	1.0-2.0	
POLSHIN & TOKAR (1958)	USSR				2.0	
SOWERS (1962)	US				2.0-2.5	
BRETH & CHAMBOSSSE (1975)	US				2.2	
OROURKE (1976)					1.3	
ATTEWELL (1977)					2.0	
THOMAS (1953)	US	2	FUNC	AD	2.5-3.3	
SKEMPTON & McDONALD (1956)					3.3-6.6	
STARZEWSKI (1974)					POLAND	
MAHAR & MARINO (1981)	US	3	ARCH	AD	2.0	DEPENDENT ON FOUNDATION
GOTO (1968)	JAPAN	3	ARCH	HS	1.0	
STARZEWSKI (1974)	POLAND	3	FUNC	AD	5.0-10.0	
BROMS & FREDRIKSSON (1976)	SWEDEN				3.3-5.0	

APPLICABILITY AND LIMITATIONS

The proposed criteria are based on data obtained from the literature. Consequently the relative reliability of the various sources of data strongly influences the validity of the criteria.

It is generally known that only a fraction of the actual ground strain or distortion is transferred to the structure. The extent of this transfer is a function of several complex variables including the soil and structure characteristics. The criteria developed in this study do not take this soil-structure interaction into account. Also the characteristics of individual structural components are not addressed explicitly in the criteria. For these reasons, the applicability of the criteria is likely to vary from one structure to another.

Since the categorization of the buildings is based on superstructure characteristics alone, some discrepancies may be noted in those instances where the damage originates from a part of the structure besides the superstructure.

No distinction has been made between compressive and tensile ground strain limits in the recommendations made in this study. Generally, tensile strains are more harmful to surface structures and, therefore, most of the horizontal strain data pertain to deformations of tensile origin.

TABLE 5. Damage Criteria for Buildings

BUILDING CATEGORY	LEVEL OF DAMAGE	GROUND MOVEMENT LIMITS											
		ANGULAR DISTORTION ($\times 10^{-3}$)			HORIZONTAL STRAIN ($\times 10^{-3}$)			DEFLECTION RATIO ($\times 10^{-3}$)			RADIUS OF CURVATURE MILE (in)		
		RANGE OF VALUES	RECOMMENDED VALUES	NO. OF SOURCES	RANGE OF VALUES	RECOMMENDED VALUES	NO. OF SOURCES	RANGE OF VALUES	RECOMMENDED VALUES	NO. OF SOURCES	RANGE OF VALUES	RECOMMENDED VALUES	NO. OF SOURCES
1	ARCH	0.5-2.0	1.0	8	0.25-1.5	0.5	13	0.3-1.0	0.3	3	—	—	—
	FUNC	2.0-6.0	2.5-3.0	12	1.0-4.0	1.5-2.0	5	0.14-0.6	0.5	3	2-12 (3-20)	12 (20)	5
	STRUC	7.0-8.0	7.0	1	2.75-3.5	3.0	2	—	—	—	—	—	—
2	ARCH	1.0-2.5	1.3	6	—	—	—	—	—	—	—	—	—
	FUNC	2.5-6.6	3.3	3	—	—	—	—	—	—	—	—	—
	STRUC	—	—	—	—	—	—	—	—	—	—	—	—
3	ARCH	2.0	1.5	1	1.0	1.0	1	—	—	—	—	—	—
	FUNC	3.3-10.0	3.3-5.0	2	—	—	—	—	—	—	—	—	—
	STRUC	—	—	—	—	—	—	—	—	—	—	—	—
4	ARCH	—	—	—	—	—	—	—	—	—	—	—	—
	FUNC	—	—	—	—	—	—	—	—	—	—	—	—
	STRUC	—	—	—	—	—	—	—	—	—	—	—	—

— INDICATES DATA NOT AVAILABLE

In conclusion, it should be emphasized that the quantitative limits provided in the criteria are approximate, and are intended to serve only as rough guides for ascertaining the extent of damage to surface features. Depending on specific site conditions, it is quite possible that the actual damage may differ significantly from that predicted on the basis of the proposed criteria. However, due to the degree of conservatism built into these criteria, it is anticipated that if the magnitude of ground movements are maintained below the prescribed limits, the extent of damage to the surface feature should be less than the level of damage specified.

ACKNOWLEDGEMENTS

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The American Medical Association is a non-profit corporation organized for the purpose of promoting the interests of the medical profession and the public. It is composed of members who are physicians and surgeons, and who are engaged in the practice of medicine and surgery. The Association is organized into various departments and committees, and it holds regular meetings and conventions. It is also engaged in various other activities, such as the publication of journals and the holding of exhibitions.

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Chapter 92

STABILITY OF SLOPES IN DISCONTINUOUSLY JOINTED ROCK

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INTRODUCTION

Attempts to analyze the stability of slopes, foundations and underground openings in discontinuously jointed rock have generally assumed full joint continuity or ignored the role of stress concentration at the tips of discontinuous joints. The most common approach in rock slopes is to consider a failure plane consisting of coplanar, but discontinuous joints separated by intact rock. The intact rock is assumed to fail in shear as a Mohr-Coulomb material and to contribute a strength component proportional to the fraction of the failure plane that is intact rock (Jennings, 1970).

An alternative approach by Stimpson (1978) applies Lajtai's (1969) theory for shear strength of discontinuously jointed rock masses to slopes. Lajtai allows tensile failure in the intact rock of a composite surface. This theory yields lower shear strength than that of Jennings (1970) except in the presence of high normal compression, however it also fails to account for the stress concentration at the tip of a discontinuous joint.

For a given slope geometry, stresses increase with slope height, but in both of these models rock mass strength is independent of size. A fracture mechanics analysis shows that particularly for small slopes, rock mass strength decreases with size. A continuous stepped failure surface produced by propagation of preexisting joints may develop at lower shear stress than predicted by the Jennings (1970) or Lajtai (1969) models. This fracture mechanics model applied to slopes in competent rock is more realistic in its assumed failure mechanism, and for large slopes more conservative than the Jennings (1970) or Lajtai models (1969).

To understand the behavior of slopes, both tensile and shear strength must be considered, because both may resist movement in discontinuously jointed rock. Tensile strength criteria govern the zone of tensile stress and separation at the head of the slope, while shear strength provides resistance over the balance of the slope.

TENSILE STRENGTH

Rock masses are typically assumed to have no tensile strength, but significant strength is possible where joints are discontinuous. For a single isolated crack the uniaxial tensile stress applied to the rock mass for incipient crack propagation is (Rooke and Cartwright, 1976):

$$\sigma_t = K_{IC} (\pi a)^{-1/2}$$

where K_{IC} is critical stress intensity and a is half the crack length. If the conservative assumption is made that tensile failure results immediately from crack propagation, σ_t defines the tensile strength of the material. For strong rock K_{IC} varies over a relatively small range and a value of $1.0 \text{ MNm}^{-3/2}$ is assumed for purposes of calculation. For long-term loading this value may be reduced to account for stress corrosion (Anderson and Grew, 1977; Atkinson, 1982). For widely spaced joints 10 cm long, the tensile strength of the rock mass is 2.5 MPa (370 psi), a substantial fraction of the strength of strong intact rock. Even for joints 10 m long the strength is 0.25 MPa (37 psi).

Based on Westergaard (1939) and Rooke and Cartwright (1976), the strength σ_t for a rock mass with tension applied perpendicular to an array of equally spaced, identical coplanar cracks is:

$$\sigma_t = K_{IC} (2b \tan[\pi a/2b])^{-1/2}$$

where crack length is $2a$ and the center-to-center distance between cracks is $2b$. For joint continuity (a/b) of 50%, the strength is only 12% lower than for single isolated joints. For 90% continuity the strength is slightly less than half. Even this extreme condition gives a strength of 0.38 MPa (55 psi) for joints 1 m long and 0.12 MPa (17 psi) for joints 10 m long.

Failure may also occur by downward propagation of a pre-existing joint intersecting the free surface of the slope (Brown and Srawley, 1966). If the joint is perpendicular to the free face, rock mass tensile strength is 89% of that for a single isolated joint that is twice as long. Predicted strengths will often be sufficient to prevent crack propagation at the crest of slopes as high as several tens of meters. This may contribute significantly to the stability of small slopes, but in larger slopes shear strength is critical.

SHEAR STRENGTH

Strength of a joint against propagation by shear stress is also a function of the normal stress acting across the joint. For slopes the normal stress perpendicular to a possible sliding joint is compressive. This reduces tensile stress at the crack tip and thereby resists crack propagation. The normal stress may also close the crack. Part of the normal stress would then be transmitted across the crack faces and the compressive (negative) mode I stress intensity K_I would be reduced. Closure also allows mobilization of shear stress on the crack face which reduces the mode II stress intensity K_{II} (McClintock and Walsh, 1962). For open cracks analysis is much simpler and leads to unambiguous results. It is therefore important to establish whether open joints are likely to exist in a discontinuously jointed rock mass. For a penny-shaped crack, maximum closure w for compression σ normal to the crack is given by Sneddon and Lowengrub (1969):

$$w = \sigma a(4/\pi)(1-\nu)/G$$

where a is crack radius, ν is Poisson's ratio and G is shear modulus. Closure stress σ_c is stated in terms of aspect ratio A (maximum aperture/length):

$$\sigma_c = A(\pi/4)(G/(1-\nu))$$

Closure stresses are evaluated in Table 1 for $G = 3 \times 10^4$ MPa and $\nu = 0.2$ for several aspect ratios.

TABLE 1. Joint closure stress as a function of aspect ratio.

Aspect Ratio	Closure Stress (MPa)	Closure Stress (psi)
10^{-5}	0.3	43
10^{-4}	2.9	427
10^{-3}	29.5	4 271
10^{-2}	294.5	42 717

Aspect ratios and joint lengths were measured in the field and laboratory for igneous rocks, limestones, siltstones and sandstones of Paleozoic and Precambrian age. Since the geometry of confined cracks is of interest, the measured outcrop length was taken as the crack length when both ends were visible. The measured length was doubled when only one end was visible. The 19 joints ranged in length from 0.08 to 3 m, with aspect ratios of 2×10^{-4} to 8×10^{-3} . Similar values are reported for microcracks. These data suggest that many joints on all scales will be open in slopes. Even for a joint with aspect ratio 10^{-4} , approximately 140 m of overburden would be necessary to affect closure.

Shear failure in discontinuously jointed rock requires mixed-

mode crack propagation, so K_{II} must be evaluated in addition to K_I . For equally spaced, open coplanar cracks, K_{II} is (Rooke and Cartwright, 1976):

$$K_{II} = \tau (2b \tan[\pi a/2b])^{1/2}$$

where τ is shear stress parallel to the crack plane, $2a$ is crack length and the center-to-center distance between cracks is $2b$. The condition for crack propagation with combined compressive and shear stresses is evaluated by a mixed-mode criterion based on maximum tensile stress at the crack tip (Erdogan and Sih, 1963; Ingraffea, 1981):

$$K_{IC} = \cos(\theta/2) (K_I \cos^2(\theta/2) - 1.5 K_{II} \sin\theta) \quad (1)$$

where θ is the angle the new crack extension makes with the plane of the original crack. This angle satisfies the following equation (Erdogan and Sih, 1963) which is solved numerically for θ :

$$K_I \sin\theta + K_{II}(3\cos\theta - 1) = 0$$

With these relationships and the equations for K_I and K_{II} , the conditions for failure are stated in terms of normal stress σ and shear stress τ . The results are plotted in Fig. 1 for joints 5 and 20 m long and several values of joint continuity J . The failure envelopes become asymptotic to a 45° line with increasing normal stress. This effective friction angle of 45° , which is independent of K_{IC} , provides a conservative estimate of shear strength for a discontinuously jointed rock mass. Along the $\tau=\sigma$ line that defines the asymptotic 45° effective friction angle, θ approaches 90° , i.e. the initial joint extension is nearly perpendicular to the original joint.

It is assumed above that crack surfaces remain free of traction. This assumption is reasonable at least when the length of crack extension is small. For mated but separated surfaces, asperities on opposite sides of the crack will impinge at some value of shear displacement unless separation is greater than asperity height. For long joints and moderate aspect ratio, aperture may be large enough to disengage grain-scale asperities. This would only allow impingement for longer wavelength asperities with smaller i angle (Patton, 1966). For small i angle, asperity impingement requires shear displacement much larger than joint aperture. It will be shown that even for short joints impingement will not generally occur without substantial joint propagation.

Joint shear displacement accompanying incipient propagation is calculated for the condition of zero normal stress ($\sigma=0$). For this condition $K_I=0$, and if τ_c is the shear stress for propagation, $K_{II} = \tau_c(\pi a)^{1/2}$. For pure mode II failure $\theta = 70.5^\circ$

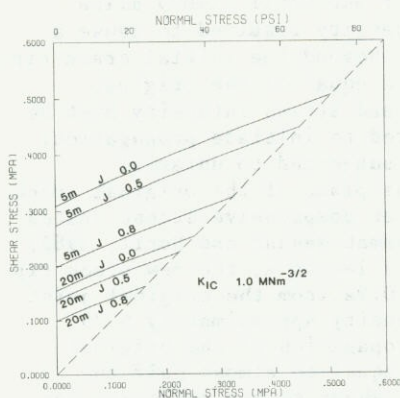


Fig. 1. Shear strength for joints 5 and 20 m long with joint continuity J of 0.0, 0.5 and 0.8.

$$K_{IC} = 1.0 \text{ MNm}^{-3/2}$$

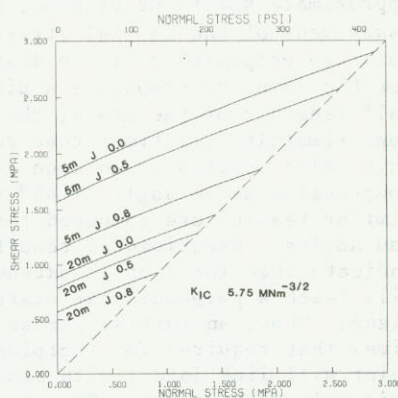


Fig. 2. Shear strength for joints 5 and 20 m long with joint continuity J of 0.0, 0.5 and 0.8.

$$K_{IC} = 5.75 \text{ MNm}^{-3/2}$$

(Erdogan and Sih, 1963). On substitution of these values the mixed mode crack propagation criterion (eq. 1) is solved for τ_c :

$$\tau_c = 0.5a^{-1/2}K_{IC} \quad (2)$$

maximum shear displacement u (half the maximum shear offset) occurs at the center of the crack and is derived for plane strain by Jaeger and Cook (1971):

$$u = \tau a(1-\nu)/G \quad (3)$$

where ν is Poisson's ratio. Substituting eq. 2 into eq. 3:

$$u = 0.5a^{1/2}K_{IC}(1-\nu)/G$$

If u is small relative to maximum joint aperture no asperity impingement occurs and joint surfaces remain traction free. Even for a crack with aspect ratio 10^{-4} , maximum crack aperture is more than an order of magnitude larger than the maximum displacement u . For this large difference the i angle would have to be greater than 85° for asperity impingement and shear traction on joint surfaces.

Joint propagation is assumed to link previously discontinuous joints into a stepped failure surface. This requires some finite joint propagation, and because crack geometry changes with propagation, stress intensity at the crack tip generally decreases unless there is a tensile principal stress. As a result, the joint may become stable before it reaches an adjacent joint.

Approximate solutions by Nemat-Nasser and Horii (1982) allow assessment of the initial stress intensity required to cause a joint to propagate a finite distance beyond the initial crack tip. For the joint to propagate a distance equal to the original half-length a of the crack, the imposed stress intensity must be approximately 5.75 times that required to initiate propagation. This value applies to an open crack subjected to uniaxial compression at an angle of 45° to the plane of the original crack. Similar results are computed for other compressive stress states and angles. Experimental results (Nemat-Nassar and Horii, 1982) indicate that for a crack extended by length a , the new crack tip will reach a perpendicular distance $0.9a$ from the original joint plane. Thus, an initial stress intensity approximately 5.75 times that required for incipient propagation of the original joint will link joints with spacing equal to almost half the original crack length. Fig. 2 shows shear stress necessary for this extent of propagation (calculated by setting $K_{IC} = 5.75 \text{ MNm}^{-3/2}$). Strength is greater for low normal stress in this case than for incipient propagation, but for high stress the 45° effective friction angle again governs. With substantial joint propagation, shear displacement along the original joint may become significant and asperity impingement may occur. In the present paper this factor will be ignored.

APPLICATION TO ROCK SLOPES

The mixed-mode failure criterion for discontinuously jointed rock is applied to slopes with the geometry shown in Fig. 3. Normal and shear stresses applied to joints are approximated by average values on the ultimate failure plane computed by two-dimensional wedge analysis. The dashed line defines joint dip δ on the composite failure surface and the weight of the potential sliding block is that of the rock mass above the dashed line (unit weight 27 KNm^{-3}). The use of average stresses, and stress intensity equations for simple geometries ignores stress concentrations in the slope and the interaction of joints with the free surface. For this reason the results must be considered approximate.

Fig. 4 shows maximum slope heights H for slope angle β of 90° and single isolated joints. Incipient propagation ($K_{IC} = 1.0 \text{ MNm}^{-3/2}$) and propagation equal to half the initial crack length ($K_{IC} = 5.75 \text{ MNm}^{-3/2}$) are the two failure criteria illustrated. The plotted curves flatten with increasing joint length, and for the incipient propagation failure criterion the curves are terminated at the maximum joint length possible for the given slope heights and discontinuity dips. For this criterion failure will not occur in slopes less than 20 m high unless joint continuity is very great. When substantial propagation is required ($K_{IC} = 5.75 \text{ MNm}^{-3/2}$), slopes less than 75 m high are generally stable. Fig. 4 indicates that instability requires joints falling within a relatively narrow dip range, and for high

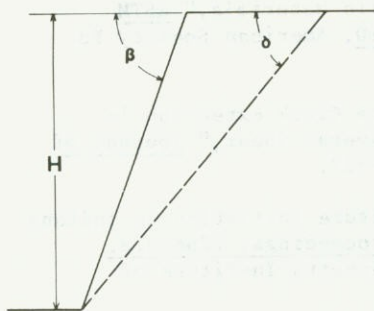


Fig. 3. Slope geometry.

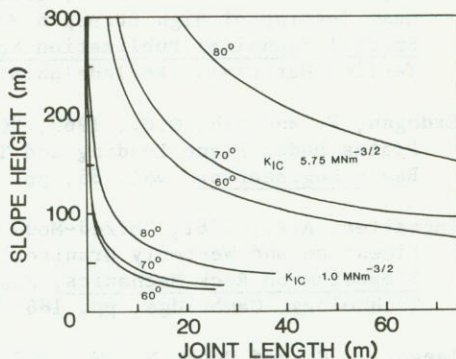


Fig. 4. Maximum slope height for isolated joints dipping 60, 70 and 80 degrees. Upper three curves for $K_{IC} = 5.75 \text{ MNm}^{-3/2}$, lower three for $K_{IC} = 1.0 \text{ MNm}^{-3/2}$.

slopes stability is strongly a function of joint length. Strongly jointed slopes of less than 50 to 75 m height are often stable, but vertical slopes several hundred meters high will be stable only if joints are widely spaced and of limited extent. Otherwise, joint propagation is expected and stepped failure surfaces will be established. For a 70° slope face, stable height is nearly doubled for a given joint length, and the range of hazardous dip angles is further restricted.

Fracture mechanics is useful in understanding and quantifying the relationship between slope height and stability. Application of these methods may also allow assessment of the importance of joint continuity in particular slopes. For some high, steep slopes and proposed slopes the existence or probable formation of a stepped failure surface may be predicted from maximum observed joint extents and average spacing. A more objective assessment may also be made of the stability of small slopes, for which rigid block analyses may be overly conservative.

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INTRODUCTION

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CONSOL (Consolidation Coal Company) is the owner of the mineral rights to the Pittsburgh Coal Seam under the subject structure, and had projected longwall mining under the bridge. They subsequently approached the Department with a proposal to mine beneath the bridge and determine the measures necessary to insure continuity of service and safety of the travelling public.

The average height of mining in the coal seam was 2.47 m. The average depth from the surface to the bottom of the coal seam was approximately 140 m. The bridge structure was centrally located near the end of the longwall panel. The bridge was oriented on a skew of approximately 16° to the direction of mining. See Fig 2 for panel dimensions.

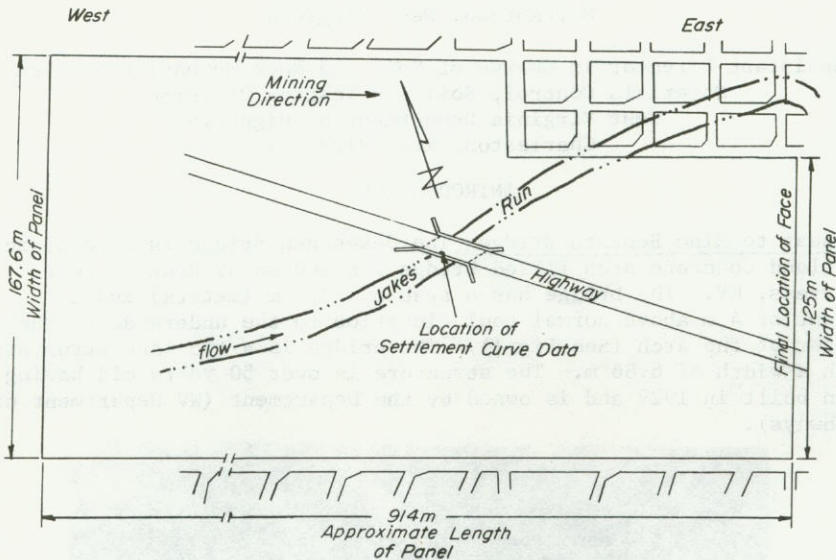


Fig. 2 Plan view of Eastern End of Mining Panel

Mining Method The long wall method was the system employed to mine beneath the structure and leaves no support for overlying strata. Because of this, rapid subsidence is normal immediately after mining. Longwall mining is highly desirable because of the high percentage of recovery, high productivity and conservation of resources.

EVALUATION OF THE SITE AND PROPOSAL

Agreement Conditions CONSOL performed an in-house engineering evaluation of the bridge. It was determined that the bridge could subside to the projected depths without critically damaging bridge or disrupting traffic. In May of 1982, CONSOL approached the Department

concerning mining beneath the bridge. In June of 1982, the Department and CONSOL concluded an agreement which allowed mining under the structure.

The agreement provided measures necessary to insure the safety and continuity of public travel on the bridge. The following are some of the significant conditions of the agreement: 1 - CONSOL was to install a support system comprised of beams and cribbing under the bridge that would subside with the structure and prevent any catastrophic failure. The contact points of the support were situated a few millimeters under the bridge so as not to impede the settlement of the structure. Approval by the Department was required for all protective measures. 2 - CONSOL was responsible for obtaining temporary detour easements and/or right of ways, and to provide a temporary creek crossing in the event the Department determined the bridge unsafe. The determination of the safety was to be the sole province of the Department. 3 - CONSOL was responsible for repair to the structure after subsidence was complete. 4 - CONSOL was to alleviate damage to the bridge by mining continuously so that the settlement process would be uninterrupted. CONSOL agreed to mine continuously within a distance of 21.34 m in advance of the nearest points of the structure to 33.5 m beyond the furthest point of the bridge. This measure would reduce the degree of differential settlement. 5 - CONSOL would be responsible for monitoring the structure by securing an independent engineering firm with extensive bridge experience that was acceptable to the Department. The engineering firm was to monitor the structure prior to, during and after the mining and determine the extent of the damage. CONSOL and the Department also co-monitored the structure simultaneously.

Condition of Bridge Prior to Mining The Department had conducted a routine bridge inspection on August 17, 1984. The report found the bridge in poor condition with the most serious deficiencies being: 1 - The outside edge of the upstream and downstream sides of the arch were spalled with rebars exposed (See Fig 1). 2 - The outside edge of the deck on both upstream and downstream sides at the curbline had moderate to heavy spalling. 3 - The approach flex beam at abutment no. 2 (east) on the downstream side had pulled loose. 4 - The parapet wall was spalled and cracked on both the upstream and downstream side of abutment no. 2.

There was no posted load limit on the bridge. The Department's District Bridge Engineer had assigned a rating for an H-Type vehicle loading of 13608 kg (kilogram) and an operating rating for a 3S2-Type vehicle loading of 40823 kg. The average daily traffic for this 2 lane roadway is 1300.

CONSOL employed John C. Giese and William J. Rota of John C. Giese Engineering, Inc. of Clarksburg, WV to evaluate the bridge prior to mining on June 28, 1982. Their report (3) recorded areas of major

spalling and surface cracks in the arch and spandrel walls. The report stated that little change from the Department's August 17, 1981 report had occurred.

Geology of the Site The bedrock from the coal seam to the bridge is of Pennsylvanian System and includes all of the Monongahela Group and part of the Dunkard Group. The lowest member of the Monongahela Group is the Pittsburgh Coal and at this location, this group is approximately 106m thick with the upper most member being the Waynesburg Coal. There is approximately 32m of the Dunkard Group above the Monongahela Group to the flow line of the stream. A boring had been performed for the CONSOL for the mine site. This boring was used to determine the lithologies and their total thicknesses. The following thicknesses do not include the 2.47m of Pittsburgh Coal that was mined or the soil overburden at the surface beneath the flow line of the stream: Coal 4.8m including stringers and seams - shale 50.6m - limestone 75.6m - slate 3.4m. The regional dip for this area is 0 to 0.5° to the northwest. This area is at the center of a syncline.

Anticipated Affects of Mining on the Bridge Due to the age and condition of the bridge CONSOL and the Department were unsure of the extent and nature of the subsidence damage that might result from the mining operation. The amount of subsidence can be predicted if the subsidence factor can be determined. The subsidence factor simply stated is the ratio of the maximum possible subsidence to the mining height with respect to the width and length of the panel and is a function of the lithology of the geologic strata overlying the mining (1). The predicted subsidence at the bridge was 1.219m (2). See Table 1.

Maximum Predicted Subsidence Meters	Average Mining Height Meters	Depth at Bridge Meters	Gob Width Meters	Subsidence Factor
1.219	2.469	140.21	167.64	0.65

Table 1 Predicted Subsidence at Bridge

The subsidence factor was determined empirically from case studies of this particular area (5). This subsidence factor is representative of the weak strata typical of northern West Virginia.

AFFECT OF MINING ON BRIDGE

Mining Sequence Beneath Bridge The mining beneath the bridge occurred from July 18, 1982 to July 22, 1982, a period of 4 days. The mining was stopped from July 23 to July 25, 12.2m beyond the bridge to reduce the panel width from 167.6m to 125m. Except for the 1.5 day shut down, mining was continuous around the clock and on week-

ends from June 29 through July 30 to minimize the differential settlement effects. Discontinuous extraction can cause a nonuniform settlement rate. The maximum subsidence velocity at the bridge was 0.20m per day.

The settlement (See Fig 3) for 450 days after mining was 1.259m or 3.3 percent greater than the predicted value. The settlement curve is for settlement of the south (upstream) side of the bridge in the center of arch. The settlement data begins on June 29, 1982, 19 days before the face reached the bridge. There may have been some heaving prior to the face reaching the bridge but no survey readings were taken between June 29 and July 19 to record it.

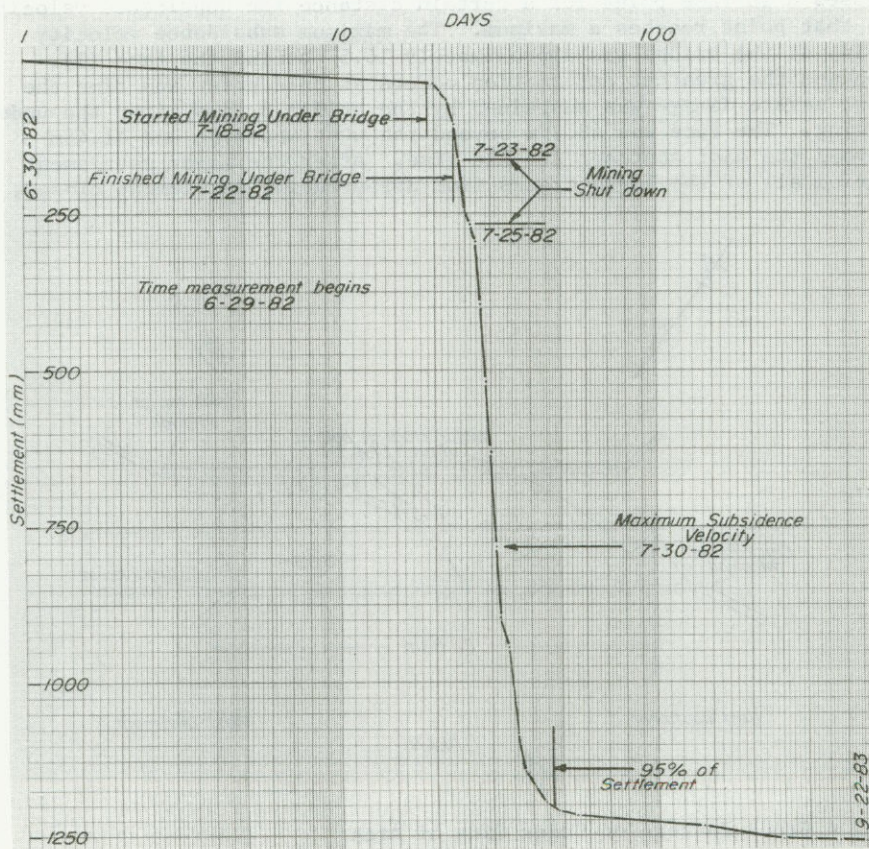


Fig. 3 Settlement of Center of Arch Upstream Side June 30, 1982 through September 22, 1983

Affect on Bridge The bridge experienced differential settlement at varying rates. The arch deformed first dipping at abutment no. 1 (west abutment) which was first to be reached by the mining. This part of the bridge settled faster than the remainder of the bridge. This may have been due impart to the 1.5 day shut down 12.2m beyond the bridge to reduce the face width. Three distinct crack patterns developed (See Fig 4). They were at each quarter point of the under-side of the arch and extended up the sides of the arch. These cracks opened to approximately 3.18mm (millimeter). The average rate of face advance was 6.10m per day. The maximum subsidence velocities varied over the 4 days, contributing to the differential settlement. Since the bridge was skewed to the direction of mining, the settlement also caused a twisting of the bridge during settlement. When the face passes a surface point for a certain distance the subsidence velocity at that point reaches a maximum. The maximum subsidence velocity point at the bridge was approximately 16.5° behind the face (1). Because the greatest deformation occurs at this point and time the most severe damage was sustained by the bridge 8 days after the under-mining. The face was 48.77m beyond the west abutment and 41.45m beyond the east abutment at this time. After the major settlement period all of the large cracks from differential settlement closed,

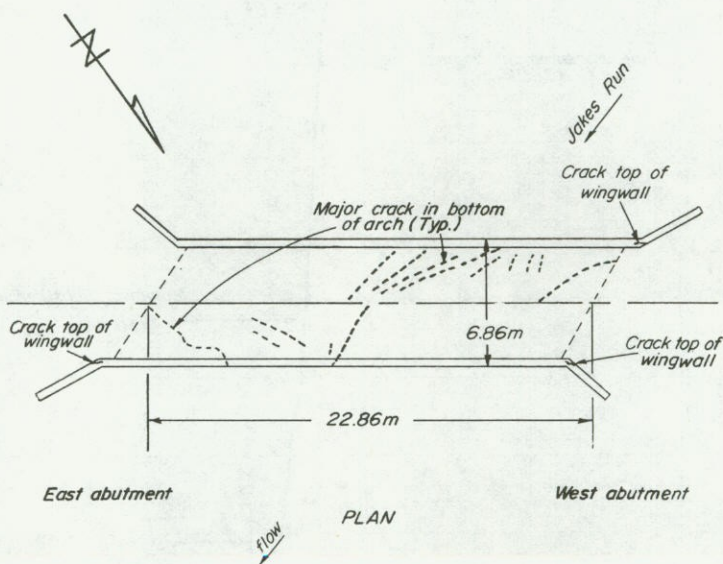


Fig. 4 Crack Pattern on Under Side of Arch

and were almost indiscernible. From the subsidence data evaluated by Geng and Peng (2), 95% of the subsidence occurred in a 37 day period. After a years monitoring, small adjustments in the bridge

were still being noted, but by 15 months after mining, these adjustments were negligible (3mm for a 3 month period for 2 points in the center of the arch). All other points located at the corner of the arch recorded no change for the 3 months.

Surface Drainage Conditions The lowering of the bridge by 1.26m reduces the effective waterway by 41%. This is not critical for this bridge since it will still have a satisfactory capacity for a 100 year storm. The stream did not enter the mine during or after mining. The fracture zone for the lithology at this site would be anticipated to extend approximately 70m above the mined coal seam and the shale units probably provided satisfactory seals for vertical movement of surface water.

CONCLUSIONS

The bridge after the subsidence, retained its structural integrity and adequately carries the required live loads. Normal traffic has been maintained continuously during and after the subsidence period. The only repair to the structure required as a result of mining is epoxy injection of the cracks. Although the bridge settled differentially 1.26m it was never out of service for the travelling public. Temporary bridge supports were installed for safety reasons under the bridge, but the supports never became functional nor was the safety of the travelling public ever in question.

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Chapter 106

MICROCOMPUTER SIMULATION OF ROCK BLASTING TO PREDICT FRAGMENTATION

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ABSTRACT

In order to reduce the complexity of mechanisms influencing rock fragmentation by blasting a simulation approach is proposed, using the capabilities of microcomputer interactive graphics.

Situations involving cratering and bench blasting are simulated, and both jointed and intact rocks can be subjected to explosive action, taking into consideration its breakage mechanisms.

Size analysis curves relating rock distribution before and after the blasts are determined and compared with field data.

INTRODUCTION

Productivity in rock blasting can be measured in terms of fragmentation, and Rock Mechanics has an essential role to improve it. In practice, distinction between good and bad blasting is established upon observation of resulting breakage, which is the most important outcome of the rock excavation process that can be evaluated, and judged upon the amount of resources expended to perform this operation. By definition, optimum fragmentation is the degree of breakage that minimizes the total cost of rock processing, including the successive unit operations of drilling, blasting, loading, hauling and crushing.

Despite its importance, rock fragmentation by blasting is still poorly understood by many engineers and rock mechanics experts, due to the complex mechanisms involved

in this phenomenon. Several contributions to explain and to predict fragmentation by blasting have been produced by many authors; Rustan, 1983, described seven different criteria proposed by various experts, showing distinct approaches and leading to different results, as far as predicting breakage is concerned.

A common aspect of those explanations is that fragmentation by blasting is considered a size reduction process in which several parameters are involved (powder factor is present in most criteria) but no relationship is established between breakage itself and the effects of detonating explosive charges within rock masses, taking into consideration the properties of these two entities and the geometric constraints of the problem.

Furthermore, although everyone agrees that the presence of natural joints and other types of discontinuities in the rock mass has a significant influence on breakage, none of the criteria seems to take into account this aspect. In an attempt to quantify that influence of joints in fragmentation, Gama, 1983, proposed an application of comminution theory, incorporating data on the rock mass fracture situation, in the mathematical expressions developed to predict fragmentation by blasting.

However, blasting mechanics are not yet included in the above criterion, though it seems to constitute the basics of any acceptable explanation to his question.

In order to attack the problem, one should start with knowledge on the behavior of the detonation process within rocks.

Many authors studied the sequence of events that happen around a borehole just after the detonation of an explosive takes place (Duvall and Atchison, 1957; Langefors and Kihlstrom, 1963; Hino, 1959; Coates, 1967, etc.).

In homogeneous rock, the essential reactions can be divided in three phases, as Figure 1 represents.

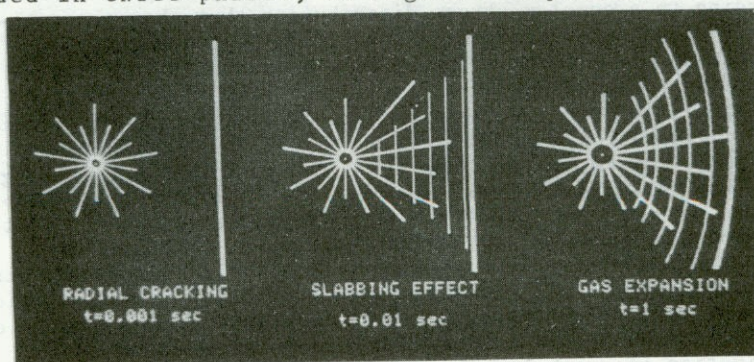


Fig. 1. Sequence of main events in the breakage process of rock blasting

Phase 1 - Immediately after the detonation of the charge, an outgoing shock wave produces crushing and radial cracking around the borehole. The velocity of radial crack propagation is a fraction of the wave velocity through the rock.

Phase 2 - As the compressive wave reaches the free face, it will reflect as a tension wave travelling back to the hole. Spalling or slabbing will occur, under the form of cracks parallel to the free face, when the dynamic tensile strength of the rock is exceeded by the tensile stress associated with the reflected wave.

Phase 3 - Expansion of explosion gases occurs, along previously opened cracks, enlarging and developing them, thus causing the major fragmentation mechanism of blasting. Gas expansion is mainly oriented towards the free faces, through the volume of loosened rock (previously fractured by the wave effects) which is the rock volume exerting less resistance around the borehole. There is little gas penetration in cracks that don't reach the free faces of the rock mass, and therefore no significant breakage can be ascribed to gas effects if there was no fracturing previously done by the shock wave, as in the case of deflagrant explosives.

If the rock mass contains joints, there are interferences between radial cracks, slabs and gaseous induced crack propagations, thus complicating the pattern of fractures within the rock volume, and consequently breakage will be greatly influenced by the attitudes and spacings of those joints.

As far as fragmentation is concerned, the following assumptions are proposed in a simplified model:

- a) The origin of rock fragments is the intersection of fractures developed by the explosive action.
- b) The origin of fractures is related to pre-existing flaws in the rock mass, especially around the borehole where the charge has detonated.
- c) These flaws are of three different types: very small Griffith cracks, fractures created by the drilling operation preceding blasting, and natural joints of the rock mass.
- d) The explosive action tends to enlarge and to propagate those flaws, according to a complex stress distribution that changes continuously with time. Gama and Nelson, 1970, have proposed the application of Griffith's theory of rock behavior in cases of failure by compression, to analyse conditions for initiation and propagation of pre-existing flaws around the charge hole, taking into consideration rock properties and explosive performance.
- e) When large joints are not present in the vicinity of the charge hole, the pre-existing flaws develop as radial cracks and slabs parallel to the free face, from which intersections the resulting fragment size distribution can be deduced.
- f) In greater detail, radial cracking depends on the location, size, and orientation of pre-existing flaws around the borehole, which can be determined by three different methods: after a survey of existing cracks before loading the holes; admitting a random distribution of crack lengths and orientations; or assuming a certain crack distribution in accordance to the intergranular bonds,

depending on the grain sizes of the rock. Bieniawski, 1967 has utilized this hypothesis to analyse brittle failure of rocks.

Using the above assumptions, it is feasible to simulate rock fragmentation by blasting in different situations of increasing complexity: cratering, bench blasting (both in continuous rock) and blasts in jointed rock.

SIMULATION OF CRATERING IN CONTINUOUS ROCK

When blasting is done by locating the explosive charge in the vicinity of one free face or rock, a crater is formed with conical geometry. Fig. 2 shows a scheme of cratering with its geometric parameters.

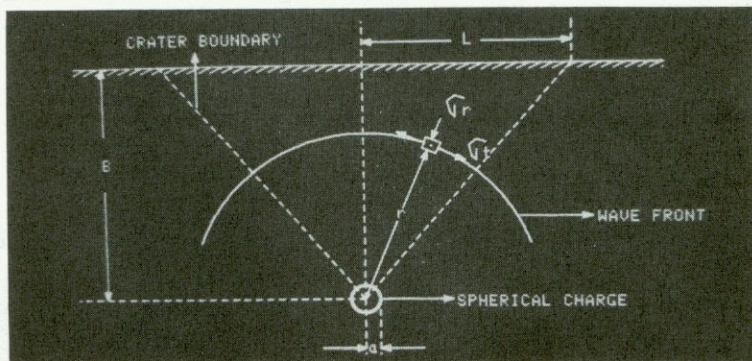


Fig. 2. Cratering geometry: charge radius a , burden of the charge B , and crater radius L .

The crater dimensions must depend on the rock and explosive properties, as well as the depth of the charge (its burden).

Due to the nature of fracturing mechanisms in blasting, radial cracks and slabs are formed by the emitted wave and further expanded by the gas expansion, as mentioned earlier. The crater radius is thus mainly governed by the slabbing effect which, according to a rock failure criterion based on the tensile stress carried by the reflected wave, assumes that slabbing finishes when the reflected wave pressure is less than the dynamic tensile strength of the rock. Calling this one T_0 and using the symbols of Fig. 2, we equate for the above condition:

$$T_0 = P_0 \left(\frac{a}{\sqrt{B^2 + L^2}} \right)^m$$

where P_0 is the detonation pressure of the explosive (*) and m is the attenuation factor of the wave propagation.

(*) Approximately it is given (Coates, 1981) by: $P_0 = \frac{\rho D^2}{4}$ where ρ is the explosive density and D its detonation velocity

The ideal crater radius is then given by:

$$L = [a^2 \left(\frac{P_0}{T_0}\right)^{\frac{2}{m}} - B^2]^{1/2}$$

an expression involving rock parameters (m and T_0), explosive properties (P_0 and a) and geometry of blasting (burden B).

It must be pointed out that slabbing itself is generally unable to form the crater, but it creates cracks that later on are developed by gas expansion, in order to reach final crater geometry.

It is interesting to note that the above expression provides the basis to calculate optimum burden (the one that maximizes crater volume) and critical burden (corresponding to zero crater volume) according to the well-known tests of Livingston, 1965. In fact, assuming that the crater volume is conical, its value is

$$V = \frac{\pi}{3} B L^2$$

and the maximum condition $\frac{\partial V}{\partial B} = 0$ leads to:

$$B_0 = \frac{a}{\sqrt{3}} \left(\frac{P_0}{T_0}\right)^{\frac{1}{m}}$$

as the optimum burden. On the other hand, $V = 0$ implies $L = 0$ and therefore

$$B_c = a \left(\frac{P_0}{T_0}\right)^{\frac{1}{m}}$$

is the critical burden. This suggests that

$$B_c = \sqrt{3} B_0$$

which is consistent with many cratering experiments in intact rock.

In a particular blast which burden is pre-established by the operator, we may thus evaluate if it is smaller or greater than the optimum burden, and the corresponding crater radius will change accordingly: in the first case there is an excess of explosive energy and the angle of cratering is assumed to be equal to that of optimum crater formation; for the second case, insufficient amount of explosive energy is supplied and the crater angle is less than the optimum, its radius being given by the expression for L .

Returning to the spalling mechanism it is possible to evaluate the thickness of the various slabs. According to Duvall and Atchison, 1957, the thickness of the first slab, assuming normal incidence, is:

$$t_1 = \frac{s}{2} \frac{T_0}{\sigma}$$

where s is the emitted wave shape length, T_0 is the dynamic tensile strength and σ the stress at the wave front after reflection.

We assume that s is twice the optimum burden because it is the most favourable condition to breakage, when the reflected wave reaches the borehole with enough energy to produce slabs. Therefore, the first slab has a thickness:

$$t_1 = \frac{B_0 T_0}{P_0 \left(\frac{a}{B}\right)^m}$$

and the successive slabs will be thinner due to multiple reflections of the stress wave at new free faces created by spalling (Rinehart, 1975).

So, the second slab thickness is:

$$t_2 = \frac{(B_0 - t_1) T_0 (B - t_1)^m}{P_0 a^m}$$

and the i slab:

$$t_i = \frac{(B_0 - d_i) T_0 (B - d_i)^m}{P_0 a^m}$$

where $d_i = t_1 + t_2 + \dots + t_{i-1}$

under the conditions $d_i \leq B - a$, and $d_i \leq B_0$.

In this way, we can estimate the thicknesses t_i of various slabs, in a fracture pattern parallel to the free face.

The other dimensions of each fragment result from the radial cracking mechanism of fracture, by means of which each slab will be broken in symmetrical shape, as Fig. 3 schematically represents.

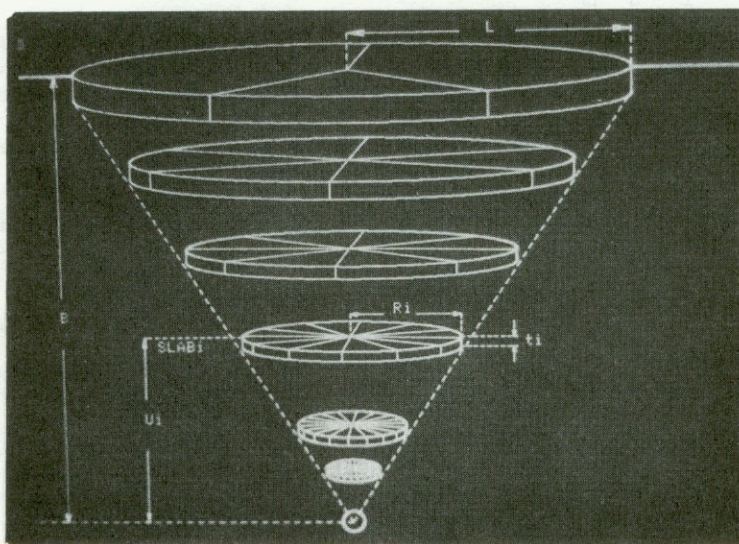


Fig. 3. Three dimensional representation of a crater with typical fragment shapes

In order to determine the number of radial cracks the criterion suggested by Coates, 1981 is utilized: their number equals the nearest integer to the ratio of tangential stress associated with the emitted wave and the dynamic tensile strength of the rock. As that tangential

or hoop stress is given by

$$\sigma_t = -P_o \left(\frac{a}{r}\right)^m$$

where the negative sign means tension, the number of radial cracks is:

$$N = \text{INT} \left[\frac{P_o}{T_o} \left(\frac{a}{r}\right)^m \right]$$

showing that it reduces as the distance r from charge center increases.

If this distance r is established in terms of multiples of slab thickness, it can be considered as U_i in Fig. 3, (with $U_i = B - d_i$) and therefore the number of radial cracks contained in the i slab is:

$$N_i = \text{INT} \left[\frac{P_o}{T_o} \left(\frac{a}{U_i}\right)^m \right]$$

This provides a means to calculate the size of fragments at any slab, because the volume of a fragment is given by:

$$V_i = \frac{1}{N_i} \pi R_i^2 t_i$$

where R_i is the slab radius (see Fig. 3):

$$R_i = \frac{L}{B} U_i$$

The size of any fragment can thus be estimated by the intermediate dimension of an enveloping prism, which three edges are: slab thickness t_i , slab radius R_i , and chord of the circular sector represented by the fragment basis.

With this information on each fragment shape, it is possible to obtain the size distribution curve of fragments from cratering blasts, and putting that sequence of calculations in a microcomputer, the size distribution curve is easily obtained for each combination of rock, explosive and depth of burden.

BENCH BLASTING IN CONTINUOUS ROCK

Multiple hole bench blasting is different from cratering not only in terms of symmetry, but also due to the mutual interference of adjacent charges, which act in the vicinity of two free faces.

In continuous rock blasts the two basic mechanisms (radial cracking and slabbing) are essential to breakage, particularly because they create conditions for gas expansion to penetrate and intersect other fractures, thus leading to a certain fragmentation pattern.

Fig. 4 shows a scheme of bench blasting with its principal characteristics, and using the above mentioned criterion of breakage by slabbing to define spacing between successive charges (simultaneously fired in the same row, as usual), we have:

$$S = 2L = 2 \left[a^2 \left(\frac{P_o}{T_o} \right)^m - B^2 \right]^{1/2}$$

where the symbols have the already described meaning. This expression may be used when no information on spacing between holes is given, and

in the absence of joints, supplying values in the range of the commonly utilized spacings from empirical blasting formulae.

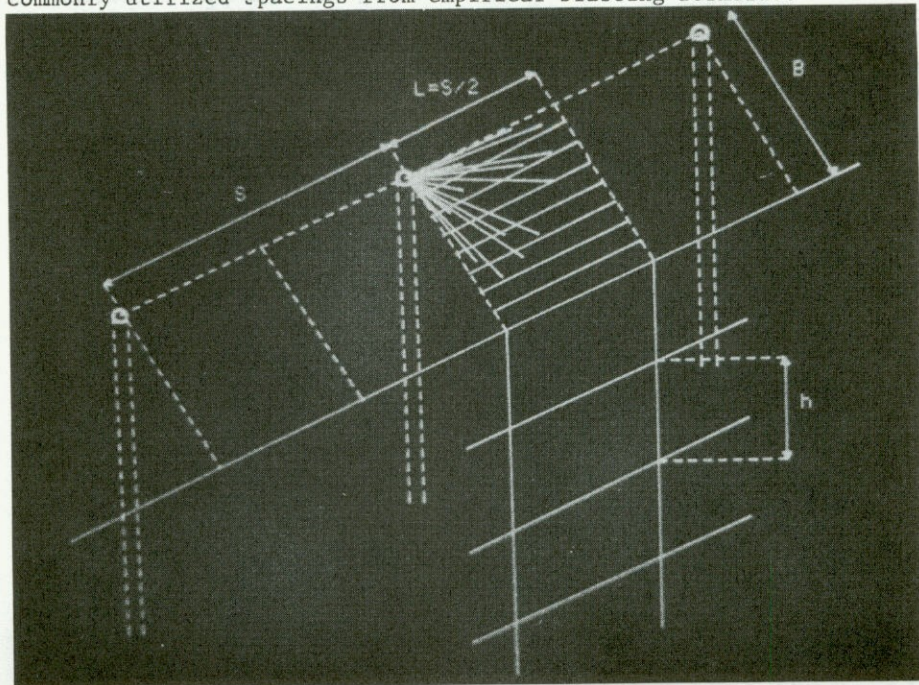


Fig. 4. Three dimensional view of bench blasting with its breakage pattern

After the blast a cut is obtained through the boreholes and the situation in terms of fragmentation can be studied in a quadrant around any borehole, because the same occurs in every similar rectangle. Under these circumstances the number of radial cracks in each quadrant is:

$$N_i = \text{INT} \left[\frac{P_o}{4 T_o} \left(\frac{a}{U_i} \right)^m \right]$$

and similarly, the thickness of each slab parallel to the free face is:

$$t_i = \frac{(B_o - B + U_i) T_o U_i}{P_o a^m}$$

as in the case of cratering. In order to evaluate the sizes of individual fragments, the horizontal section is divided in rectangles of width t (average slab thickness), and for each rectangle the number of cracks is computed by the above expression and individual block volumes and sizes are then calculated. When no cracks occur (or when $N_i = 0$) in contiguous rectangles of the same slab, the corresponding fragment width is obtained by adding t as many times as there are uncracked rectangles.

These two types of fractures will not define the fragment volumes, because a third dimension is required along the vertical direction. In order to evaluate this, we shall use the so called bending criterion (Ash, 1976) by considering the volume of blasted rock to be detached as a beam subjected to a certain distributed pressure p represented by the explosive action (see Fig. 5).

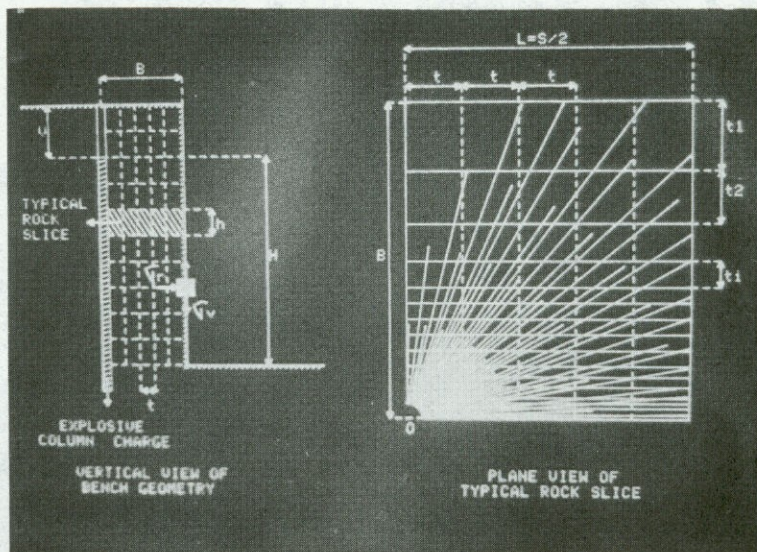


Fig. 5. Bench blasting geometry to define horizontal cracks for breakage

As the maximum tensile stress at the vertical free face can be obtained from beam theory (a beam with built-in ends submitted to uniform pressure $p = P_0/S$ per unit thickness) by:

$$\sigma_v = -p \frac{H^2}{4B^2}$$

where H is beam span, which is bench height less stemmed borehole length, and thus the number of horizontal cracks will be given by:

$$N_h = \text{INT} \left(\frac{\sigma_v}{T_0} \right)$$

Therefore, the height of each horizontal slice of the bench is:

$$h = \frac{H}{N_h} = \frac{4T_0 B^2}{p H}$$

with the symbols indicated in Figs. 4 and 5.

Assuming that the breakage situation repeats in each horizontal bench slice, typical fragments will be prismatic in shape, and their individual volumes will be:

$$V_i = h t_i \frac{L}{N_i} \quad (i=1,2,\dots,N_{\text{slabs}})$$

varying in size from the largest ones near the free face to the smallest in the vicinity of the borehole.

This method is easily programmed in a microcomputer, which outputs the size distribution curve of fragments from this type of blasting.

Breakage in the top of the bench, along the stemmed part of the borehole, can be estimated by the criterion proposed for cratering, and the corresponding fragments are included in the size distribution curve.

BLASTS IN JOINTED ROCK

Discontinuities in a rock mass completely change the fragmentation picture, because the explosive action (particularly the gas expansion stage) is controlled by the presence and attitude of these discontinuities. In practice this is the commonest situation, leading to a modification in the conventional breakage mechanisms, (such a Fig. 1 indicates) especially in the slabbing effect, which disappears in most cases (Ash, 1976; Lang and Favreau, 1972).

Also the conical symmetry of fragmentation in cratering blasts, and the cylindrical symmetry of bench blasting has no meaning when one or more joint families occur in the vicinity of the explosive charge. In essence, the problem is typically three-dimensional, and only in particular situations, depending on the spacings and attitudes of existing joints, it can accept axial or planar symmetry.

Generally, fragmentation from blasts in jointed rock is the result of intersections between radial cracks and the various pre-existing discontinuities located between the explosive charges and the free faces, and those intersections are initiated by the emitted stress wave (with secondary reflexions and refractions along such joints) and latter on they are extended and enlarged under the influence of the high pressure in the detonation gases.

To predict fragmentation under these circumstances is a task that requires a preliminary survey of discontinuity planes around the borehole, together with a set of data on the geometry of the blast, such as Fig. 6 represents.

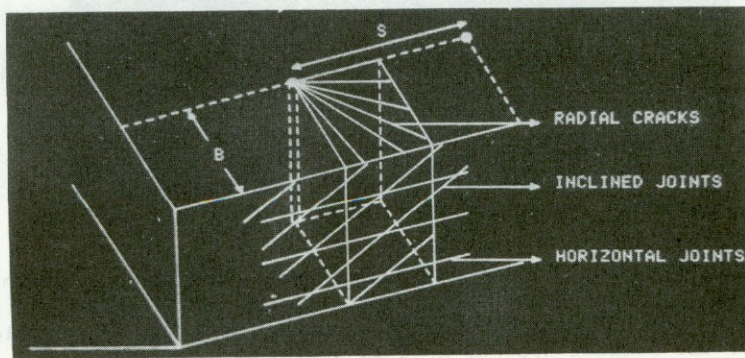


Fig. 6. Prediction of breakage for bench blasting in jointed rock

The mutual intersections of fracture planes and joints in a three-dimensional space can be accomplished through computer models such as COMPART (Gamaz, 1977). In this code, size distribution of blocks within a certain rock mass volume can be determined upon the input of information on dip and strike of each discontinuity, and coordinates of its position in space. The program defines rock elementary blocks by means of polyhedra that are created when four or more fracture planes cross each other. Its output is a size distribution curve, in terms of cumulative percent of weight of blocks passing a series of sizes, like a sieving operation.

Application of COMPART to actual problems is dependent upon an initial survey on existing joints and prediction of radial cracking density, which is a time consuming job, but afterwards is simply a matter of running the computer program.

MODEL VALIDATION

Agreement between theoretical predictions and practical results is necessary to assess the validity of any blasting model. For that purpose, several comparisons between simulation outcomes and cratering experiments in hard granite, conducted in the laboratory, were accomplished.

Using for parameter values of granite $T_0=20 \text{ MN/m}^2$ and $m=2$, and an explosive with $P_0=5000 \text{ MN/m}^2$, for small scale blasts with burdens from 15 to 30 mm, various size distribution curves were obtained and plotted against tests results (Fig. 7).

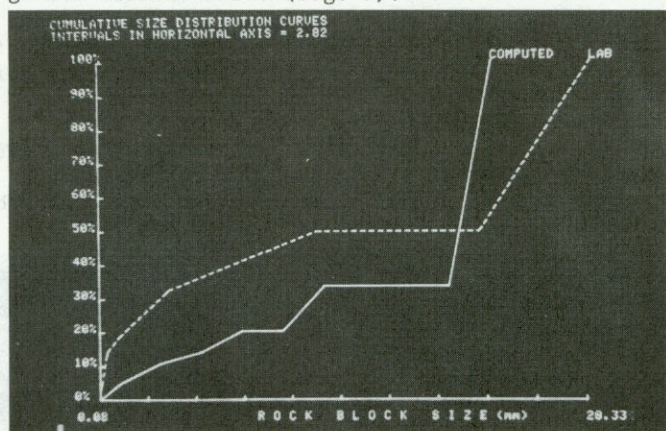


Fig. 7 - Size distribution curves of blasted rock in cratering experiments

In spite of the variability of experimental outcomes, the mathematical model has produced size distribution curves that match the general trend of the fragment grading, although it conducts to smaller values for the biggest blocks, as well as for the finest sizes.

In terms of bench blasting the difficulty of getting breakage data

for continuous rock is very serious, and comparisons were possible by the use of published results by Langefors and Kihlstrom, 1963 (Tables 2.4 and 2.5) for hard rock subjected to LFB dynamite ($P_0=9000 \text{ MN/m}^2$). Taking $T_0=15 \text{ MN/m}^2$ and $m=1.4$ for rock properties, and for the same burdens and spacings, we obtained the range of values of the following table.

Charge burden (m)	Powder factor (kg/m ³)	AVERAGE FRAGMENT SIZE (m)	
		Field results	Math. model
1.1	0.28	0.60	0.53
2.5	0.28	1.00	1.23
2.5	0.40	0.60	0.87

Again, the trend is observed, but unfortunately no size distribution curves were available from field blasts to improve the comparisons. More information is needed to confirm this model, both from laboratory and actual bench blasts in sound rock.

As far as jointed rock is concerned, comparisons were done against results of seven field blasts (Gama, 1983) showing reasonable agreement for the finer and average fragment sizes.

CONCLUSIONS

A predictive model of rock fragmentation by blasting was developed in microcomputer, with the purpose of explaining the breakage process upon knowledge of explosive action within rock.

Mathematical expressions provided the way to estimate fracturing densities of various types, and from these size distributions of fragments can be obtained.

The model seems to indicate the sensitivity of fragmentation with respect to the process variables (rock and explosive properties), thus allowing optimization studies by the computer, instead of expensive field tests.

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Chapter 107

PROBABILISTIC ANALYSIS OF BLASTING IMPACT ON OPEN PIT STABILITY

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ABSTRACT

A new probabilistic approach is introduced for slope stability analysis including adverse impact of blasting vibrations. The material strength within a slope area is randomized in this bi-variate joint probability analysis. To be a completely general engineering method, the new probabilistic approach employs the random variable transformation technique: the Hermite model of the Gaussian transformation function, which transforms the experimental histogram of strength values to the standard Gaussian variables. Because bi-normal joint probability is analyzed on the true probability region projected on the Gaussian transformed variable plane, it is an exact solution of slope stability based on the available sample data.

Details of the probabilistic approach are given through a case study on an iron ore mine. The case study demonstrated that the probabilistic approach is effective and simple enough to be a routine engineering practice for slope analysis.

INTRODUCTION

The probabilistic approach for slope analysis is a natural consequence of considering the actual field condition of the pit slopes rather than a deterministic method of yielding a single numerical indicator of the slope safety factor. A common method of the probabilistic approach is randomizing the strength values of slope materials (rocks) and assuming that the strength of the rock at a random location is a particular outcome of a random phenomena observed in the pit slope area. The strength value (random variable) is obtained from the triaxial test in terms of the internal friction angle (ϕ)

and cohesion (c). Therefore, the bi-variate joint probability analysis is necessary for slope analysis.

Another important aspect to be considered in these random variables is the spatial distribution within the pit slope area and its variability analysis over the space (5). An attempt was made unsuccessfully to define the spatial variability of random variables. Prominent research has not been done yet and further investigation should be made to explore the potential applicability of spatial variability for optimum pit analysis.

In this probabilistic approach the true probability region with a desired confidence level was employed to represent the strength condition of the entire pit slope (9). To define the probability region in the $(\phi-c)$ -plane, the law of bi-variate joint distribution of ϕ and c should be known. In general the bi-variate distribution is not known or is not definable with the limited sample data available.

Therefore, to be a more general and practical engineering method that can handle any type of distribution of strength values, the new probabilistic approach employs the variable transformation technique; the Hermite model of the Gaussian transformation function (1,4). Then, the probability analysis was made on the new standard variables (ϕ' and c') and the probability region was projected on the $(\phi'-c')$ -plane. The probability region draws concentric ellipses with various confidence levels, whose major axis is inclined at an angle of 45° from the horizontal coordinate axis due to equal variances ($\sigma_{\phi'}^2 = \sigma_{c'}^2 = 1$). The degree of distortion of the contour ellipse depends upon the correlation coefficient (ρ) and the ratio of two variances. The distortion indicates the magnitude of potential errors involved with the stochastic simulation method of safety factors.

The probability of failure or the failure zone projected on the probability region of a slope was determined from the back calculation of any slicing method of slope analysis with blasting vibrations. The back calculation generates a series of necessary strength values (pairs of ϕ and c values). The necessary strength values transformed and projected onto the $(\phi'-c')$ -plane, will draw the critical boundary line on the probability region and will define the failure zone. The final probability of slope failure then can be obtained simply by integrating the volume of the bi-normal distribution surface above the failure zone.

The adversity of blasting vibrations on slope stability was included in the probability analysis through the back calculation of the pseudo-static equilibrium analysis. The probabilistic approach is flexible enough to replace any dynamic or static method of slope analysis. Any governing factors that will dominate slope stability can be analyzed in probabilistic terms such as pore pressures.

Details of the new probabilistic approach were described with numerical examples from an iron ore mine. As demonstrated in the case study, it is very effective and simple enough to be a routine engineering practice. An additional advantage of the approach is a graphical presentation of whole conceptual pictures of the probability analysis thus making it easy to understand the slope condition.

PROBABILISTIC APPROACH

Ultimate Pit and Its Model. A geological cross section map was idealized in Fig. 1, which shows the current and ultimate pit profiles. Iron ore is classified into 5 different types for milling and blending purposes based on net iron content and base minerals. The ultimate pit wall at the highest point is 1,600 ft (487.7 m) high and inclined at a 45° angle. The opposite side of the pit is 1,350 ft (411.5 m) high with a 35° slope angle, which is the dip angle of bedding planes in iron formations.

Residual shear strength (ϕ and c) was obtained from triaxial tests on 48 core samples taken from each type of ore as well as from the footwall slate (waste rock). Also, cores are oriented in different angles with the bedding planes to verify their influence on the shear strength.

As summarized in Table 1, statistics of shear strength were not varied significantly with different types of rocks. It leads to the conclusion that the entire pit model for slope analysis can be built in a rock of unique strength, and 48 sample data can be considered as a random variable from a population. The correlation coefficient (ρ) between ϕ and c -values was -0.34 , which indicates a poor correlation.

Gaussian Transformation. The distribution of ϕ and c -values are given in the form of an experimental histogram which are (often) too complex to be correctly represented by a simple two or three-parameter statistical distribution model. To be general in the experimental histogram shape, the new probabilistic approach employs the Gaussian transformation function to transform the experimental random values (ϕ and c) to the standard normal variables (ϕ' and c'), whose distribution is normal with zero mean and variance of one.

The Gaussian transformation function (T_ϕ) of a variable ϕ which follows any distribution with a finite variance can be expressed in the form:

$$T_\phi(x) = \sum_{i=0}^{+\infty} \frac{\psi_i}{i!} H_i(x) \approx \sum_{i=0}^n \frac{\psi_i}{i!} H_i(x)$$

and approximated with a limited expansion of order n ,

where: $H_i(x)$ = the Hermite polynomial in the i th order and
 ψ_i = the coefficient of the Hermite expansion

The coefficients ψ_i are known from the Hermite polynomial integration of either numerical or graphical methods (1). Then, the transformation of ϕ to ϕ' is expressed in $\phi = T_{\phi}(\phi')$.

The further mathematical theory of the Hermite orthogonal polynomial function is referred to in Backmann (1), and its application and modeling is well described in Matheron (4) and Journel (3).

The Gaussian transformation function is, in practice, expanded up to the order of n until the Hermite polynomial function is fitted well to the experimental histogram. In other words, the experimental histogram is modeled with an n -parameter distribution function rather than 2 parameters as in the normal distribution function.

Transformation of ϕ -values. This was done by fitting the Hermite polynomial of order 18 to the experimental histogram of ϕ -values. The accuracy of this transformation was shown in Fig. 2, where the upper class bound of experimental histograms was plotted against those of the Hermite model fitted. The difference between them was less than 1.4% and a good agreement was observed. The accuracy of the transformation was also confirmed by the statistics of the Gaussian transformed variable (ϕ') as given in Table 2, where the theoretical parameters, zero mean and variance of one, are observed well.

Transformation of c -values. A satisfactory Gaussian transformation of c -values was achieved with the Hermite expansion of order 14. The result was plotted in Fig. 3 which indicates less than 1.7% difference between the experimental histogram and the Hermite polynomial model. The scattergram of ϕ' and c' was plotted in Fig. 4 and a poor correlation coefficient ($\rho' = -0.31$) resulted as expected from the experimental variables. The transformed variables ϕ' and c' are almost perfectly normal, so the bi-normal probability analysis can be made exactly.

Bi-Variate Normal Probability Analysis. The bi-normal joint probability region can be specified with a certain confidence level as follows; in general

$$\frac{(x_1 - \mu_1)^2}{\sigma_1^2} + \frac{(x_2 - \mu_2)^2}{\sigma_2^2} - 2\rho \frac{(x_1 - \mu_1)(x_2 - \mu_2)}{\sigma_1 \sigma_2} = (1 - \rho^2) \chi^2(R_2)$$

where: x_1, x_2 = bi-variate normal variables
 μ_1, σ_1 and μ_2, σ_2 = means and variances
 ρ = correlation coefficient
 $\chi^2(R_2)$ = chi-square variate with 2 degrees of freedom

By use of simple analytic geometry it can be noticed that the probability region in the (x_1, x_2) plane is an ellipse centered at the point (μ_1, μ_2) and oriented at an angle of θ with the x_1 -axis: $\theta = 45^\circ$ due to equal variances ($\sigma_1 = \sigma_2 = 1.0$).

The probability region defines that the point (x_1, x_2) , representing a random observation from a normal population, has a specified probability of falling in this region. This probability is equal to the probability that a chi-square variate with 2 degrees of freedom (R_2) has a value not exceeding the value selected for χ^2 .

For example, by choosing $p\{\chi^2 \leq R_2 (4.605)\} = 0.90 = 90\%$, the above equation defines the 90% probability region with $(1 - \rho^2) \times (4.605)$. Geometrically, this means that the volume of the three-dimensional region between the bi-variate normal surface and the (x_1, x_2) -plane, bounded laterally by the right elliptic cylinder based on the ellipse, is equal to 0.90. The statistical implication is that if we draw many independent observations at random from a bi-variate normal population and each time determine a point with coordinates (x_1, x_2) given by the paired observation, then, 90% of these points will lie in the region $R_2 (4.605)$, that is, inside of the ellipse (see Fig. 5).

For the probabilistic approach of slope analysis, the probability region function is further simplified:

$$(\phi')^2 + (c')^2 - 2\rho' \phi' c' = R_2 (1 - \rho'^2)$$

The orientation angle $\theta = -45^\circ$ or 45° from the negative horizontal axis due to the equal variances and negative $(-\rho')$.

This is a true probability region with a desired confidence level $\{\chi^2 (R_2)\}$, and any probability analysis made on this region is an exact solution for the slope without making any assumptions.

Probability of Failure and Critical Line. The probability region described above represents the true strength condition of a pit slope in probabilistic terms based on the experimental sample values. It implies that if samples are taken at random from the slope area and their strengths are tested, the probability of shear strength values falling inside of the elliptic probability region is the desired confidence level. Therefore, the true probability of slope failure (p_f) can be defined by locating the critical boundary line, which will divide the probability region into two zones; stable and instable zones.

This critical boundary line can be obtained from the back calculation of any limited equilibrium analysis of slope stability (9). The back calculation made on the critical failure surface with a safety factor of design criteria generates a series of necessary strength values for the slope. This is the strength required to maintain the design safety factor. Then, these necessary strength values are

transformed by the Gaussian transformation function used for experimental histograms, and plotted on the probability region in the (ϕ', c') -plane. The transformed necessary strength values will describe the critical boundary line on the (ϕ', c') -plane.

Finally, the probability of slope failure (p_f) at a desired risk level can be obtained simply by integrating the bi-normal distribution surface above the instable zone bounded by the probability contour ellipse and the critical line. The integration was done by a numerical slicing method with the subroutine MDBNOR from the IMSL Library (2).

Dynamic Impact Analysis of Blasting. The adversity of blasting vibrations on pit slopes was quantitated through the back calculation of the pseudo-static equilibrium analysis. Consequently, the necessary strength values or the critical boundary line is altered with the peak particle acceleration of the ground motion induced by blasting (see Fig. 5).

In the probabilistic approach, the dynamic impact can be expressed in two different ways depending on the slope design criteria: (1) the final probability is given in a single probabilistic term with a critical boundary line as the response of a slope described in a deterministic term, and (2) if the slope response is expressed in a marginal range at a certain risk level, the slope stability is given in a range of probability corresponding to the band of the critical boundary projected on the (ϕ', c') -plane. In either case the final condition of slope stability will be determined in probabilistic terms that can be read off directly from the probability chart of the slope (Fig. 6).

Probability of Slope Failure. The case of the iron ore mine is described in this section as a numerical example to illustrate details of the probability analysis.

Blasting was monitored at 42 locations throughout the pit by two seismographs. The peak particle velocity-scaled distance chart was developed in Fig. 7, which shows the regression line and confidence band at a risk level of 90%. By using the Fast Fourier transfer (FFT) function, the vibration record was converted to frequency spectrums as shown in Fig. 8. The dominant frequency range observed in these frequency spectrums was 25-55 cps and its mode and mean were 40 cps. After the calculated peak particle accelerations by a sinusoidal wave function with 40 cps were compared with those of FFT frequency spectrums, the peak particle acceleration calculated from the velocity record was used in the pseudo-static equilibrium analysis.

A probability region of a 90% confidence level for the mine was plotted on the (ϕ', c') -plane as shown in Fig. 5, which shows the critical boundary lines of various peak particle accelerations (k) in terms of the gravity acceleration (g). The probability of potential

slope failure was obtained from the volume integration about the instable zone and plotted in Fig. 6. The design safety factor used in the back calculation was 1.2. Also, the sensitivity of safety factor on the seismic coefficient (k) was plotted on Fig. 9 for various slope heights. By comparing these results the following conclusions are reached.

CONCLUSIONS

1. The probabilistic approach for slope stability analysis is general in the distribution of rock strength parameters (ϕ and c), which are randomized for the probability analysis. This is an important advantage over the simple mathematical distribution models or the stochastic simulation of safety factors used currently.

2. The bi-variate joint probability is analyzed for the slope, which is projected on the true probability region of a desired confidence level (or risk level). Therefore, it is an exact solution to the slope failure probability. The advantage of bi-variate joint probability against the stochastic simulation method can be illustrated graphically as shown in Fig. 5. The elliptic probability region of $\rho = -0.31$ for the iron ore mine will be a perfect circle, if the independency ($\rho = 0$) between ϕ and c is assumed as in the stochastic analysis. The degree of distortion of the ellipse depends on ρ and the ratio of two variances (σ_1 and σ_2).

3. The dynamic impact from blasting is included quantitatively through the critical boundary line, which is determined by the pseudo-static back calculation method. As given in Fig. 5 and 6, a systematic sensitivity test was made with various seismic coefficients (k). In this way the impact of blasting on slope stability is shown graphically. The probability of slope failure was directly related with blasting and can be incorporated into the field blasting design. In other words the risk level of blasting and slope failure were quantitated in probabilistic terms at a desired confidence level.

4. For the final pit slope of the iron ore mine, the factor of safety decreased from 2.91 to 1.2 of the design criteria with $k = 0.73$ g. Corresponding probabilities of potential slope failure with a 90% confidence level were increased to 50% as expected. Whenever the peak particle acceleration exceeds 0.73 g, the probability of potential slope failure will be higher than 50% of the critical condition at a 10% risk level. This 0.73 g is equivalent to the peak particle velocity of 1.12 in/sec (28.5 mm/sec) and the scaled distance of 4.4 (ft/lb^{1/2}) from the chart in Fig. 7.

5. As a 5% risk level is accepted on the peak particle velocity-scaled distance chart, the final pit failure probability will be in the range of 20.5%-85.0% rather than 50%. It means that one out of 20 blasts will induce the ground vibration that can increase the

failure probability beyond 85.0% with a confidence level of 90%, as 4.4 (ft/lb²) was limited.

6. Recently the natural frequency of structures (residential) and its phase angle or the propagation wave length were emphasized to determine the blasting damage criteria (7,8). However, little is known about these impacts on slope stability. Specially it is true when the deterministic design criteria of slopes and the quantitative prediction of blasting damage on slopes are concerned.

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Table 1. Statistics of Shear Strength Parameters

Ore Type	Shear Strength				No. of	Specific
	ϕ (degrees)		c (psi)		Samples	Weight
	Mean	σ	Mean	σ	Tested	lb/cu ft
A	45.66	1.47	378.3	75.7	7	204.6
B	45.19	3.14	299.3	107.9	10	204.6
C	43.46	1.23	238.5	60.8	8	204.6
D	42.95	4.21	269.3	115.6	4	204.6
E	43.33	1.32	326.4	64.5	11	204.6
Slate	44.78	2.91	279.4	86.8	8	200.0
Global	44.29	2.62	301.1	95.2	48	
Correlation coefficient between ϕ and c: $\rho = -0.34$						

Table 2. Statistics of Gaussian Transformed Variables, ϕ' and c'

	ϕ' -value	c' -value
Mean	0.001	0.003
Variance	1.008	0.996
Correlation coefficient between ϕ' and c' : -0.31		

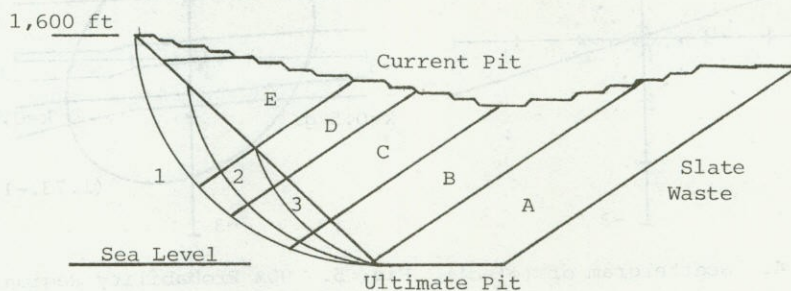


Fig. 1. Geological Cross Section of Ultimate Pit.

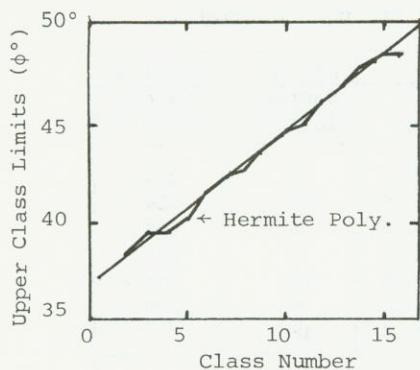


Fig. 2. Comparison of Experimental Histogram and Hermite Fitting for ϕ -Values.

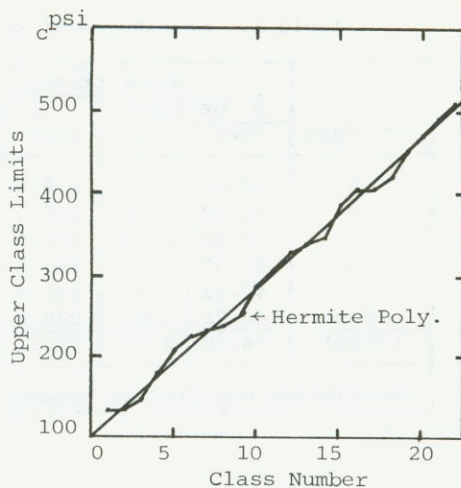


Fig. 3. Comparison of Experimental Histogram and Hermite Fitting for c -Values.

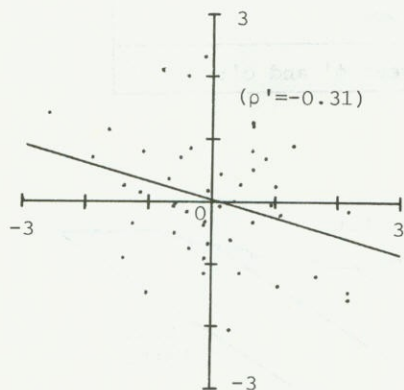


Fig. 4. Scattergram of $(\phi' - c')$.

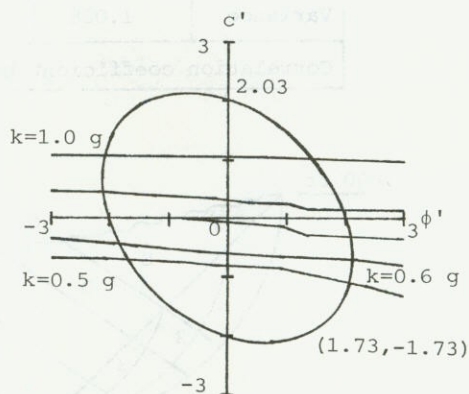


Fig. 5. 90% Probability Region and Critical Lines With k 's.

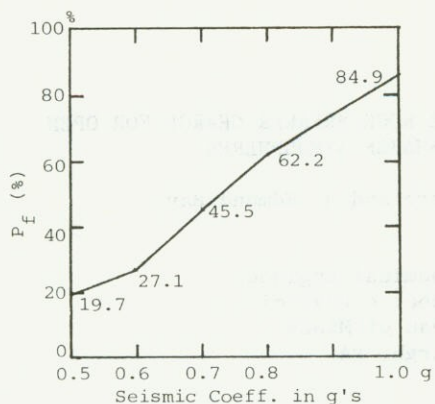


Fig. 6. Probability of Slope Failure With Various k 's.

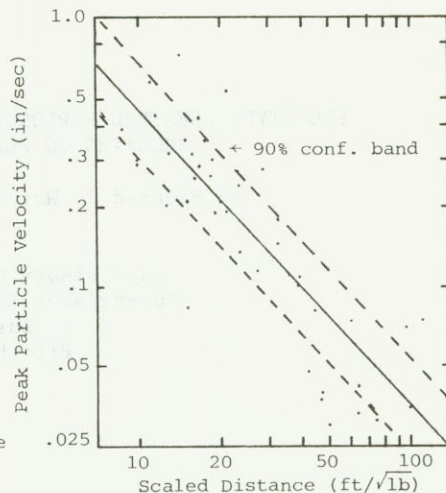


Fig. 7. Peak Particle Velocity - Scaled Distance.

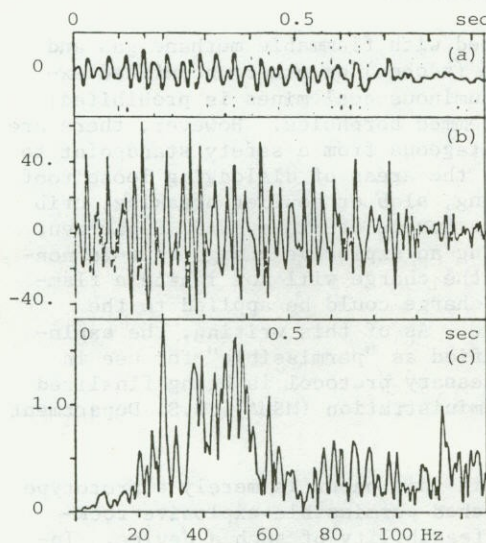


Fig. 8. (a) Seismogram, (b) Fourier Spectrum-Time, (c) Fourier Spectrum-Freq. Domain ($\text{in/sec}^2/\text{B.W.}$).

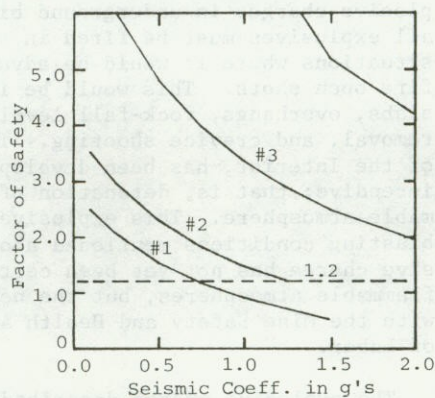


Fig. 9. Safety Factors With k 's for Different Slope Heights.